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GEOTECHNICAL REPORT
Valley City SW Backslope Slide Repair,
2-094(185)290 PCN 23338
VALLEY CITY, NORTH DAKOTA



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Submitted To: North Dakota Department of Transportation
608 East Boulevard Ave.
Bismarck, North Dakota 58505
Attn: Brent Flaa, P.E.

Subject: GEOTECHNICAL REPORT, VALLEY CITY SW BACKSLOPE SLIDE REPAIR,
2-094(185)290 PCN 23338, VALLEY CITY, NORTH DAKOTA

We are pleased to submit this report for the above-referenced project. This report presents our geotechnical findings and was prepared by the undersigned. Our scope of services was specified in Contract Number 19211442 with the North Dakota Department of Transportation (NDDOT) dated April 25, 2022.

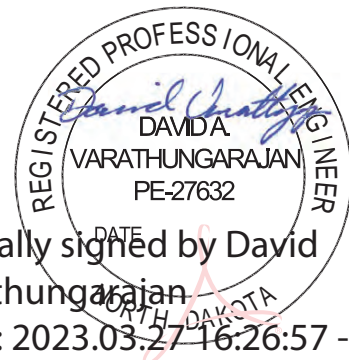
We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us at (303) 825-3800.

Sincerely,

SHANNON & WILSON, INC.



Luke Strom
Senior Geotechnical Staff



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EXECUTIVE SUMMARY

Landslide activity has been affecting the southwest side of Interstate 94 at the Exit 290 Interchange (approximately MP 290.65). Currently, the toe of the landslide is flowing into an approximately 400-foot-long portion of the ditch adjacent to the offramp, impacting drainage and requiring periodic maintenance to maintain functionality of the ditch. The most recent maintenance project to mitigate slide activity took place in 2019 and consisted of grading and installing erosion control. However, landslide movement has continued since then.

The purpose of this study is to characterize the landslide and to develop conceptual designs for mitigation alternatives. Shannon & Wilson completed subsurface explorations and installed instrumentation to characterize the landslide and developed the following mitigation alternatives:

- Maintenance,
- Drainage Improvements,
- Grading,
- Ground Anchors and Blocks,
- Anchored Drilled Shafts.

Due to the relatively minor impacts of the landslide, a maintenance approach may be acceptable to NDDOT. To compare maintenance costs with mitigation costs, we estimated the net present value for the maintenance approach and compared these values to relative construction costs for the mitigation alternatives. Our evaluation indicates that the maintenance costs are generally less than or comparable to relative construction costs for the grading options and less than structural options. However, selection of the preferred alternative by NDDOT may consider other factors, such as the desire not to complete routine maintenance, the risk of more frequent maintenance being required than we assumed, and the desire to limit landslide impacts to adjacent property.

If mitigation is desired, we anticipate that grading would be most cost-effective alternative. However, this alternative requires acquiring adjacent property. Alternatively, the landslide could be mitigated within the existing right-of-way using ground anchors and blocks. Anchored drilled shafts are not recommended for the site due to constructability considerations associated with constructing a suitable temporary work platform on the landslide.

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1 INTRODUCTION

This report presents the results of our site reconnaissance, subsurface exploration program, and geotechnical engineering recommendations for the Valley City Southwest (SW) Backslope Slide Repair (the Project) in Valley City, North Dakota. The report summarizes our field reconnaissance findings, subsurface explorations, laboratory testing, and geotechnical engineering studies, and presents conclusions and recommendations for slide mitigation alternatives. Our services were conducted in general accordance with our Contract No. 19211442 with the North Dakota Department of Transportation (NDDOT) dated April 25, 2022.

We understand this report will be used for selecting the preferred slope instability mitigation approach for the Project. This report should not be used for other purposes without Shannon & Wilson's review. Our scope of services included:

- Phase I Activities
 - Developing a project-specific Quality Control/Quality Assurance (QC/QA) plan to be instituted and used for the duration of the Project.
 - Attending a virtual Field Review meeting with NDDOT Materials & Research.
 - Evaluating existing geotechnical information provided by NDDOT and from nearby projects completed by Shannon & Wilson.
 - Retaining a subconsultant (KLJ) to conduct a survey of the Project site for the purpose of developing cross-sections for slope stability analysis.
 - Conducting a geologic field reconnaissance and landslide mapping.
 - Developing a scope of services for Phase II.
- Phase II Activities
 - Completing ten geotechnical borings to depths ranging from approximately 26 to 70 feet.
 - Installing inclinometer casings in nine of the ten borings to monitor potential slope movement.
 - Installing a total of seven vibrating wire piezometers (VWPs) in six of the borings to characterize groundwater conditions.
 - Installing time domain reflectometry (TDR) wires in three of the borings to monitor potential slope movement.
 - Completing geotechnical engineering analyses of feasible slope stability improvements.
 - Developing a list of potential mitigation alternatives, completing preliminary analyses to evaluate the feasibility of potential mitigation alternatives, and completing a qualitative comparison of the mitigation alternatives.

- Preparation of this geotechnical report.

Our conclusions and recommendations are based on the following:

- The limitations of our approved scope, schedule, and budget described in our contract;
- Our understanding of the Project and information provided by NDDOT;
- Subsurface conditions observed in the explorations at the time they were completed;
- The results of laboratory testing performed on samples collected from the explorations; and
- The data obtained from monitoring of instrumentation installed in select borings during the monitoring period.

The objective of our geotechnical studies was to provide recommendations and construction considerations, as presented herein, for the proposed slide repair. The authorized scope of services was based on this objective and this report should not be used for other purposes without Shannon & Wilson's review. If a service is not specifically indicated in this report, do not assume that it was performed.

2 PROJECT DESCRIPTION

The Project is located in Valley City, North Dakota on the southwest side of Interstate 94 (I-94) at the Exit 290 Interchange (approximately MP 290.65). The proposed project consists of mitigating the landslide adjacent to the eastbound exit ramp (see Figures 1 and 2; report tables and figures follow the main text). Based on discussions with the NDDOT, review of Google Earth Pro historic imagery, and our field reconnaissance, we understand that the landslide complex that encompasses the slope has been creating slope stability issues for at least 30 to 40 years. The landslide complex is relatively large with the following approximate dimensions: 1,500 feet long (as measured generally along the I-94 alignment) by 550 feet wide. Currently, debris from the landslide is flowing downslope into an approximately 400-foot-long portion of the ditch adjacent to the offramp and impacting drainage, requiring maintenance to preserve functionality of the ditch.

The NDDOT is considering mitigating the landslide to reduce the need for continuous maintenance and to provide a long-term solution to address slope stability and improve drainage at this location.

3 SITE DESCRIPTION

3.1 Site History

As discussed in Section 2, the slide is located adjacent to (south of) the eastbound exit ramp of I-94 at the Exit 290 interchange. Based on historic images on Google Earth Pro and other historic imagery, the slide showed signs of movement in 2010, when many scarps formed within the slide mass. Images from 2016 show further deformation near the toe of the slide, approximately 70 feet south of the offramp. Generally, the slide appears to be creeping north towards the interstate and offramp over the last 12 years.

The most recent documented maintenance project was completed in 2019 and consisted of removing landslide debris from the ditch, grading to flatten the upper part of the slope, placing erosion control, and clearing an existing underdrain aligned with the ditch (NDDOT, 2019). The re-graded portions of the slope ranged from 3H:1V to 5H:1V (horizontal to vertical). In addition, NDDOT installed three inclinometers to monitor future potential slope movement.

Based on conversation with NDDOT, we understand the eastbound offramp was realigned in the past to avoid slide debris that was impacting the previous offramp. Historic aerial images from Google Earth show this realignment occurred sometime between 1984 and 1997.

Portions of the slide complex are located outside of current NDDOT right-of-way (ROW), primarily in a privately owned quarry/stockpile property (referred to herein as the quarry) south of I-94 and west of 24th Avenue SW (see Figure 2).

3.2 Field Reconnaissance and Surface Conditions

In March 2022, two field representatives from Shannon & Wilson completed a site reconnaissance with representatives from the NDDOT. Key observations from the site visit are summarized below:

- The landslide complex appears to consist of multiple areas of active movement. The current geometry of the slide area suggests that the current landslide affecting the drainage ditch is a relatively more active portion near the center and toe of a larger landslide that encompasses the entire slope (see Exhibit 3-1). The head scarp of the larger landslide is more pronounced (approximately 20 feet tall) on the western side of the landslide complex. On the eastern side of the complex, the head scarp has largely been reworked and obscured by quarry operations at the top of the slope. However, historic imagery shows the head scarp extending into the current quarry property as

recently as 2010. The estimated landslide limits as well as internal features are shown on Figure 3.

- The lower portion of the graded slope has continued to fail since being regraded in 2019 (see Exhibit 3-2).
- The lower limit of the movement is characterized by an overridden toe feature with ponded water in parts of the drainage ditch (see Exhibit 3-3). Landslide debris is spilling over the toe of the failure and accumulating in the drainage ditch. Drilling the material that has spilled into the ditch was not possible due to access issues, but we anticipate that the thickness of the spilled-over debris is less than 10 feet.
- Hummocky terrain is present throughout the slide mass (Exhibit 3-3). The overall slope is relatively flat, approximately 3H:1V to 5H:1V (horizontal to vertical).
- Hydrophytic vegetation (cattails) and ponded water were visible in March 2022 (and in June 2022 during drilling) at several locations within the landslide footprint (see Figure 3).
- An embankment in the quarry behind the landslide mass appears to potentially impound water at various times from a drainage area of approximately 30,000 square feet (see Exhibit 3-4).
- Outlets for three trench drains installed during the 2019 grading project are showing signs of displacement with broken concrete. Ponded water and ice at the drain outlet suggests water may still be seeping from the drains.



Exhibit 3-1: Site Reconnaissance Photo 1 - Overview of the landslide complex taken from the north side of Interstate 94, looking south. Green fabric on the slope is an erosion control blanket/turf reinforcement mat (TRM) placed during a grading project in 2019.

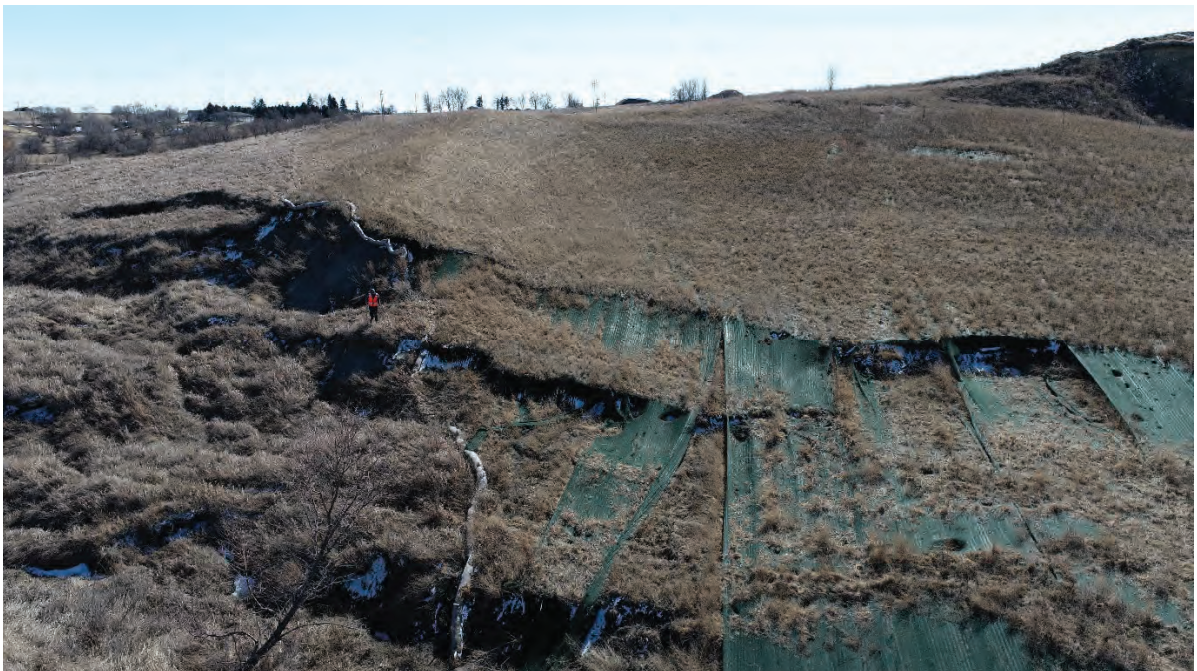


Exhibit 3-2: Site Reconnaissance Photo 2 - View of the stretched TRM (green fabric) and the disturbed soil erosion control log at the eastern grading tie-in point.



Exhibit 3-3: Site Reconnaissance Photo 3 - View of part of the slide mass with the overriding toe feature, facing west. Note the person in the orange vest in the upper left for scale.



Exhibit 3-4: Site Reconnaissance Photo 4 - View of the slide mass with location of embankment and dry pond, facing west.

4 FIELD EXPLORATIONS AND LABORATORY TESTING

4.1 Field Explorations

Shannon & Wilson conducted a field exploration program to evaluate subsurface conditions within the slide mass. Ten borings, designated SW-01 through SW-10, were drilled at the site between June 13 and 16, 2022. Borings were drilled to depths ranging from approximately 26 to 70 feet below ground surface (bgs). Locations of the borings are shown in Figure 2.

Appendix A presents a discussion of the drilling and sampling procedures used to complete the borings, as well as the individual exploration logs and an explanation of the symbols and terminology used.

As part of our field explorations, we retained a subconsultant to survey the boring locations and top-of-ground elevations, as well as the topography of the slope. The surveyed northing, easting, and top-of-ground elevation for each boring are included on the boring logs in Appendix A.

4.2 Laboratory Testing

Geotechnical laboratory tests were completed on selected samples retrieved from the borings to estimate index and engineering properties. Index tests included natural water content, in-place density, grain size analysis (sieve and hydrometer), and Atterberg limits. Engineering properties tests included corrosion tests and torsional ring shear tests of drained residual shear strength. Laboratory test results and a discussion of the testing procedures are included in Appendix B. The natural water content, fines content, and Atterberg limits are also shown on the individual boring logs included in Appendix A.

4.3 Instrumentation

Upon completion of drilling, instrumentation consisting of inclinometers, vibrating wire piezometers (VWPs), and time-domain reflectometry (TDR) wires was installed in the borings as detailed in Exhibit 4-1. NDDOT has completed periodic data collection from the slope and groundwater monitoring instrumentation.

Exhibit 4-1: Instrumentation Summary

Boring Designation	Inclinometer	TDR	VWP Installation Depth (ft)
SW-01	X	-	33.0 ¹
SW-02	X	-	45.0
SW-03	X	X	19.0 ¹
SW-04	X	-	-
SW-05	-	-	25.0, 40.0 ^{1,2}
SW-06	X	-	-
SW-07	X	X	17.8
SW-08	X	-	-
SW-09	X	X	17.8
SW-10	X	-	-

Notes:

1 Installation included a datalogger for continuous monitoring. Periodic manual readings were taken at other VWP locations.

2 Boring SW-05 included two nested VWPs installed at the depths indicated above.

X = installed; - = not installed; ft = feet; TDR = time domain reflectometry; VWP = vibrating wire piezometer

As indicated in Exhibit 4-1, two VWPs were installed in boring SW-05 to monitor groundwater conditions above and below the estimated depth of slide movement.

Appendix C presents a discussion of the instrumentation installation procedures, calibration data, and the readings obtained during monitoring.

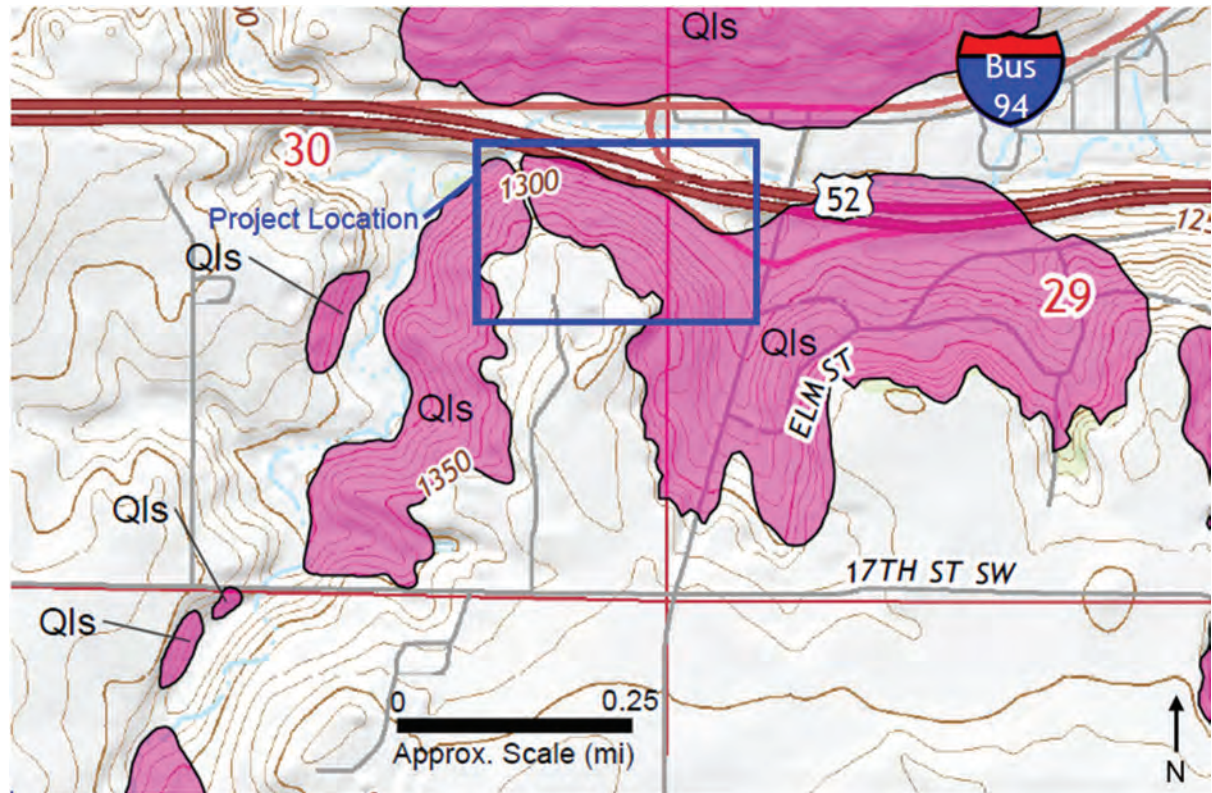
5 REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

A review of the Surface Geology - Valley City West Quadrangle, North Dakota (Manz, 2008) indicates that the surficial geology at the site consists primarily of Quaternary-age glacial and/or preglacial landslide deposits overlying Cretaceous-age Pierre Formation shale (referred to on the boring logs and figures as Pierre Shale). Quaternary-age glacial till is also indicated in the vicinity of the Project site. The Pierre Formation is described as “dark gray, fissile, non-calcareous shale ... Prone to slope failure.” Landslide mapping from the North Dakota Geological Survey (Anderson, 2020) indicates that the Project area is part of a much larger landslide complex (see Exhibit 5-1). This complex, and other nearby slide complexes,

are observed along several of the slope faces to the south and southeast of the upland agricultural fields.

Exhibit 5-1: Landslide Map of Project Site



NOTES:

- 1 Adapted from Anderson (2020).
Qls = landslide deposits

5.2 Subsurface Conditions

5.2.1 Overburden

Overburden at the site consisted of three geologic units: till, landslide debris, and displaced bedrock. These three units were grouped as the overburden unit for our analyses (see Section 6.3) due to the similarity in engineering properties.

5.2.1.1 Till

Till at the site typically consisted of very loose to medium dense, silty to clayey sand and clayey gravel with sand. A boulder was encountered within this unit during drilling of boring SW-01. In general, the till unit was encountered within the upper portion of the slope and ranged in thickness from approximately 12.5 to 18.0 feet, where encountered.

5.2.1.2 Landslide Debris

Landslide debris at the site typically consisted of soft to stiff, fat clay (occasionally lean clay) with variable sand and gravel content. Claystone fragments were observed in some of the samples. Granular layers, consisting of a thin lens of saturated sand and a 2-foot-thick sandy gravel layer, were observed within the unit in boring SW-07. The thickness of the landslide debris varied from 4.0 feet near the top of the slope to 23.0 feet within the lower third of the slope.

5.2.1.3 Displaced Bedrock

A zone of displaced claystone and/or shale was encountered in borings SW-01, SW-02, SW-03, and SW-09. This bedrock may have been displaced by previous landslide activity in the area, but is likely part of the current slide activity as well, based on our observations of the inclinometer monitoring (see Section 5.4). The displaced bedrock was differentiated based on tilting of the laminations at angles up to 60 degrees and inclinometer movement. In addition, we observed slickensides (which are indicative of landslide movement) within the displaced claystone in boring SW-02, at depths ranging from 39.5 to 51.0 feet bgs. The bottom of the displaced bedrock ranged from 4.0 to 51.0 feet bgs in the borings where it was encountered, generally varying according to the position of the boring on the slope. The displaced claystone is not a part of the intact bedrock and is therefore included in the overburden.

5.2.2 Bedrock

Bedrock was encountered at the site in all of our borings and consisted of extremely weak to very weak claystone and shale of the Pierre Formation. The claystone and shale observed was generally thinly bedded to laminated and fresh to moderately weathered.

5.3 Groundwater

During drilling, groundwater was encountered in borings SW-01, SW-03, and SW-05 through SW-09. Depth to groundwater in these borings was measured at 0 to 31.3 feet bgs (at boring SW-08, ponded water was observed above the ground surface). The groundwater level at the time of drilling is noted on the boring logs included in Appendix A.

No measurable groundwater was encountered in the other borings at the time of exploration. However, groundwater observations during drilling may not be representative of actual groundwater conditions, particularly for clayey soils, such as those present at the site, in which it may take several days for water levels to equilibrate in a borehole. As such, we used the VWP's installed in the borings to monitor longer-term groundwater conditions.

The data from the VWP's indicate relatively shallow groundwater in the overburden near the toe of the slope, which is consistent with observations of ponded water and hydrophytic vegetation in that location. The overburden and bedrock groundwater regimes appear to be hydraulically separate; however, there may be portions of the subsurface where the overburden and bedrock groundwater are connected through fractures in the bedrock.

Groundwater fluctuations are likely and will depend on seasonal variations, local precipitation, and runoff.

5.4 Inclinator & TDR Monitoring

Inclinometers were installed in all borings except for SW-05, where installation could not be completed due to access issues related to the steep terrain at that boring location. As a supplement to the inclinometers, we installed TDRs at borings SW-03, SW-07, and SW-09. Since the installation of the inclinometers and TDRs in June 2022, NDDOT obtained periodic readings through November 3, 2022. A summary of the movement observed in the inclinometers is provided in Exhibit 5-2.

Exhibit 5-2: Summary of Inclinator Data

Inclinator Designation	Movement Observed (Y / N)	Depth of Movement (feet bgs)	Elevation of Movement (feet above sea level)	Geologic Unit
SW-01	Y	30.8 - 34.8	1,131.7 – 1,327.7	Displaced Bedrock / Bedrock contact
SW-02	Y	51.2 - 53.3	1,306.6 – 1,304.5	Displaced Bedrock / Bedrock contact
SW-03	Y	6.7 - 8.7	1,307.4 – 1305.4	Displaced Bedrock / Bedrock contact
SW-04	N	N/A	N/A	N/A
SW-06	Y	9.1 - 11.1	1,313.0 – 1,311.0	Landslide Debris / Bedrock contact
SW-07	Y	20.3 - 24.3	1,313.8 – 1,309.8	Landslide Debris / Bedrock contact
SW-08	N	N/A	N/A	N/A
SW-09	Y	16.6 - 20.6	1,307.5 – 1,303.5	Displaced Bedrock / Bedrock contact
SW-10	N	N/A	N/A	N/A

N/A = Not Applicable

The data obtained from the inclinometers installed within the mapped slide mass (SW-01 through SW-03, SW-06, SW-07, and SW-09) indicate a discernable slip surface. The slip surface generally coincides with the overburden-bedrock contact.

Interpretation of TDR data can be challenging because while the reflection data can provide information about the location of movement along the cable, they do not indicate the magnitude, direction, or rate of movement. In addition, the cables are sensitive enough to

disturbance to produce 'noise' in the data that may resemble data indicating slide movement and depend on the cable deforming in a manner that produces relatively strong signal reflection. However, based on side-by-side comparisons of the observed inclinometer movements and the TDR reflection data, both instruments appear to indicate slide movement at the same or similar depths. We observed pronounced TDR reflections in boring SW-03, where the inclinometer data indicated substantial slide movement (on the order of 6 inches in 5 months). Reflections were not as well-defined in borings SW-07 and SW-09, where the inclinometer data showed less movement (on the order of 1.5 inches and 1 inch in SW-07 and SW-09, respectively). Plots showing the inclinometer-TDR comparisons are provided in Appendix C.

5.5 Subsurface Profiles

We prepared subsurface profiles of four potential critical cross-sections (locations indicated on Figure 2) to illustrate representative subsurface conditions at the site. The subsurface profiles are presented in Figures 4 through 7. The subsurface profiles include graphical logs of nearby borings, groundwater levels measured by the VWPs, our interpretation of geologic contacts, and our interpretation of the slip surface geometry.

5.6 Subsurface Variations

The explorations were performed to evaluate subsurface conditions at the Project site. Our observations are specific to the locations, depths, and times noted on the logs and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to:

- The conditions between explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.

If conditions different from those described herein are encountered during construction, we should review our description of the subsurface conditions and reconsider our conclusions and recommendations.

5.7 Corrosion

The soil encountered at the Project site can be corrosive to substructure elements. To assist in estimating the corrosion potential at the site, select samples were tested for pH, resistivity, water soluble sulfates, and chlorides. The results are presented in Table B-1 in Appendix B.

AASHTO (2020) Section 10.7.5 defines soil as aggressive for steel if resistivity is less than 2,000 ohm-centimeters, pH is less than 5.5 (or between 5.5 and 8.5 in highly organic soils), or sulfate concentrations are greater than 1,000 parts per million (0.1 percent by weight). The laboratory test results for pH ranged from 7.6 to 8.3, and the soils did not classify as organic. By this criterion, the soils do not classify as aggressive for steel. However, the resistivity measured in the samples ranged from 340 to 1,330 ohm-centimeters, and sulfate concentration ranged from 0.01 to 1.70 percent by weight, both of which are indicative of aggressive soil conditions for steel.

Recommendations developed by the American Concrete Institute (ACI, 2019) provide guidelines for characterizing sulfate exposure for concrete. The sulfate concentrations measured in the samples correspond to an exposure class of S0 to S2.

The test results and the above discussion are provided to assist the design team in the selection of project materials, concrete type, or other features with respect to corrosion. As appropriate, the designer should consider protective measures, such as coatings, upsizing for section loss, or using alternative materials to reduce the corrosion potential.

6 SLOPE STABILITY MODEL AND BACK-ANALYSIS OF LANDSLIDE

To assess potential landslide mitigation options (see Section 7), we used the cross-sections subsurface profiles discussed in Section 5.5 to develop slope stability models for the landslide. Because the slope has moved significantly, the factor of safety (FS) of the slope can be considered to be 1.0 at the time of failure (Duncan and Wright, 2005). Assuming a FS value of 1.0, a stability model that is representative of conditions at the time of failure can then be developed and used as a baseline for comparison with various landslide mitigation alternatives. The results of the back-analyses are shown in Appendix D.

6.1 Site Topography

The topography of the existing ground surface used in our stability model was developed from the cross sections produced by KLJ as part of their survey, provided to us on August 16, 2022 and January 9, 2023. From the survey information, we selected four cross-sections we considered critical based on variations in site topography and failure surface geometry (A-A', B-B', C-C', and D-D'). We used these cross-sections to develop our slope stability back-analyses. We evaluated the mitigation alternatives for each cross-section to a target FS value of 1.3. Back-analyses for each section are included in Appendix D.

6.2 Slip Surface Geometry

We estimated the geometry of the slip surface based on the depths of movement indicated in the inclinometers and TDRs, as well as the materials encountered in our borings. As discussed in Section 5.4, the slide movement depth corresponded with the intact bedrock contact. The estimated slip surface geometry is shown on the subsurface profiles in Figures 4 through 7.

The zone of slip surface movement as indicated by the inclinometer data ranged in thickness from approximately 2 to 4 feet. For our slope stability analyses, we assumed a uniform slip surface thickness of 2 feet.

6.3 Soil/Rock Material Properties

The material properties we used in our analyses are summarized in Exhibits 6-1, 6-2, and 6-3. We estimated unit weights based on the available laboratory data and our experience with similar materials in North Dakota. For the overburden material and intact bedrock which have not experienced shear deformations resulting in a residual condition, we estimated a representative fully softened friction angle based on correlations with liquid limit (LL) and clay fraction (CF) by Stark and Fernandez (2020), back-analysis (see below), and our judgement.

Exhibit 6-1: Soil/Rock Parameters Used for Stability Analyses

Unit	Typical Description	Total Unit Weight, γ (pcf)	Effective Stress Friction Angle, ϕ' (deg)	Effective Stress Cohesion, c' (psf)
Overburden	Soft to very stiff, Fat Clay to extremely weak, displaced Claystone	120	24	0
Slip Surface	Soft to stiff, Fat Clay to extremely weak, Claystone	120	Varies ¹	0
Bedrock	Extremely weak to very weak, highly weathered to fresh Claystone/Shale (Pierre Shale)	125	24	100

NOTES:

1 See Exhibits 6-2 and 6-3 for strength parameters used for the Slip Surface unit.
deg = degrees; pcf = pounds per cubic foot; psf = pounds per square foot

Bedrock strength parameters were conservatively selected to a near lower-bound value to preclude failure into bedrock below the interpreted slip surface based on existing failures occurring in overburden and comparatively weaker displaced bedrock. Intact bedrock strengths were further reduced on near-horizontal planes to assess the potential impacts of weaker planes in the bedrock. At the outset of this analysis, reducing the strengths to fully

softened values on near-horizontal planes produced failures that are not representative of field conditions. Therefore, a nominal reduction in strengths was applied for the anisotropic strength case based on the performance of the existing slopes and lack of failures in the intact bedrock.

To model the slip surface, we utilized a two-foot-thick layer of material at the location of the interpreted slip surface. As discussed in Section 4.2, we completed torsional ring shear testing on composite soil samples comprised of material taken from near the slip surface, to compare the residual shear strength parameters of the soil/rock to those obtained from the Stark and Fernandez (2020) correlation. The material we tested consisted of clay and/or claystone with LL values ranging from 129 to 215.

Back-analyses using the ring shear-derived residual strength envelopes indicated that an unrealistically low groundwater surface would be required to obtain a FS value of 1.0. Based on the back-analyses and our laboratory results, in our opinion the average plasticity of the slip surface is likely lower than that of the material used for the torsional ring shear composite samples. Therefore, we repeated the back-analyses using the correlation by Stark and Fernandez (2020) to estimate residual shear strength based on LL and CF values measured in soil samples within or near the interpreted slip surface. We used the following inputs for the analyzed cross-sections:

- Sections A-A' & C-C': CF > 50, LL = 80.
- Section B-B': CF > 50, LL = 75
- Section D-D': CF > 50, LL = 100.

The strength properties from the Stark and Fernandez (2020) correlation were input as a shear strength vs. normal strength function in the slope stability program. The shear-normal functions used in the analyses are shown in Exhibit 6-2 (with torsional ring shear test results included for comparison) and represented in terms of residual friction angle in Exhibit 6-3.

Exhibit 6-2: Residual Shear Strength Envelopes Used for Slip Surface

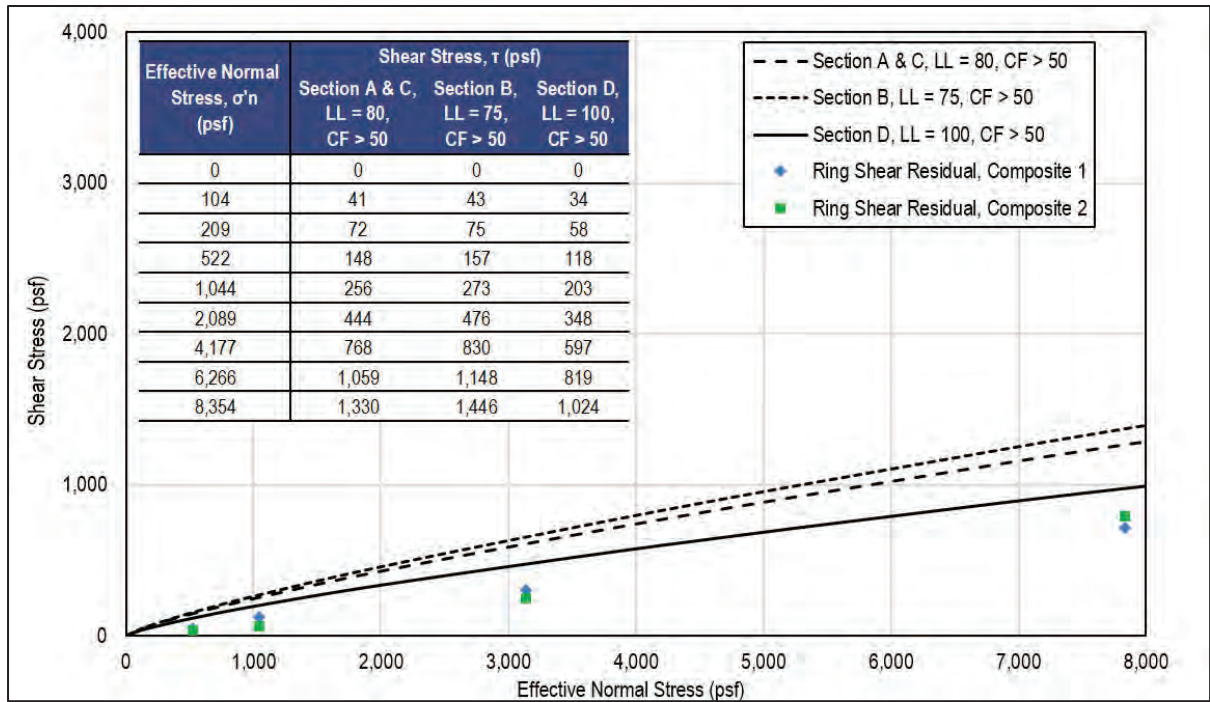
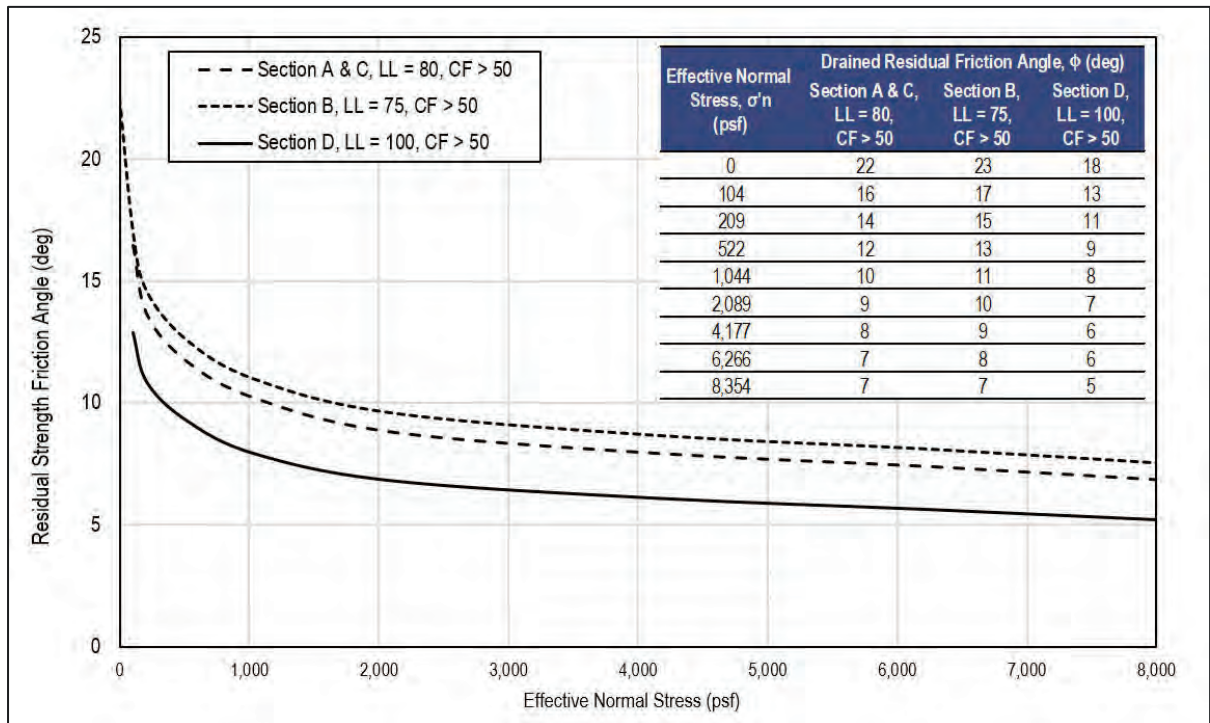


Exhibit 6-3: Residual Shear Strength Friction Angles Used for Slip Surface



6.4 Groundwater Conditions

We represented pore water pressures (i.e., groundwater conditions) by a groundwater table. The groundwater table we assumed for our analyses is shown in the provided slope stability output. It was initially developed based on groundwater levels observed during drilling and measured by the vibrating wire piezometers (VWPs) which are shown on the subsurface profiles (Figures 4 through 7). We then adjusted the groundwater table by a few feet until the model showed a FS value of 1.0 using the shear strength parameters discussed in Section 6.3. Compared to the groundwater levels indicated by the VWPs and our observations of ponded water at the surface, relatively minor changes were required to obtain a FS value of 1.0 for the cross-sections we considered.

6.5 Analysis Methodology

We completed two-dimensional limit-equilibrium stability analyses using the software Slide2, Version 9, by RocScience (2021), and Slope/W, Version 11, by GeoSlope (2021). We used the Morgenstern and Price (1965) method of slices, which satisfies force and moment equilibrium, to compute FS values. We assumed excess pore water pressures in the slope have equilibrated. Thus, we completed drained, or long-term, stability analyses. Based on the relatively low seismic hazard for the site, we did not consider seismic loading. The results of the slope stability analyses are presented in Appendix D and include the back-analyses as well as analyses of the mitigation alternatives discussed in Section 7.

6.6 Factor of Safety

We assumed a target FS value of 1.3 for mitigation alternatives in accordance with AASHTO (2020). The FS value of 1.3 specified by AASHTO (2020) for slope stability assumes that the FS is applied to a slope that has not yet been constructed or to an existing slope that has not failed. In part, the FS value is intended to reflect the uncertainty and variability of the inputs of the slope stability model, such as shear strength parameters and groundwater conditions. However, a slope stability model based on back-analysis of an existing landslide has essentially been calibrated to reflect actual conditions, i.e., a FS value of 1.0 at the time of failure. As such, the back-calculated model can be considered to be more reliable than a typical slope stability model based on data from borings and laboratory test results. Therefore, lower target FS values may be justified for mitigation of existing landslides (Duncan and Wright, 2005). The target FS value for final design should be determined by NDDOT. Based on discussions with NDDOT, we understand that a target FS value of 1.3 is desired for all mitigation alternatives.

7 POTENTIAL LANDSLIDE MITIGATION ALTERNATIVES

We evaluated several potential alternatives to improve the stability of the landslide. We evaluated improvement to the slope stability, risk, and cost of each improvement alternative as discussed in the following sections. We evaluated drainage improvements, grading improvements, ground anchor and block stabilization, and anchored drilled shafts. The following sections describe details of the proposed alternatives, and a summary of these alternatives is presented in Table 1.

As discussed in Section 2 and shown in Exhibit 5-1, landslide activity is extensive around the Project site. For our evaluation, we assumed that the length of mitigation will be approximately 800 feet, centered around the portion of the landslide toe that is closest to the EB I-94 exit ramp pavement and impacting the drainage ditch, i.e., we assumed that areas of the landslide not actively impacting the drainage ditch would not be mitigated. NDDOT may elect to increase the length of stabilization, depending on risk and cost considerations.

7.1 Maintenance

Depending on the availability of funding for improvements, NDDOT may elect not to implement the mitigation alternatives described in the following sections. This alternative would consist of periodic grading efforts and erosion control to clean out the ditch and remove encroaching landslide material. To decrease the frequency of grading, a pipe or culvert could be constructed along the ditch alignment, and the landslide material allowed to flow over the pipe/culvert. The existing underdrain installed below the ditch could also be augmented to provide additional drainage. However, the additional maintenance required for such features may offset potential benefits.

This alternative may be desirable due to the relatively low cost and the relatively minor impacts of the existing landslide. However, NDDOT should consider that the rate and magnitude of future movement is difficult to predict and will strongly depend on future precipitation. Landslide movement has necessitated maintenance work in recent years. Without mitigation, a similar or greater level of maintenance can be anticipated in the future.

To better estimate the actual cost of the maintenance approach over several years, we calculated the net present worth value of future maintenance operations. For this evaluation, we assumed a discount rate of 1%, which was assumed to be a “real discount” rate that deducts inflation. Therefore, present-day maintenance costs were used in place of assumed future (escalated) costs. Based on discussions with NDDOT, we assumed a

present-day maintenance cost of \$300,000 for a maintenance project that occurs every 10 years. As noted in Table 1, we considered 25-, 50-, and 75-year maintenance periods.

7.2 Drainage Improvements

Groundwater can play a significant role in slope instability. The VWP's installed at the site indicate relatively high groundwater near the toe of the slope, which is likely strongly influenced by infiltration in more permeable upland till deposits. Stability can be improved by lowering groundwater levels in the slope. Therefore, we recommend that drainage improvements be implemented with any of the following improvement alternatives.

While a limited improvement of the FS can be obtained through drainage improvements alone, NDDOT may consider implementing drainage improvements without other improvements as a relatively low-cost approach to decrease the frequency of required maintenance.

7.2.1 Surface Drainage

Appropriate surface drainage measures would consist of re-grading the slope to seal cracks and smoothing out the hummocky terrain to eliminate ponding and to direct runoff to a suitable location. The FS increase that could be obtained from these improvements is difficult to quantify, but we recommend that they be implemented regardless of the selected stabilization approach.

7.2.2 Subsurface Drainage

Gravel-filled finger drains extending from the middle of the slide mass to the toe of the slope could be used to drain ponded water and remove shallow groundwater from the slide mass. The finger drains could be allowed to daylight and flow into the ditch at the toe of the slope.

We anticipate that a drain design for the site would likely consist of finger drains, spaced approximately 50 feet center-to-center, extending from the middle of the slope to the toe of the slope (approximately 150 feet). To effectively lower groundwater levels, the finger drains would need to be installed to a depth of up to 15 feet to intercept the groundwater. The drains typically have a base width of about five feet.

We estimate that that subsurface drainage measures would be able to increase the FS of the slope by approximately 5%. As such, subsurface drainage measures would need to be combined with other measures to achieve a target FS value of 1.3, or a lower FS selected by

NDDOT. As with the surface drainage improvements, we recommend that subsurface drainage improvements be implemented, regardless of the selected stabilization approach.

7.3 Grading

In addition to the drainage improvements discussed in Section 7.2, a grading alternative could be implemented to improve stability and achieve a higher FS value than is possible with finger drains alone. The grading alternative would consist of cutting material from the top of the slope and using the cut material to construct a buttress fill slope at the landslide toe. This improves stability by decreasing the driving force of the slide and increasing the resisting force at the toe of the slope.

A summary of the preliminary estimated grading requirements for each alternative are provided in Exhibit 7-1, and schematics of the grading alternative are shown on Figure 8.

Exhibit 7-1: Grading Alternative Requirements for Target FS = 1.3

Approximate Cut Depth at Top of Slide (ft)	Approximate Buttress Fill Slope Height from Slide Toe (ft)	Approximate Cut Volume (CY)	Approximate Fill Volume (CY)	Approximate ROW Take (acres)
13-18	30-45	54,000	38,000	3.5

NOTES:

1 Grading quantities are approximate and only intended for comparison between alternatives.
ft = feet; CY = cubic yards

For the grading alternatives presented in Exhibit 7-1, we assumed buttress slopes ranging from 2H:1V to 3.5H:1V, depending on the distance between the bottom of landslide debris and the drainage ditch to minimize any fill placed in the drainage ditch. Where a 2H:1V slope was used (in Sections C-C' and D-D'), it was limited to the base of the slope adjacent to the ditch with a height on the order of 20 feet. Construction of a buttress sloped at 2H:1V would likely require benching into the slope to remove disturbed landslide debris such that the buttress bears on intact bedrock, as well as utilizing a reinforced soil slope and granular fill material (e.g., till material from the upper portion of the site). Benching and excavation near the toe of the slope in disturbed landslide materials will likely be challenging and require working in relatively small sections to limit adverse impacts to slope stability. Depending on the hydraulic and sizing requirements for the drainage ditch, it may be possible to utilize a flatter slope that could be constructed without reinforcement and with less benching/excavation, which would be preferable to placing fills steeper than 3H:1V. A culvert could also be considered to allow fill placement to encroach into the ditch. These details would need to be evaluated during final design.

In order to achieve the target FS value of 1.3, portions of the excavation at the top of the slope would extend beyond the existing NDDOT ROW, into the adjacent quarry property. Based on discussion with NDDOT, we assumed a ROW cost of \$30,000 per acre in our evaluation. These costs were included in relative cost estimates summarized in Table 1 and further detailed in Appendix E.

7.4 Ground Anchor and Block Stabilization

This alternative consists of adding stabilizing force to the landslide by installing row(s) of ground anchors in the face of the slope and extending behind the interpreted slip surface. The ground anchors would react against precast reinforced concrete anchor blocks installed on the face of the slope. Typically, these anchor blocks are approximately 8 to 10 feet square.

Construction typically requires grading an approximately 20-foot-wide bench for drill rig access, which could potentially result in decreased stability during construction. Additionally, over-excavation or ground improvement may be required to provide adequate bearing behind the blocks, particularly in disturbed landslide debris material. Upon completion, the anchor blocks can be partially or completely buried during final slope grading. This technique was used for a similar landslide mitigation for a nearby project on the north side of I-94.

Our preliminary analysis assumed three rows of ground anchors, located horizontal distances of approximately 30 to 40, 60 to 70, and 90 to 100 feet from the toe of the slope. For the anchors, we assumed 6-strand anchors, drilled at an inclination of 20 degrees below horizontal, with minimum bond lengths of 40 feet in intact bedrock, unbonded lengths ranging from approximately 35 to 85 feet (varying by row), maximum lateral spacing of 10 to 11 feet, and 8- to 10-foot square anchor blocks. Our preliminary analysis indicates this alternative yields a long-term static FS value of 1.3. Schematics of this alternative are shown on Figure 9.

7.5 Anchored Drilled Shafts

This alternative consists of stabilizing the landslide by installing drilled shafts through the slide mass in combination with a row of ground anchors installed through a concrete cap beam atop the drilled shafts. The ground anchors would reduce bending moments in the drilled shafts and provide additional stabilizing force. However, construction of drilled shafts requires large equipment and a relatively wide work area (30 feet or more). In general, the site geometry is not favorable for the construction of such a large bench on the slide mass, which would require significant earthwork, possible temporary shoring, and adverse impacts to slope stability during construction. However, once constructed, the anchored shafts system would provide the most robust resistance to deep-seated movement.

For this alternative, the structure would be designed to resist landslide loading. Additionally, design of the structure would need to consider the stability of the landslide mass in front of the drilled shafts and the potential for this ground to move away from the structure, resulting in a decrease in passive resistance.

Based on preliminary analyses, we assumed the anchored shaft system would be located a horizontal distance of approximately 60 to 70 feet from the toe of the slope. For the drilled shafts, we assumed a diameter of 4 feet and, a minimum length of 50 feet bgs, and a maximum lateral spacing of 11 feet (center-to-center). For the ground anchors, our analyses assumed 6-strand anchors drilled at an inclination 30 degrees below horizontal, with a lateral spacing of 11 feet. We assumed free lengths of 32 to 40 feet and a 40-foot-long bond

zone in bedrock. Our preliminary analyses indicate this alternative yields a long-term static FS value of 1.3. Schematics of this alternative are shown on Figure 10.

7.6 Relative Costs

We evaluated the relative costs of the mitigation alternatives and prepared preliminary estimates based on past project experience. Our preliminary estimates are summarized in Table 1 as both the total relative cost assuming an 800-foot mitigation length and the cost per foot of repair. Details of our estimates are provided in Appendix E. The estimated costs should be taken as order-of-magnitude estimates for the purpose of selecting a mitigation alternative and the landslide mitigation extents. They should not be used for any other purposes (such as preparing a construction cost estimate).

8 CONCLUSIONS

8.1 Summary of Alternatives

We considered several mitigation alternatives and evaluated each alternative based on geotechnical considerations, hydraulic impacts, cost, schedule, right-of-way impacts, and constructability to make a recommendation for a preferred alternative. Final selection of the mitigation should be made by NDDOT considering other factors such as funding availability, risk, and long-term maintenance preferences. A summary of the selection considerations for the mitigation alternatives is provided in Table 1 and discussed below.

Of the options we considered, the maintenance alternative (periodic minor regrading to maintain ditch functionality) has the lowest initial costs. If stability improvements are not implemented, the frequency of maintenance required to maintain functionality of the drainage ditch at the toe of the slope is not possible to predict. However, we anticipate that maintenance would likely be required on the order of every 10 years based on our experience and discussions with NDDOT (the maintenance may be more or less frequent depending on precipitation).

The alternative with the second lowest initial costs is surface and subsurface drainage improvements. We estimate that such subsurface drainage measures would be able to increase the FS to approximately 1.05. As such, subsurface drainage measures would need to be combined with other measures to achieve a target FS of 1.3, or a lower FS value if desired by NDDOT. Alternatively, drainage improvements could be considered on their own as a relatively low-cost option to decrease the frequency of maintenance at the site.

Additionally, we considered a grading/buttress alternative providing a FS value of 1.3. While more expensive than the maintenance or drainage alternatives, the grading alternatives are less expensive than structural mitigation approaches. However, grading improvements will require ROW acquisition at the top of the slope.

Finally, we considered two structural alternatives (ground anchors with blocks and anchored drilled shafts) which would provide a FS value of 1.3. These alternatives are the most expensive, schedule-intensive, and require specialty contractors. However, they can be constructed on the existing ROW. Due to constructability considerations associated with temporary work pad requirements, the ground anchor and block alternative is preferable to the anchored drilled shaft alternative.

8.2 Comparison of Relative Costs

We estimated relative construction costs for the purpose of comparing these alternatives. As discussed in Section 7.1, for the maintenance alternative, we estimated the net present value of future maintenance operations to better compare costs for the maintenance alternative with mitigation costs (we did not include potential maintenance costs that may be required for the mitigation alternatives).

Over an assumed 75-year period, the net present value for the maintenance alternative was approximately two times the relative cost for the grading alternative and approximately 30% of the relative cost for the structural mitigation alternatives. The estimated relative cost of the grading alternative is approximately 55% of that of the structural mitigation alternatives.

While costs alone would suggest that maintenance is the preferred alternative, other factors and considerations may govern NDDOT's selection of a preferred approach. In particular, the NDDOT may prefer the less frequent maintenance required for the mitigation alternatives and may desire to limit landslide impacts to adjacent properties.

8.3 Alternative Selection

As discussed in Section 8.1, the mitigation alternative selected by NDDOT will depend on a variety of factors including cost, risk, schedule, constructability, ROW impacts, and impacts to I-94 traffic. The selection evaluation may be considered as follows:

- First, NDDOT would need to decide whether the relatively low costs and impacts of the maintenance alternative make it an acceptable approach.
- If periodic maintenance is not acceptable, a grading or structural mitigation option would be required. If ROW can be acquired to accommodate grading improvements,

then grading would likely be preferable to structural options due to the lower relative cost.

- If ROW cannot be acquired or NDDOT prefers not to pursue additional ROW acquisition due to associated risks, the ground anchor and block alternative would be preferable to the anchored drilled shaft alternative due to the reduced temporary access and grading requirements required for the ground anchor and block alternative.

9 LIMITATIONS

This report was prepared for the exclusive use of the NDDOT. Within the limitations of scope, schedule and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice at the time this report was prepared. We make no warranty, neither express nor implied.

Subsurface conditions assumed for the analyses and preliminary designs presented herein were based on explorations completed by Shannon & Wilson. The design recommendations presented herein are preliminary and are based on the subsurface information available at the time this letter report was prepared. Additional analyses are recommended to develop final design recommendations for this project.

This report should not be used without our approval if any of the following occurs:

- Conditions change due to natural forces or human activity under, at, or adjacent to the site.
- Assumptions stated in this report have changed.
- Project details change or new information becomes available such that our recommendations may be affected.
- If the site ownership or land use has changed.
- More than five years have passed since the date of this report.

If any of these occur, we should be retained to review the applicability of our preliminary recommendations.

Shannon & Wilson has prepared "Important Information about Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

10 REFERENCES

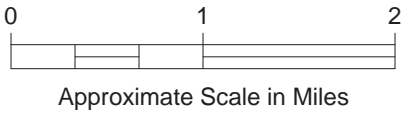
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Table 1. Summary of Alternatives

Alternative	Preliminary Design Information	Advantages	Disadvantages	Construction Schedule/Time	Relative Cost (Total) ¹	Relative Cost (per foot)
Maintenance	<ul style="list-style-type: none"> Re-grading and erosion control to maintain functionality of drainage ditch 	<ul style="list-style-type: none"> Specialty contractor not required. Relatively quick construction. Low cost option. No ROW impacts. 	<ul style="list-style-type: none"> No significant stability improvement. Similar or greater levels of maintenance may be required in the future. 	<ul style="list-style-type: none"> Quickest construction rate. 	<ul style="list-style-type: none"> \$600,000 (25 years) \$1,500,000 (50 years) \$2,000,000 (75 years) 	<ul style="list-style-type: none"> \$750 (25 years) \$1,875 (50 years) \$2,500 (75 years)
Drainage Improvements	<ul style="list-style-type: none"> Surface improvements, including sealing cracks, smoothing out hummocky ground to limit ponding. Finger drains up to 15 feet deep extending from middle of slope down to toe, spaced 50 feet on center. 	<ul style="list-style-type: none"> Specialty contractor not required. Relatively quick construction. Low cost option. No ROW impacts. 	<ul style="list-style-type: none"> Marginal increase in slope stability. Uncertainty about FS that could be achieved. 	<ul style="list-style-type: none"> Quick construction rate. 	<ul style="list-style-type: none"> \$1,100,000 	<ul style="list-style-type: none"> \$1,375
Grading ²	<ul style="list-style-type: none"> Sloped cuts at the top of the slide, up to 18 feet deep. Engineered fill slope buttress at the toe of the slide, up to 45 feet high. Buttress slope likely requires geosynthetic reinforcement. 	<ul style="list-style-type: none"> Specialty contractor not required. Relatively quick construction. Relatively low cost option. More certainty in FS that could be achieved. 	<ul style="list-style-type: none"> Temporary slope stability while excavating at toe of slide. Sequencing of excavation will need to be controlled. Significant hauling would be required to move excavated material. Material loading/unloading may require traffic control on I-94. ROW impacts where cut extends beyond existing ROW. In previous negotiations a ROW agreement could not be reached with the current adjacent landowners. This option could result in a costly condemnation. 	<ul style="list-style-type: none"> Quick to moderate construction rate. 	<ul style="list-style-type: none"> \$3,800,000 	<ul style="list-style-type: none"> \$4,750
Ground Anchors ²	<ul style="list-style-type: none"> Three rows of up to 125-foot long, 6-strand ground anchors spaced 10-11 feet on center. The rows are spaced approximately 30 feet apart and approximately 30-40 feet, 60-70 feet, and 90-100 feet horizontally from the landslide toe. Ground anchors installed through concrete reaction blocks (typically 8- to 10-foot square). 	<ul style="list-style-type: none"> More certainty in FS that could be achieved. Less construction equipment and hauling would be required; construction would have less interference with I-94. No ROW impacts. 	<ul style="list-style-type: none"> Requires specially ground anchor contractor. Moderate earth work required for access and to install anchors and reaction blocks. Over-excavation or ground improvement may be required to provide adequate bearing for reaction blocks. Relatively high cost option. 	<ul style="list-style-type: none"> Slow to moderate construction rate. 	<ul style="list-style-type: none"> \$6,800,000 	<ul style="list-style-type: none"> \$8,500
Anchored Drilled Shafts ²	<ul style="list-style-type: none"> Single row of drilled shafts installed within lower third of slope. 50-foot long, 4-foot diameter, spaced 11 feet on center. 6-strand ground anchors installed through cap beam. 	<ul style="list-style-type: none"> More certainty in FS that could be achieved. Less construction equipment and hauling would be required; construction would have less interference with I-94. No ROW impacts. 	<ul style="list-style-type: none"> Requires specially drilled shaft contractor. Requires specially ground anchor contractor. Significant/challenging earthwork required for construction access. Temporary shoring may be required. Highest cost option. 	<ul style="list-style-type: none"> Slow construction rate. 	<ul style="list-style-type: none"> \$6,900,000 	<ul style="list-style-type: none"> \$8,625

NOTES:

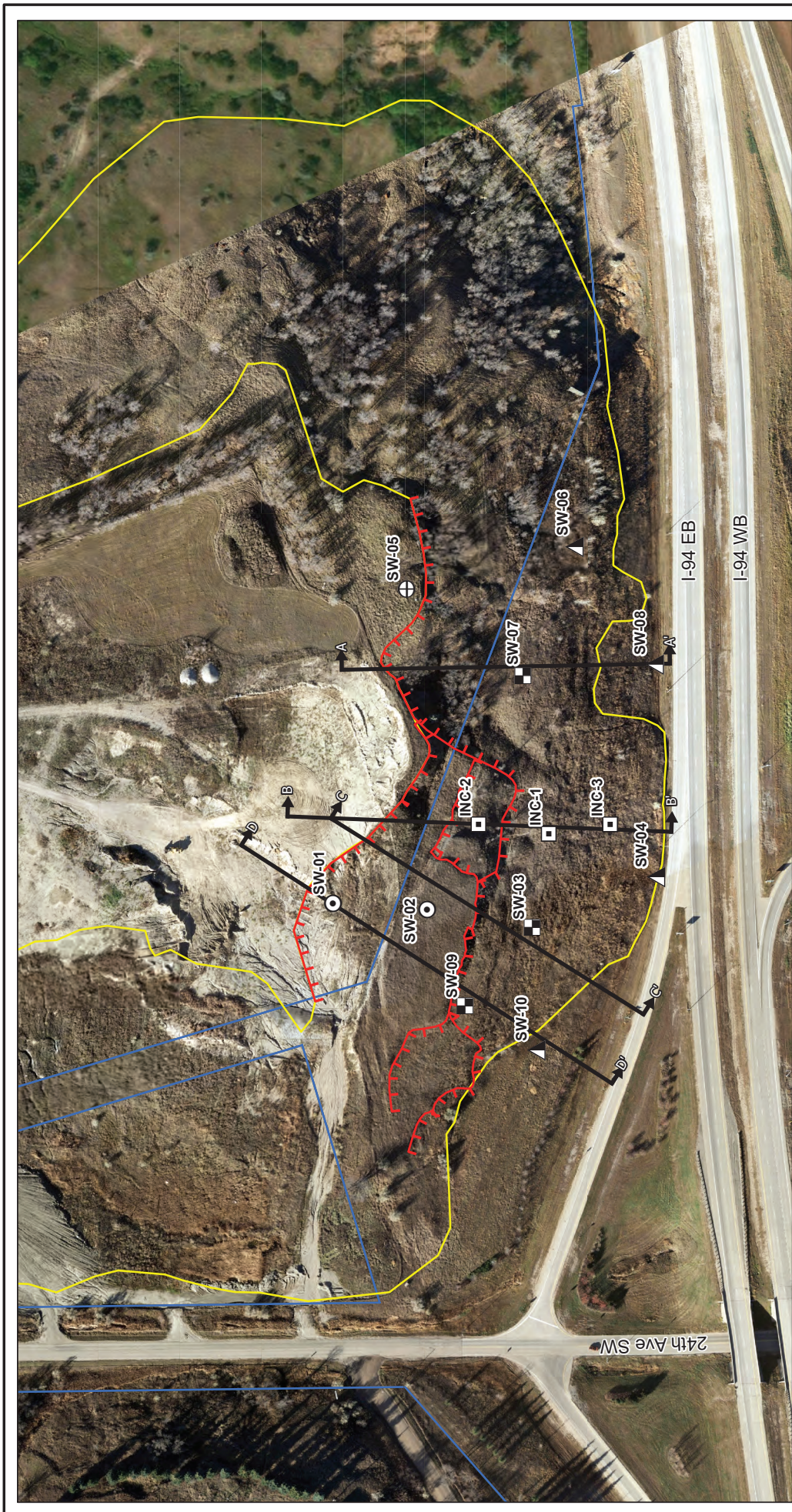
- Net present values were estimated for the maintenance alternative for the indicated assumed maintenance periods as described in the report. Costs for the other alternatives only include present construction costs and do not include potential future maintenance.
- Grading and structural mitigation alternatives include drainage improvements to reduce pore water pressures within the slope.
- See report text for more detailed discussion of each alternative.



NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.

Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
VICINITY MAP	
March 2023	107877-001
SHANNON & WILSON, INC. <small>GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS</small>	FIG. 1



Site of North Dakota, Est, HERE, Garmin, SafeGraph, GeoTechnologies, Inc., METI/NASA, USGS, EPA, NPS, USDA, NOAA, Microsoft

Valley City SW Backslope Slide Repair
2-094(185)290; PCN 23338
Valley City, North Dakota

SITE AND EXPLORATION PLAN
March 2023
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

107877-001
FIG. 2

LEGEND

- SW-05 VWP Designation
- SW-04 Inclinator Designation
- SW-01 Inclinator and VWP Designation
- SW-03 Inclinator, VWP, and TDR Designation
- INC-1 NDDOT (2019) Inclinator Designation
- B-B' Cross Section Lines
- Landslide Scarp
- Approximate Landslide Boundary
- Existing Right of Way

NOTES

- Borings completed by Shannon & Wilson were surveyed by KLJ in July 2022.
- Landslide boundaries were mapped using aerial imagery. Aerial imagery from 2010 was used to map the landslide boundary where it is buried.
- NDDOT (2019) inclinometer locations were provided by NDDOT and are approximate.



Valley City SW Backslope Slide Repair
 2-094(185)290, PCN 23338
 Valley City, North Dakota

March 2023
 107877-001

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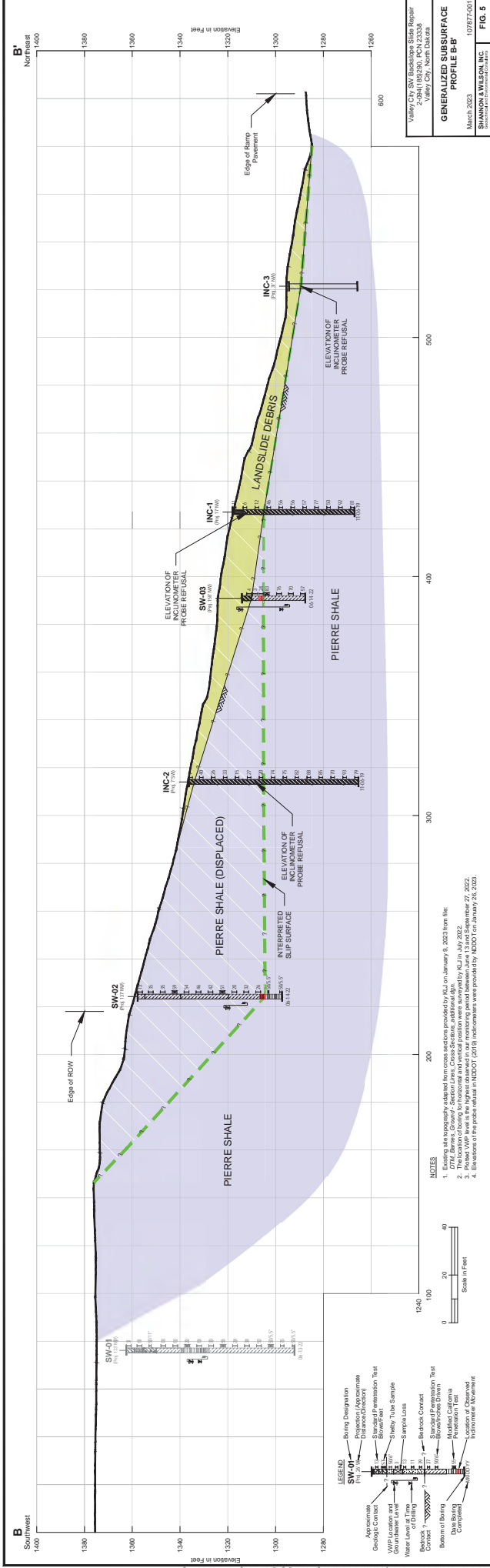
ANNOTATED SITE PHOTOGRAPH

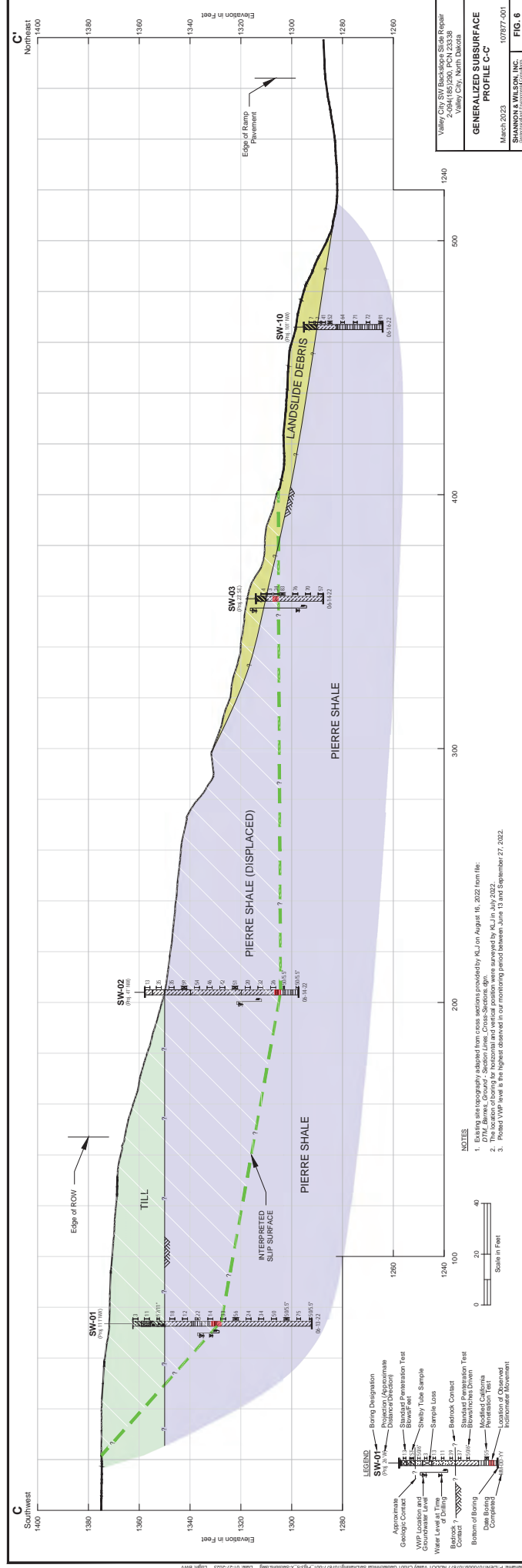
FIG. 3

FIGURE NOTES:

1. Photo taken on March 31, 2022. View due south from the north side of Interstate 94..
2. This figure schematically depicts mapped landslide-induced geomorphic features and other features.

FIG. 3







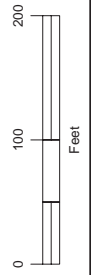
State of North Dakota, Dept. of Transportation, Geotechnical Services, Inc. (METS/NASA, USGS, EPA, NPS, USDA)

Valley City SW Backslope Slide Repair
 2-094(185)290, PCN 23338
 Valley City, North Dakota

SCHEMATIC OF GRADING AND DRAINAGE ALTERNATIVE

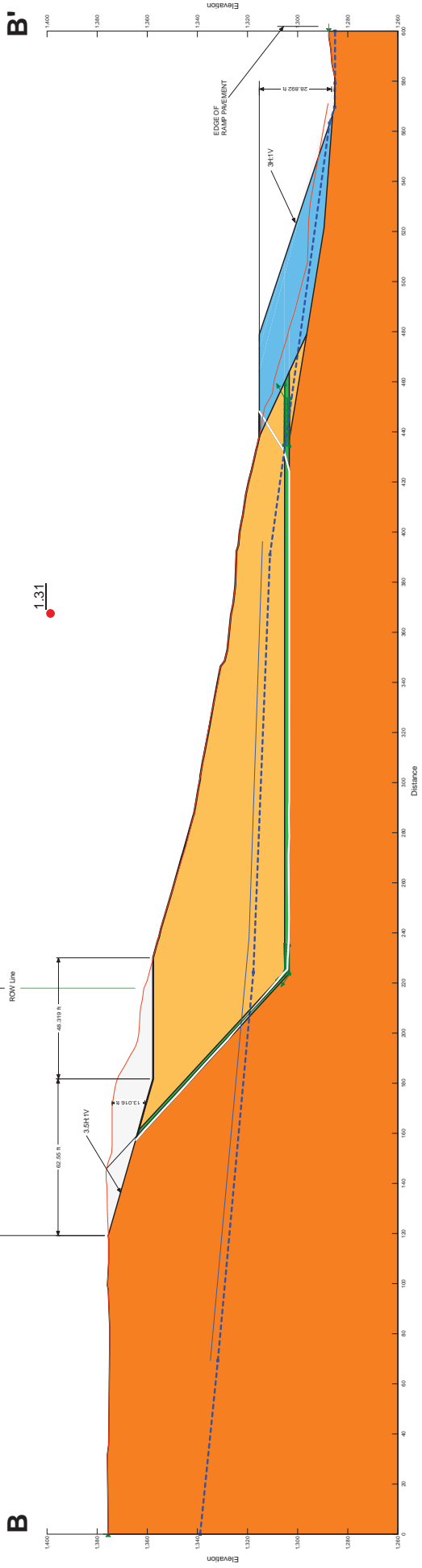
March 2023 107877-001

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FIG. 8
 Sheet 1 of 2



LEGEND

- Cross Section Lines
- Approximate Limits of Grading
- Approximate Finger Drain Locations
- Approximate Right of Way Boundary
- SW-05 VWP Designation
- SW-04 Inclinometer Designation
- SW-01 Inclinometer and VWP Designation
- SW-03 Inclinometer, VWP, and TDR Designation
- INC-1 NDDOT (2019) Inclinometer Designation



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

Valley City SW Backslope Slide Repair
 2-094 (165)290, PCN 23338
 Valley City, North Dakota

SCHEMATIC OF GRADING AND DRAINAGE ALTERNATIVE

March 2023 107877-001

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FIG. 8
 Sheet 2 of 2



State of North Dakota, ENR, HERE, Garmin, iSatGlobe, GeoTechnologies, INC, HERE/NASA, USGS, EPA, NPS, USDA

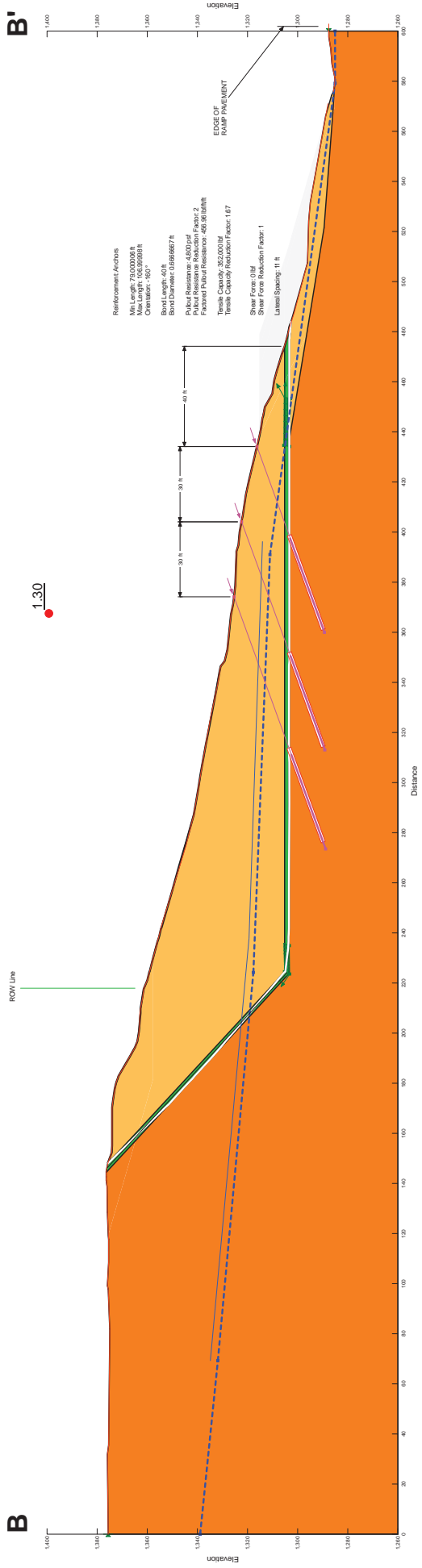
Valley City SW Backslope Slide Repair
 2-094(185)290, PCN 23338
 Valley City, North Dakota

SCHEMATIC OF GROUND ANCHORS ALTERNATIVE

March 2023 107877-001
SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants
FIG. 9
 Sheet 1 of 2

LEGEND

- Cross Section Lines
- Approximate Ground Anchor Locations
- Approximate Finger Drain Locations
- Approximate Maximum Projected Ground Anchor Extent
- Approximate Right of Way Boundary
- SW-05 VWP Designation
- SW-04 VWP Designation
- SW-01 Inclinator and VWP Designation
- SW-03 Inclinator, VWP, and TDR Designation
- INC-1 NDDOT (2019) Inclinator Designation



Reinforcement Anchors
 #4 Length: 30.0000 ft
 #4 Length: 40.0000 ft
 Orientation: 90°
 Bond Length: 40 ft
 Bond Diameter: 0.6569667 ft
 Pullout Resistance: 4,800 psf
 Axial Capacity: 480,000 lbs
 Factored Pullout Resistance: 480,000 lbs
 Tensile Capacity: 480,000 lbs
 Tensile Capacity Reduction Factor: 1.67
 Shear Force: 0 lbf
 Shear Force Reduction Factor: 1
 Lateral Spacing: 11 ft

Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

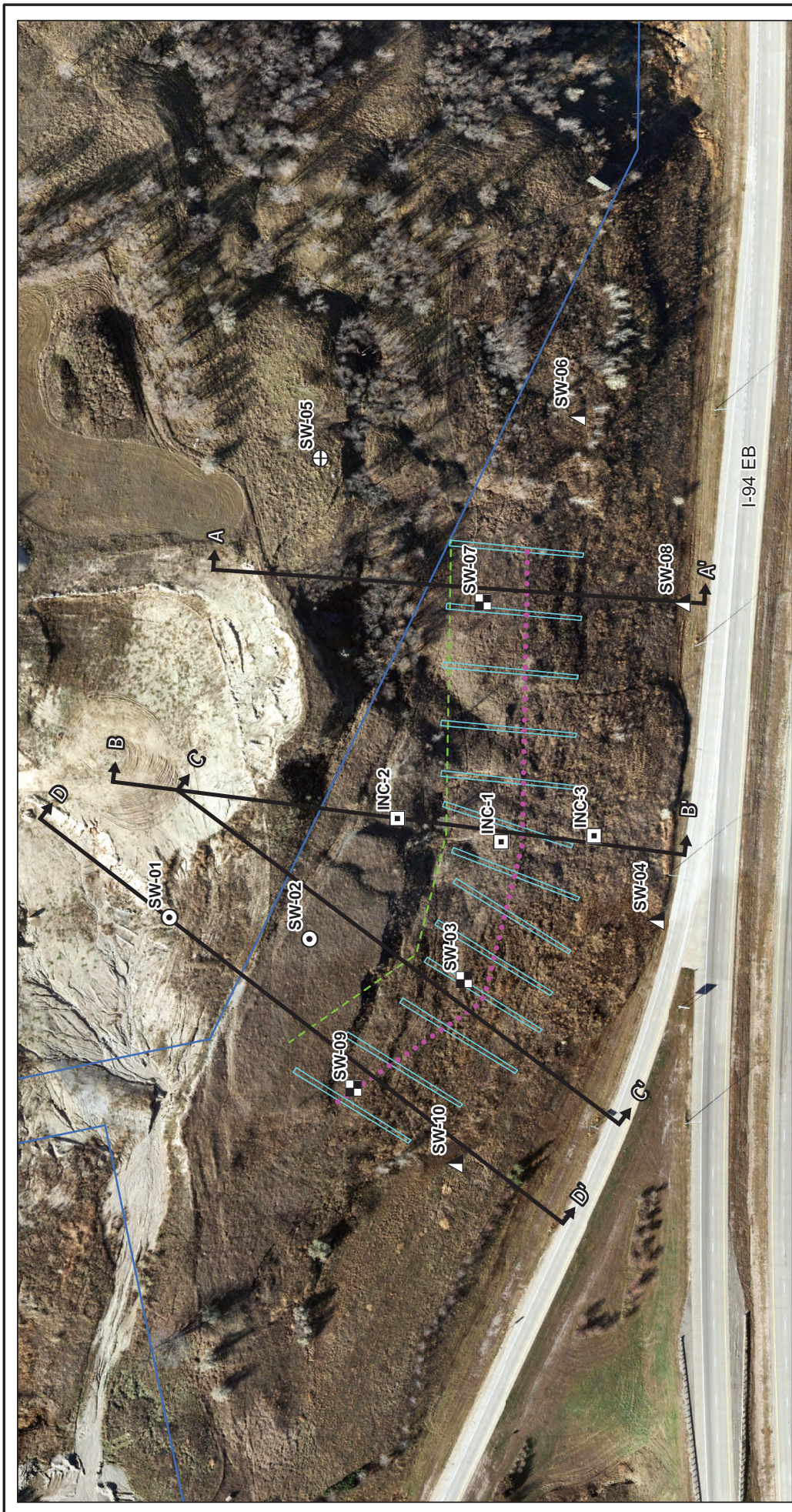
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SCHEMATIC OF GROUND ANCHORS ALTERNATIVE

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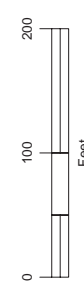
FIG. 9
 Sheet 2 of 2



Soils of North Dakota, East, West, and Middle, SAFEGROUP, Geotechnologies, INC., MET/NASA, USGS, EPA, NPS, USDA

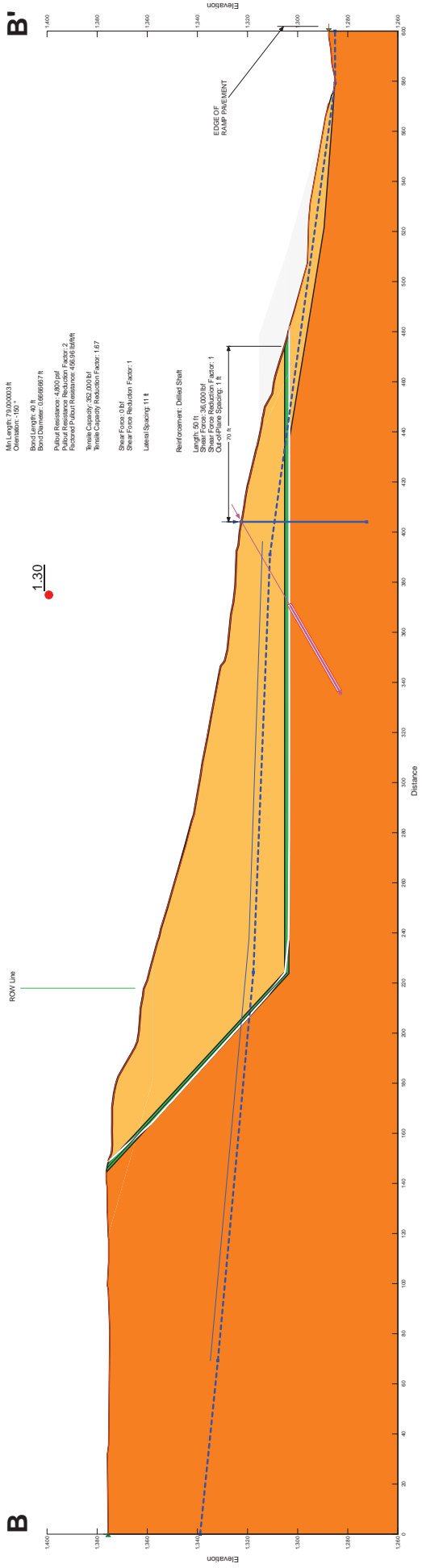
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SCHEMATIC OF ANCHORED DRILLED SHAFTS ALTERNATIVE	
March 2023	107877-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 10 Sheet 1 of 2



LEGEND

- B-B' ← Cross Section Lines
- Approximate Drilled Shaft and Ground Anchor Points
- Approximate Finger Drain Locations
- Approximate Maximum Projected Ground Anchor Extent
- Approximate Right of Way Boundary
- SW-05 VWP Designation
- SW-04 Inclinator Designation
- SW-01 Inclinator and VWP Designation
- SW-03 Inclinator, VWP, and TDR Designation
- INC-1 NDDOT (2019) Inclinator Designation



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

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**SCHEMATIC OF ANCHORED
 DRILLED SHAFTS ALTERNATIVE**
 March 2023 107877-001

SHANNON & WILSON, INC.
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FIG. 10
 Sheet 2 of 2

Appendix A

Subsurface Explorations

CONTENTS

A.1 Introduction A-1

A.2 Explorations A-1

 A.2.1 Classification System A-1

 A.2.2 Standard Penetration Test (SPT) and Sampling..... A-2

 A.2.3 Modified California (MC) Test and Sampling A-2

 A.2.4 Pocket Penetrometer A-3

Figures

Figure A-1: Soil Description and Log Key

Figure A-2: Rock Classification and Log Key

Figures A-3 to A-12: Log of Borings SW-01 to SW-10

A.1 INTRODUCTION

Shannon & Wilson's field exploration program was conducted on June 13, 2022 through June 16, 2022 and consisted of drilling and sampling 10 borings at the locations shown in Figure 2. The methods used to conduct the field exploration program are described below. Laboratory testing procedures and results are presented in Appendix B.

A.2 EXPLORATIONS

The borings were coordinated (including subcontractor coordination and utility locates) and observed by Shannon & Wilson. The NDDOT coordinated access through the adjacent quarry property and coordinated traffic control for two borings near the shoulder of the I-94 eastbound exit ramp. Individual boring logs are presented in Figures A-3 to A-12. The exploration logs represent our interpretation of the contents of the field logs and results of laboratory testing. The locations and elevation of the explorations were surveyed by KLJ after drilling was completed. Coordinates and top-of-ground elevations are included on the boring logs.

The borings were drilled by Interstate Drilling Services, LLP of Grand Forks, North Dakota (under subcontract to Shannon & Wilson) using a Diedrich D-50 track-mounted rig. The borings were advanced to depths ranging from approximately 26.0 to 70.0 feet and were advanced with 8-inch outside diameter (4.25-inch inside diameter) hollow-stem auger methods. Where groundwater was encountered, our field representative measured the approximate depth to groundwater using an electronic water level indicator. On completion of drilling, bentonite grout was used to backfill the borings. Selected borings were completed with geotechnical instrumentation as described in Appendix C. Excess drill cuttings were spread around the drill location.

After the completion of boring SW-07, the borehole was cased to a depth of 15 feet with 12-inch outside diameter hollow-stem auger (6.25-inch inside diameter) and reamed to the bottom of the hole using a 5.875-inch tricone bit. A drilling mud mixture of bentonite and water was used to flush out cuttings and keep the hole open for instrumentation installation (see Appendix C).

A.2.1 Classification System

During drilling, our representative collected samples and prepared field logs of the explorations. Soil classification for this project was based on ASTM D2487, Standard

Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), and ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The Unified Soil Classification System is summarized in Figure A-1. The Shannon & Wilson representative classified rock samples in general accordance with the International Society of Rock Mechanics (ISRM) classification method. According to this system, rocks are classified based on the stratigraphic structure, rock strength, degree of weathering, and other properties. The rock classification system is summarized in Figure A-2.

Consistent with other locations in North Dakota, the bedrock encountered in the borings was found to be hard when considered as a lithified soil material. However, when compared with other types of bedrock using the ISRM classification of rock strength, the material resembles a very low strength rock. Therefore, for completeness, the boring logs included in Appendix A contain dual descriptions of the bedrock using the Unified Soil Classification System and the ISRM classification system.

A.2.2 Standard Penetration Test (SPT) and Sampling

Disturbed samples were obtained in the borings in general accordance with the Standard Penetration Test (SPT) (ASTM Designation: D1586). The SPT consists of driving a 2-inch-outside-diameter (O.D.), 1.375-inch I.D. split-spoon sampler 18 to 24 inches. An automatic, free-falling 140-pound hammer was used to advance the split spoon sampler. During sampling, the Shannon & Wilson field representative recorded the number of blows for each 6-inch increment of penetration and summed the blow counts for the second and third 6-inch increments. This sum is recorded as the penetration resistance number, or N-value. If high penetration resistance prevented driving the total length of the sampler, the Shannon & Wilson field representative recorded the partial penetration depth and blow count. The N-values provide a means for evaluating the relative density or compactness of cohesionless (granular) soils and consistency or stiffness of cohesive (fine-grained) soils (see Figure A-1). The raw N-values are shown on the individual boring logs. Representative portions of the split-spoon sample obtained in conjunction with the SPT were placed in a screw-top plastic jar and transported to our laboratory.

A.2.3 Modified California (MC) Test and Sampling

Samples were also obtained using a Modified California (MC) barrel sampler. The MC test procedure is similar to the SPT, except the sample barrel is larger (2½-inch O.D.) and lined with 2-inch-diameter brass tubing. The MC sampler was also driven 18 to 24 inches. During sampling, the Shannon & Wilson field representative recorded the number of blows for each 6-inch increment of penetration and summed the blow counts for the second and

third 6-inch increments. As a result of the larger diameter, the MC sampler yields slightly higher raw blow count numbers when compared to SPT N-values for similar soils. Because the difference in blow counts does not significantly impact our evaluation, we used the field MC blow counts over the 18-inch increment to define the relative density and consistency/stiffness of the subsurface materials following SPT terminology. Representative samples were sealed in the brass liner tubes with plastic caps and transported to our laboratory for further testing.

A.2.4 Pocket Penetrometer

Select cohesive soil samples were also tested in the field using a pocket penetrometer. The penetrometer estimates the unconfined compressive strength of clay soil samples by penetrating the clay with a one-quarter-inch-diameter penetrometer and measuring the resistance (in units of tons per square foot [tsf]) with a calibrated spring. Measurements can be taken to the nearest 0.25 tsf increment. The field measurements from the pocket penetrometer are included on the boring logs.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT ²	FINE-GRAINED SOILS (50% or more fines) ¹	COARSE-GRAINED SOILS (less than 50% fines) ¹
Major	Silt, Lean Clay, Elastic Silt, ³ or Fat Clay	Sand or Gravel ⁴
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: Sandy or Gravelly ⁴	More than 12% fine-grained: Silty or Clayey ³
Minor Follows major constituent	15% to 30% coarse-grained: with Sand or with Gravel ⁴ 30% or more total coarse-grained and lesser coarse-grained constituent is 15% or more: with Sand or with Gravel ⁵	5% to 12% fine-grained: with Silt or with Clay ³ 15% or more of a second coarse-grained constituent: with Sand or with Gravel ⁵

¹All percentages are by weight of total specimen passing a 3-inch sieve.
²The order of terms is: *Modifying Major with Minor*.
³Determined based on behavior.
⁴Determined based on which constituent comprises a larger percentage.
⁵Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
	NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.
	NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND Fine Medium Coarse	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.) #40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.) #10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL Fine Coarse	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.) 3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

	Bentonite Cement Grout		Surface Cement Seal
	Bentonite Grout		Asphalt or Cap
	Bentonite Chips		Slough
	Silica Sand		Inclinometer or Non-perforated Casing
	Perforated or Screened Casing		Vibrating Wire Piezometer

PERCENTAGES TERMS^{1,2}

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

¹Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

²Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

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SOIL DESCRIPTION AND LOG KEY
















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FIG. A-1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
 (Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS (more than 50% retained on No. 200 sieve)	Gravels (more than 50% of coarse fraction retained on No. 4 sieve)	Gravel (less than 5% fines)	GW 	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP 	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel (more than 12% fines)	GM 	Silty Gravel; Silty Gravel with Sand
			GC 	Clayey Gravel; Clayey Gravel with Sand
	Sands (50% or more of coarse fraction passes the No. 4 sieve)	Sand (less than 5% fines)	SW 	Well-Graded Sand; Well-Graded Sand with Gravel
			SP 	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand (more than 12% fines)	SM 	Silty Sand; Silty Sand with Gravel
			SC 	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS (50% or more passes the No. 200 sieve)	Silt and Clays (liquid limit less than 50)	Inorganic	ML 	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL 	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL 	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silt and Clays (liquid limit 50 or more)	Inorganic	MH 	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH 	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH 	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor	PT 	Peat or other highly organic soils (see ASTM D4427)	

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

- Dual symbols (symbols separated by a hyphen, i.e., SP-SM, Sand with Silt) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).
- Borderline symbols (symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand) indicate that the soil properties are close to the defining boundary between two groups.

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**SOIL DESCRIPTION
 AND LOG KEY**

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FIG. A-1
 Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS¹

Weak	Crumbles or breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

PLASTICITY²

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASTICITY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Biorturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS¹

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q _u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS¹

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A-1
 Sheet 3 of 3

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WEATHERING

TERM	DESCRIPTION
Fresh	No visible sign of rock material weathering
Slightly Weathered	Slight discoloration on surface
Moderately Weathered	Discoloring evident; Less than half of the rock material is decomposed
Highly Weathered	Entire rock mass discolored; More than half of the rock material is decomposed
Completely Weathered	Rock reduced to a soil with relict rock texture
Residual Soil	All rock material is converted to soil

STRENGTH

GRADE	DESCRIPTION	APPROX. UCS (psi)
R0	Extremely Weak Rock	36 to 145
R1	Very Weak Rock	145 to 700
R2	Weak Rock	700 to 3,600
R3	Medium Strong Rock	3,600 to 7,200
R4	Strong Rock	7,200 to 14,500
R5	Very Strong Rock	14,500 to 36,250
R6	Extremely Strong Rock	>36,250

JOINT ROUGHNESS COEFFICIENT (JRC)

COEFFICIENT	DESCRIPTION
14 to 20	VERY ROUGH: Near vertical edges evident
10 to 14	ROUGH: Smooth ridges, surface abrasion
6 to 10	SLIGHTLY ROUGH: Asperities on surface can be felt
2 to 6	SMOOTH: Appears and feels smooth
0 to 2	SLICKENSIDED: Visible polishing, striated surface

DISCONTINUITY DATA

SPACING	
DESCRIPTION	SPACING
Extremely Close	< 1 in
Very Close	1 to 2.5 in
Close	2.5 to 8 in
Moderate	8 to 24 in
Wide	24 in to 6 ft
Very Wide	6 to 20 ft
Extremely Wide	> 20 ft

DISCONTINUITY TERMS

FRACTURE - Collective term for any natural break excluding shears, shear zones, and faults

JOINT (JT) - Planar break with little or no displacement

FOLIATION JOINT (FJ) or BEDDING JOINT (BJ) - Joint along foliation or bedding

INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF) - Joint or fracture not evident until wetted and dried; breaks along existing surface

RANDOM FRACTURE (RF) - Natural, very irregular fracture that does not belong to a set

BEDDING PLANE SEPARATION or PARTING - A separation along bedding after extraction from stress relief or slaking

FRACTURE ZONE (FZ) - Planar zone of broken rock without gouge

MECHANICAL BREAK (MB) - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)

SHEAR (SH) - Surface of differential movement evident by presence of slickensides, striations, or polishing

SHEAR ZONE (SZ) - Zone of gouge and rock fragments bounded by planar shear surfaces

FAULT (FT) - Shear zone of significant extent; differentiation from shear zone may be site-specific

APERTURE WIDTH	
TERM	SPACING
Very Tight	<0.1mm
Tight	0.1 to 0.25mm
Partly Open	0.25 to 0.5mm
Open	0.5 to 2.5mm
Moderately Wide	2.5 to 10mm
Wide	10mm to 1cm
Very Wide	1 to 10cm
Extremely Wide	10 to 100cm
Cavernous	>1m

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ROCK CLASSIFICATION AND LOG KEY

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FIG. A-2
Sheet 1 of 2

ROCK CLASS KEY - P2 107857-001 NPS GLACIER NP GPJ SHAN WIL GDT 8/24/22

ROCK CLASSIFICATION SYMBOLS		
BEDROCK TYPE	GRAPHIC SYMBOL	ROCK NAME
Clastic Sedimentary Rocks		Breccia
		Conglomerate
		Sandstone
		Siltstone
		Claystone
		Shale
		Coal
Carbonate Sedimentary Rocks		Limestone
		Dolomite
		Coral
Evaporite Rocks		Gypsum
		Halite
		Calcite
Extrusive Igneous Rocks		Tuff
		Rhyolite
		Dacite
		Andesite
		Basalt
Intrusive Igneous Rocks		Granite
		Grano-diorite
		Diorite
		Gabbro
Metamorphic Rocks		Marble
		Quartzite
		Slate
		Phyllite
		Schist
		Gneiss

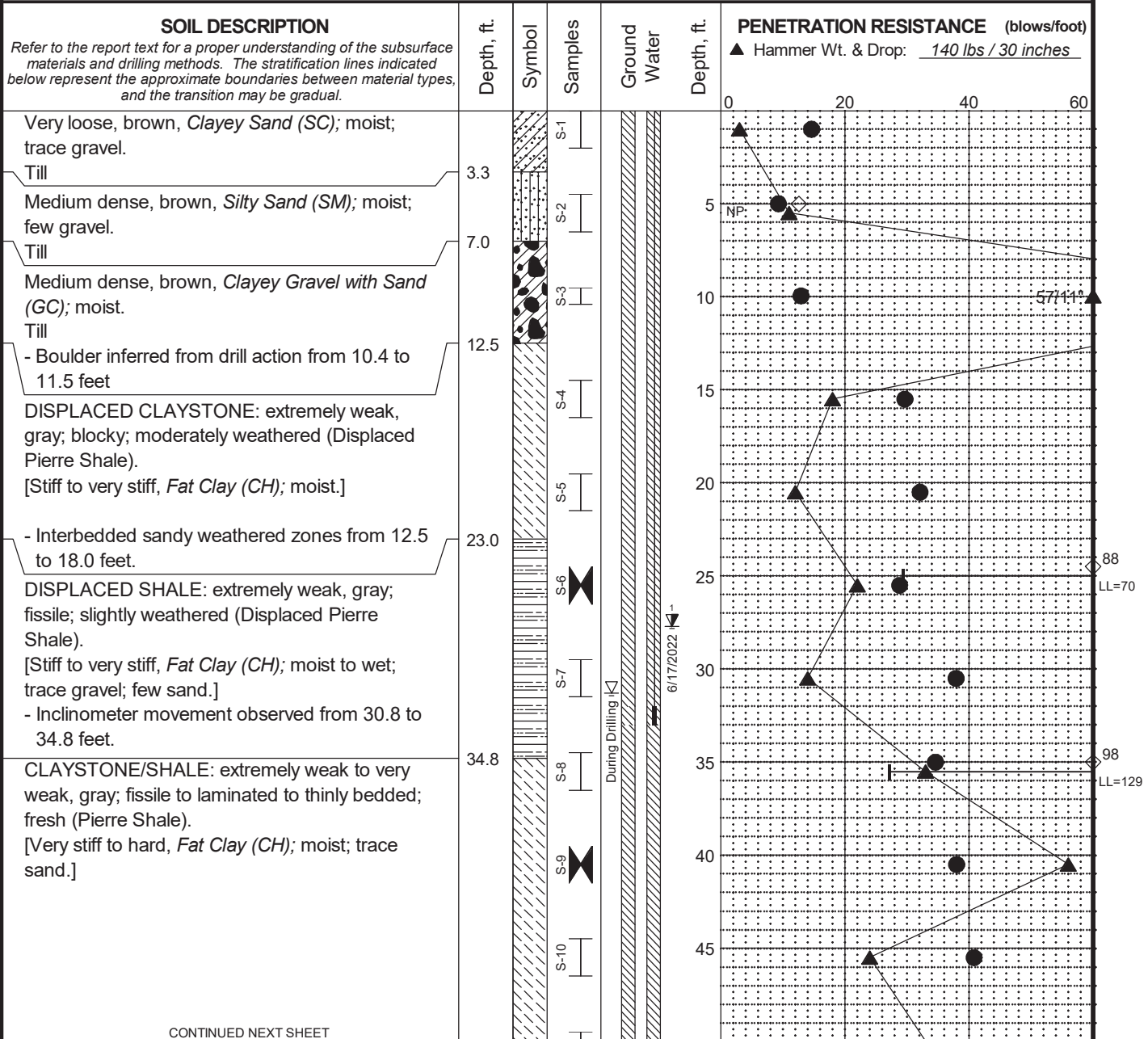
Valley City SW Backslope Slide Repair
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Valley City, North Dakota

**ROCK CLASSIFICATION
AND LOG KEY**

March 2023 107877-001

SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. A-2 Sheet 2 of 2
---	---------------------------------

Total Depth: 70.4 ft. Northing: 463,388 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1362.5 ft. Easting: 2,583,305 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: _____ Other Comments: Inclinometer and piezometer installed in bore hole.



CONTINUED NEXT SHEET

- LEGEND**
- * Sample Not Recovered
 - Standard Penetration Test
 - Modified California Sampler
 - Inclinometer with Bentonite-Cement Grout
 - Inclinometer and VWP with Bentonite-Cement Grout
 - VWP with Bentonite-Cement Grout
 - Ground Water Level ATD
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

NOTES

- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
- The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
- The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
- USCS designation is based on visual-manual classification and selected lab testing.
- KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-01

March 2023

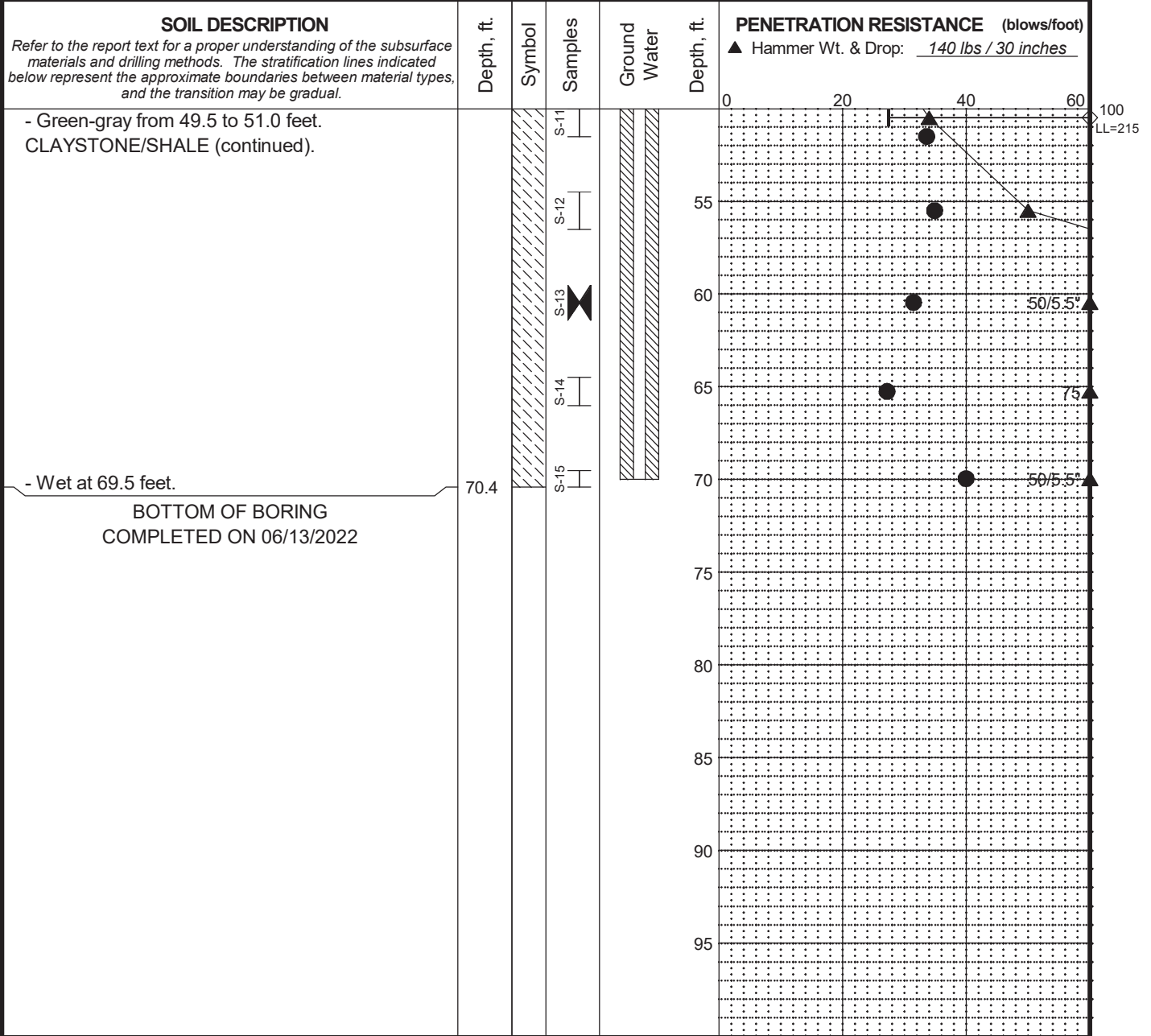
107877-001

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FIG. A-3
 Sheet 1 of 2

MASTER LOG E_POCKETPEN_INCLINOMETER_107877-001.GPJ 11/18/22

Total Depth: 70.4 ft. Northing: 463,388 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1362.5 ft. Easting: 2,583,305 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



LEGEND

* Sample Not Recovered	Inclinometer with Bentonite-Cement Grout	◇ % Fines (<0.075mm)
┆ Standard Penetration Test	Inclinometer and VWP with Bentonite-Cement Grout	● % Water Content
⊠ Modified California Sampler	VWP with Bentonite-Cement Grout	Plastic Limit —●— Liquid Limit
	▽ Ground Water Level ATD	Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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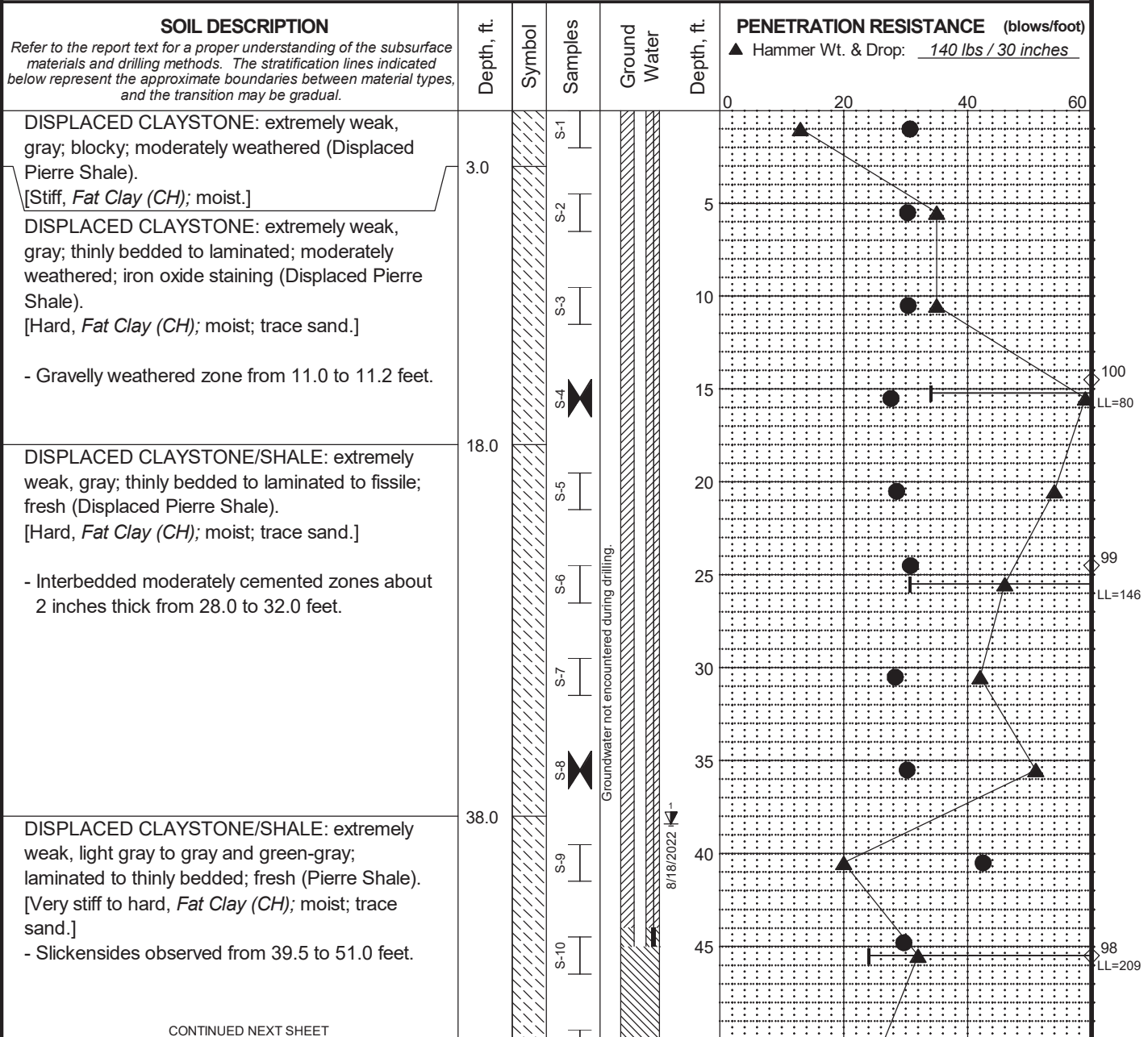
LOG OF BORING SW-01

March 2023 107877-001

SHANNON & WILSON, INC. **FIG. A-3**
 Geotechnical and Environmental Consultants Sheet 2 of 2

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 60.5 ft. Northing: 463,529 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1357.8 ft. Easting: 2,583,352 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



CONTINUED NEXT SHEET

- LEGEND**
- * Sample Not Recovered
 - ┆ Standard Penetration Test
 - ⊠ Modified California Sampler
 - ▨ Inclinometer with Bentonite-Cement Grout
 - ▩ Inclinometer and VWP with Bentonite-Cement Grout
 - ▧ VWP with Bentonite-Cement Grout
 - ◆ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

NOTES

- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
- The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
- The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
- USCS designation is based on visual-manual classification and selected lab testing.
- KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-02

March 2023

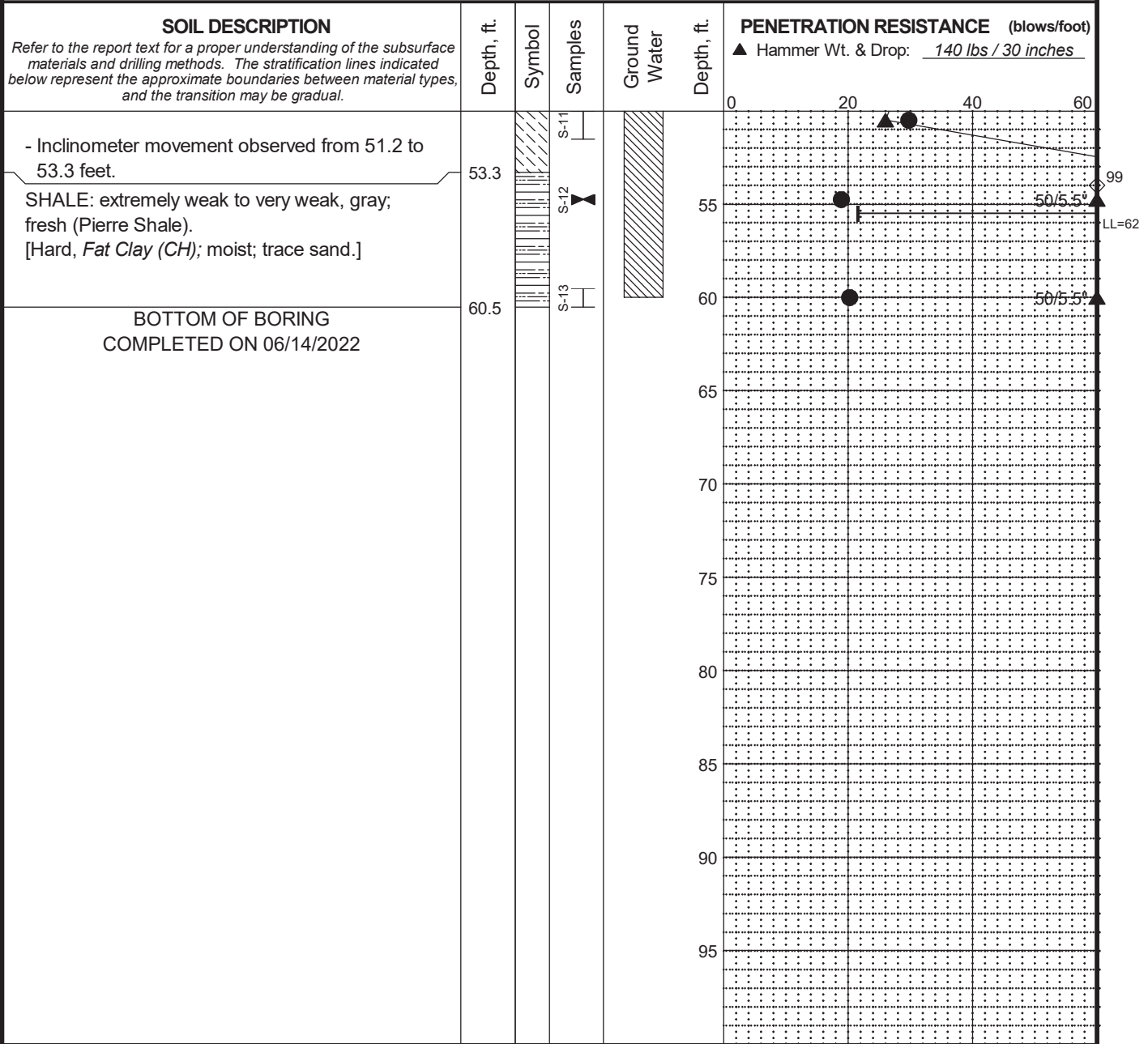
107877-001

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FIG. A-4
 Sheet 1 of 2

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 11/18/22

Total Depth: 60.5 ft. Northing: 463,529 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1357.8 ft. Easting: 2,583,352 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



- LEGEND**
- * Sample Not Recovered
 - ┆ Standard Penetration Test
 - ⊠ Modified California Sampler
 - Inclinometer with Bentonite-Cement Grout
 - Inclinometer and VWP with Bentonite-Cement Grout
 - VWP with Bentonite-Cement Grout
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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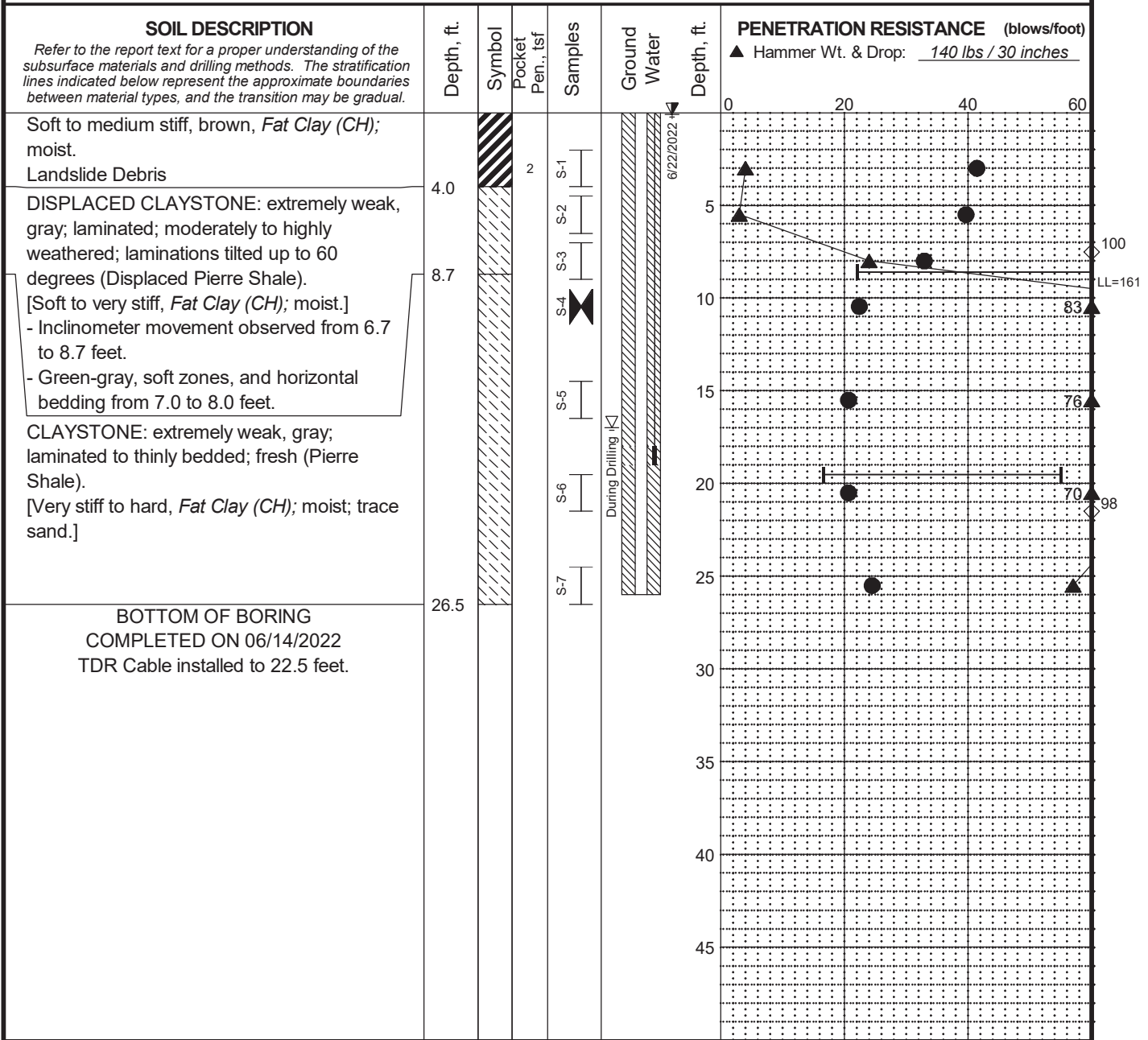
LOG OF BORING SW-02

March 2023 107877-001

SHANNON & WILSON, INC. **FIG. A-4**
 Geotechnical and Environmental Consultants Sheet 2 of 2

MASTER LOG_E_POCKETPEN_INCLINOMETER_107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 26.5 ft. Northing: 463,681 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1314.1 ft. Easting: 2,583,422 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



LEGEND

* Sample Not Recovered	[Symbol]	Inclinometer with Bentonite-Cement Grout	◇ % Fines (<0.075mm)
⊥ Standard Penetration Test	[Symbol]	Inclinometer and VWP with Bentonite-Cement Grout	● % Water Content
⊠ Modified California Sampler	[Symbol]	VWP with Bentonite-Cement Grout	— Plastic Limit — Liquid Limit
	▽	Ground Water Level ATD	— Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-03

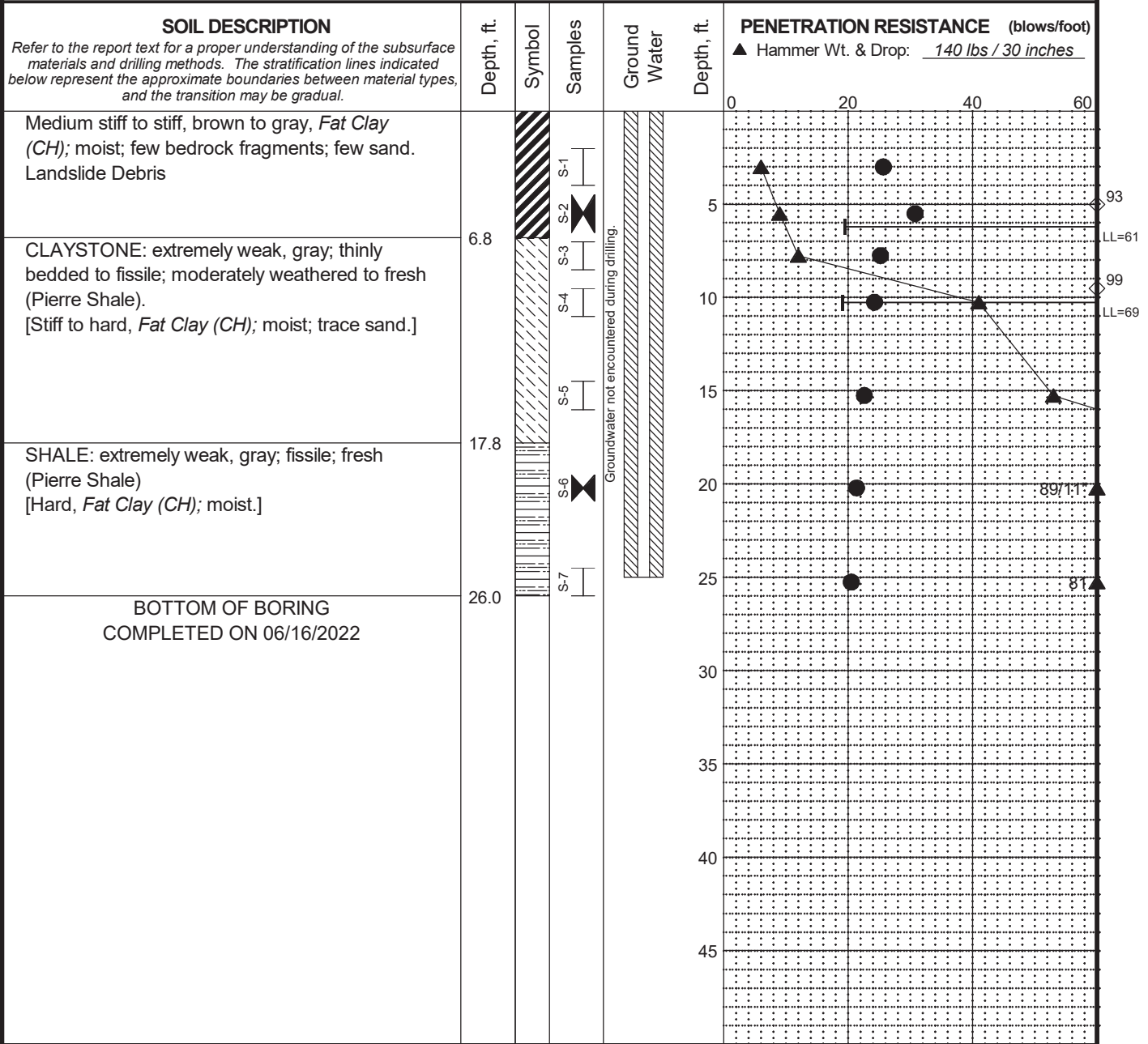
March 2023 107877-001

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FIG. A-5

MASTER LOG E_POCKETPEN_INCLINOMETER_107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 26 ft. Northing: 463,886 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1285.1 ft. Easting: 2,583,396 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



- LEGEND**
- * Sample Not Recovered
 - ┆ Standard Penetration Test
 - ⊠ Modified California Sampler
 - ▨ Inclinometer with Bentonite-Cement Grout
 - ▩ Inclinometer and VWP with Bentonite-Cement Grout
 - ▧ VWP with Bentonite-Cement Grout
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - Groundwater level, if indicated above, is for the date specified and may vary.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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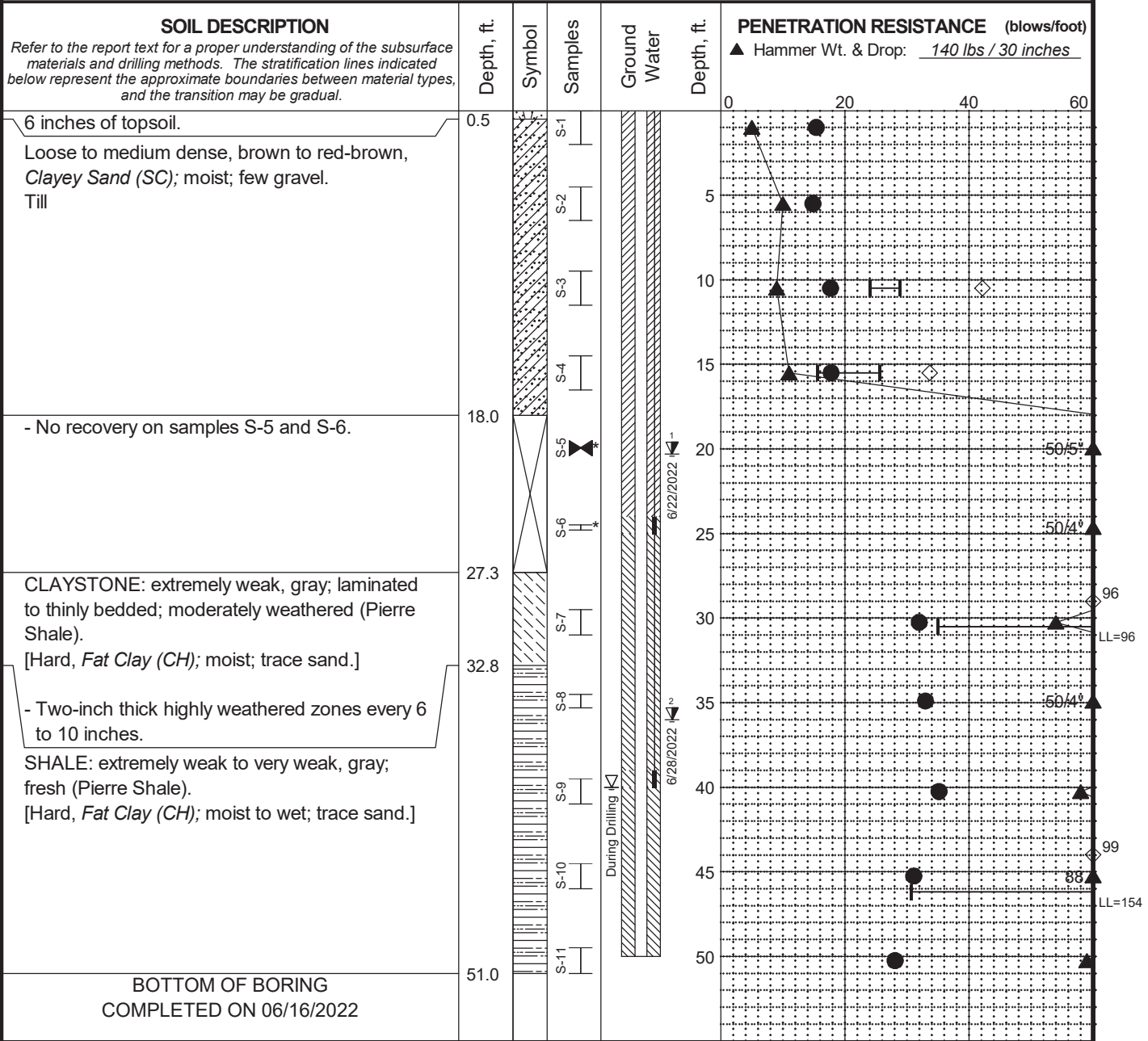
LOG OF BORING SW-04

March 2023 107877-001

SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants **FIG. A-6**

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 11/18/22

Total Depth: 51 ft. Northing: 463,626 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1378.5 ft. Easting: 2,582,858 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Two piezometers installed in bore hole.



LEGEND

* Sample Not Recovered	Piezometer Screen and Sand Filter	% Fines (<0.075mm)
Standard Penetration Test	Bentonite-Cement Grout	% Water Content
Modified California Sampler	Bentonite Chips/Pellets	Plastic Limit Liquid Limit
	Surface Concrete Seal	Natural Water Content
	Ground Water Level ATD	

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer readings are the highest groundwater elevations observed with the corresponding reading dates. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-05

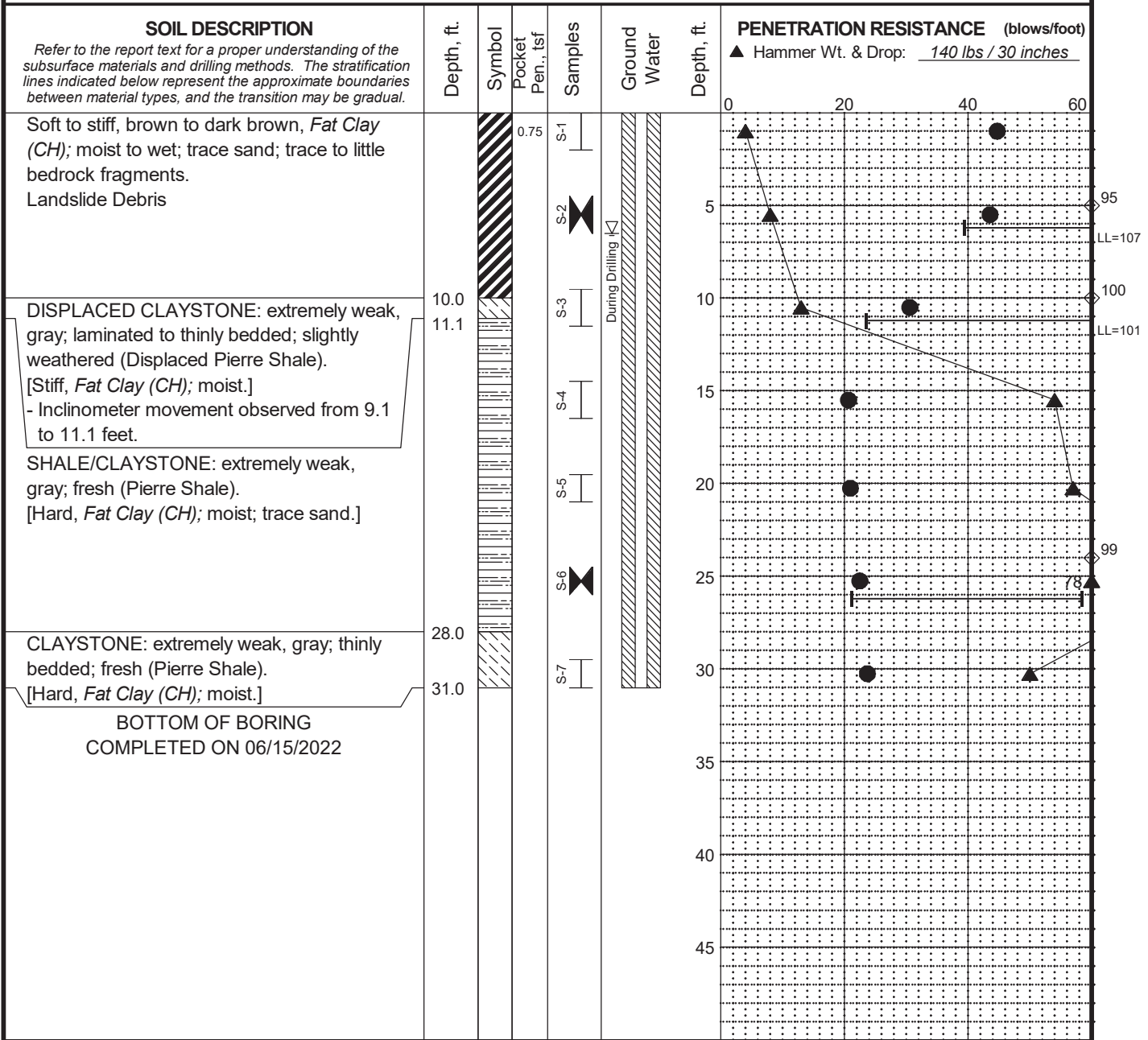
March 2023 107877-001

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FIG. A-7

MASTER LOG E_POCKETPEN 107877-001.GPJ DENVER 2021.GDT 11/18/22

Total Depth: 31 ft. Northing: 463,898 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1322.1 ft. Easting: 2,582,863 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



LEGEND

- * Sample Not Recovered
- ┆ Standard Penetration Test
- ⊠ Modified California Sampler
- ▨ Inclinometer with Bentonite-Cement Grout
- ▩ Inclinometer and VWP with Bentonite-Cement Grout
- ▧ VWP with Bentonite-Cement Grout
- ▽ Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- % Water Content
- Plastic Limit
- Liquid Limit
- Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - Groundwater level, if indicated above, is for the date specified and may vary.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-06

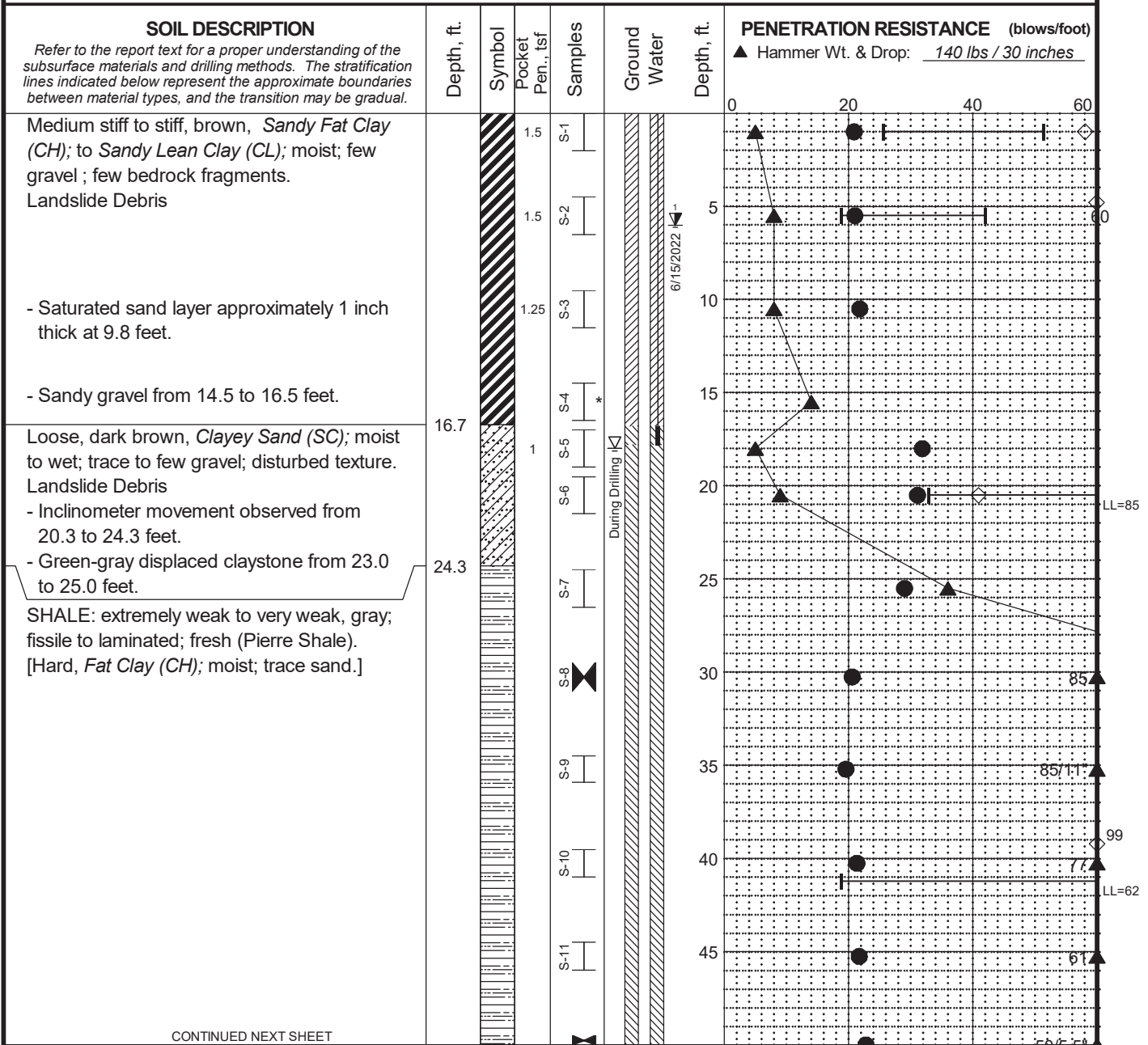
March 2023 107877-001

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FIG. A-8

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 56 ft. Northing: 463,769 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1334.1 ft. Easting: 2,583,037 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: _____ Other Comments: Inclinometer and piezometer installed in bore hole.



CONTINUED NEXT SHEET

- LEGEND**
- * Sample Not Recovered
 - Standard Penetration Test
 - Modified California Sampler
 - Inclinometer with Bentonite-Cement Grout
 - Inclinometer and VWP with Bentonite-Cement Grout
 - VWP with Bentonite-Cement Grout
 - Ground Water Level ATD
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

NOTES

- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
- The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
- The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
- USCS designation is based on visual-manual classification and selected lab testing.
- KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-07

March 2023

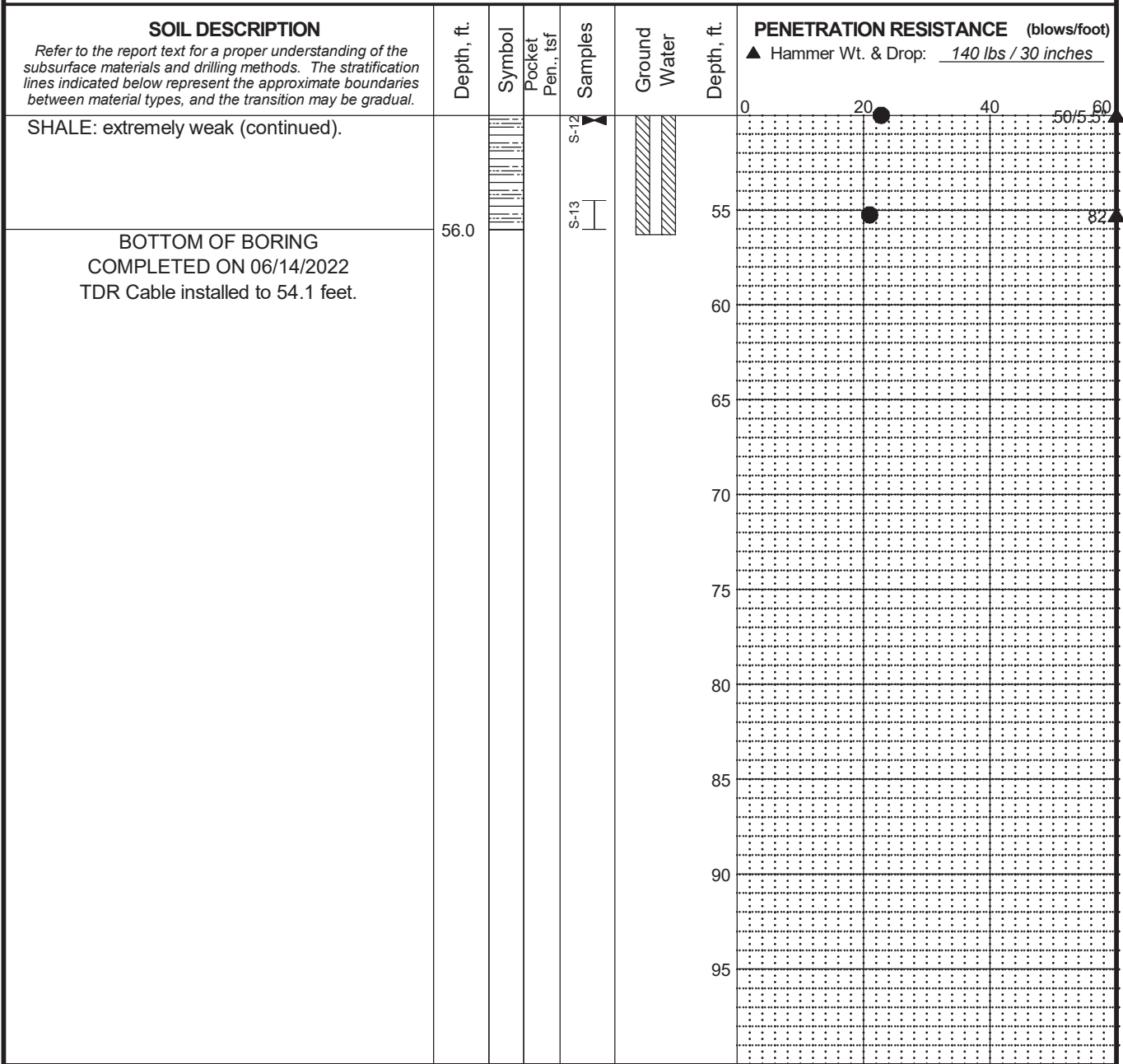
107877-001

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FIG. A-9
 Sheet 1 of 2

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 56 ft. Northing: 463,769 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1334.1 ft. Easting: 2,583,037 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



LEGEND

* Sample Not Recovered		Inclinometer with Bentonite-Cement Grout	◇ % Fines (<0.075mm)
┆ Standard Penetration Test		Inclinometer and VWP with Bentonite-Cement Grout	● % Water Content
⊠ Modified California Sampler		VWP with Bentonite-Cement Grout	Plastic Limit —●— Liquid Limit
	∇	Ground Water Level ATD	Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-07

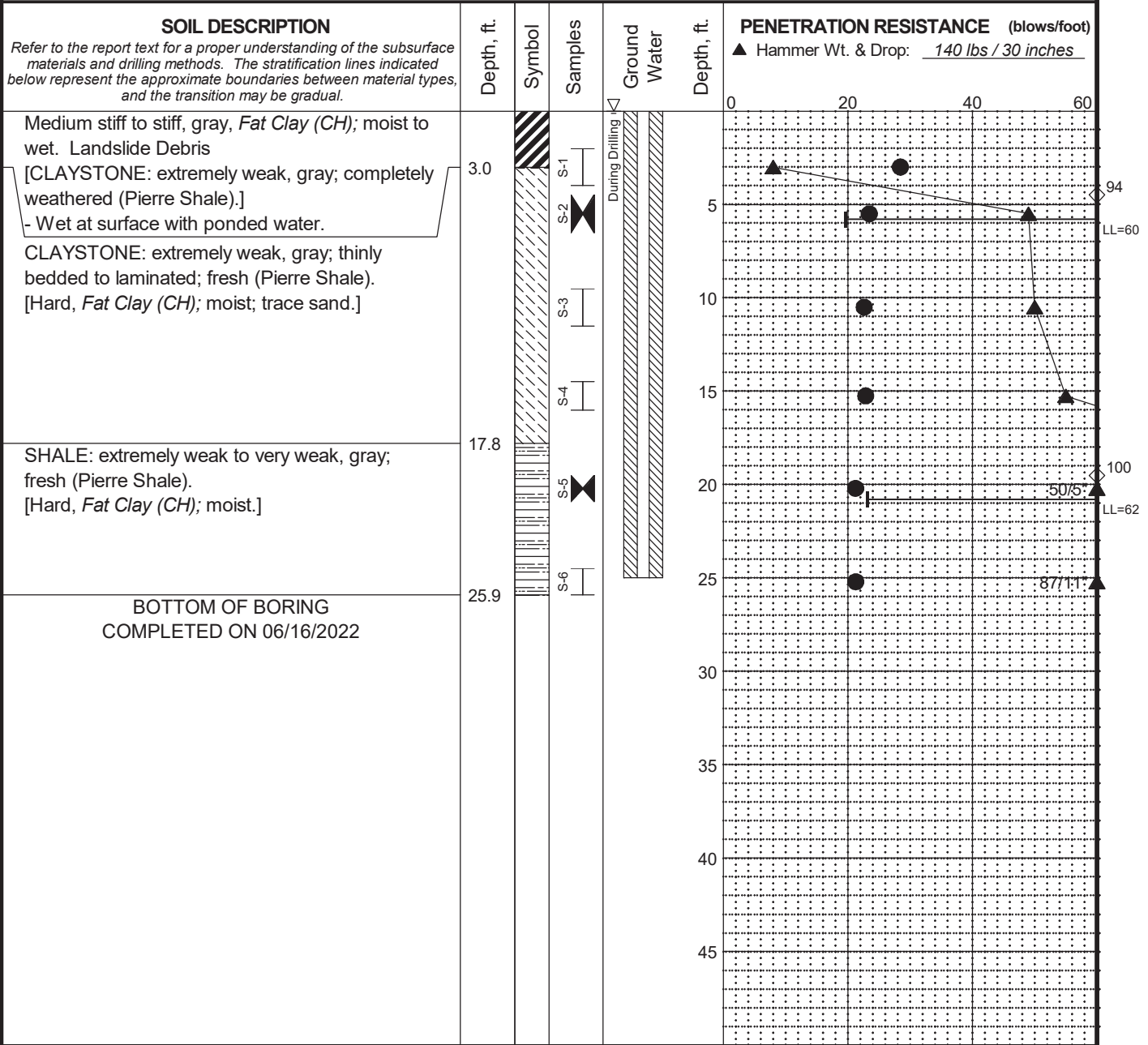
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 Geotechnical and Environmental Consultants

FIG. A-9
 Sheet 2 of 2

MASTER LOG_E_POCKETPEN_INCLINOMETER_107877-001.GPJ 107877-001.GPJ 11/18/22

Total Depth: 25.9 ft. Northing: 463,972 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1284.5 ft. Easting: 2,583,073 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: _____ Other Comments: Inclinometer installed in bore hole.



- LEGEND**
- * Sample Not Recovered
 - ⊥ Standard Penetration Test
 - ⊠ Modified California Sampler
 - ▨ Inclinometer with Bentonite-Cement Grout
 - ▩ Inclinometer and VWP with Bentonite-Cement Grout
 - ▧ VWP with Bentonite-Cement Grout
 - ▽ Ground Water Level ATD
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit —●— Liquid Limit
 - Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - Groundwater level, if indicated above, is for the date specified and may vary.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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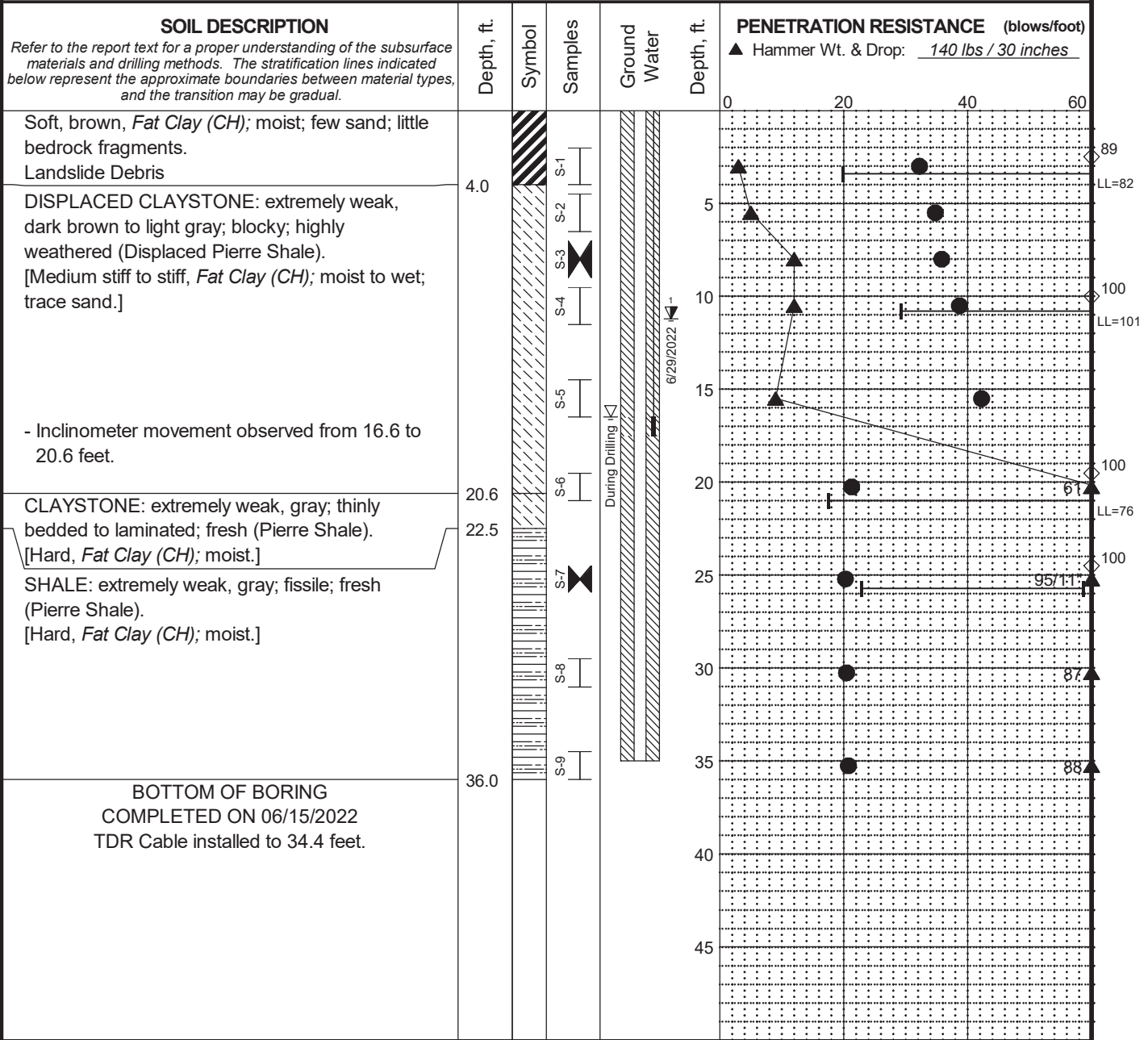
LOG OF BORING SW-08

March 2023 107877-001

SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants **FIG. A-10**

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 11/18/22

Total Depth: 36 ft. Northing: 463,547 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1324.1 ft. Easting: 2,583,514 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer and piezometer installed in bore hole.



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Modified California Sampler
- Inclinometer with Bentonite-Cement Grout
- Inclinometer and VWP with Bentonite-Cement Grout
- VWP with Bentonite-Cement Grout
- Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- % Water Content
- Plastic Limit
- Liquid Limit
- Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - The above vibrating wire piezometer reading is the highest groundwater elevation observed with the corresponding reading date. Refer to Appendix C for the complete record of vibrating wire piezometer readings.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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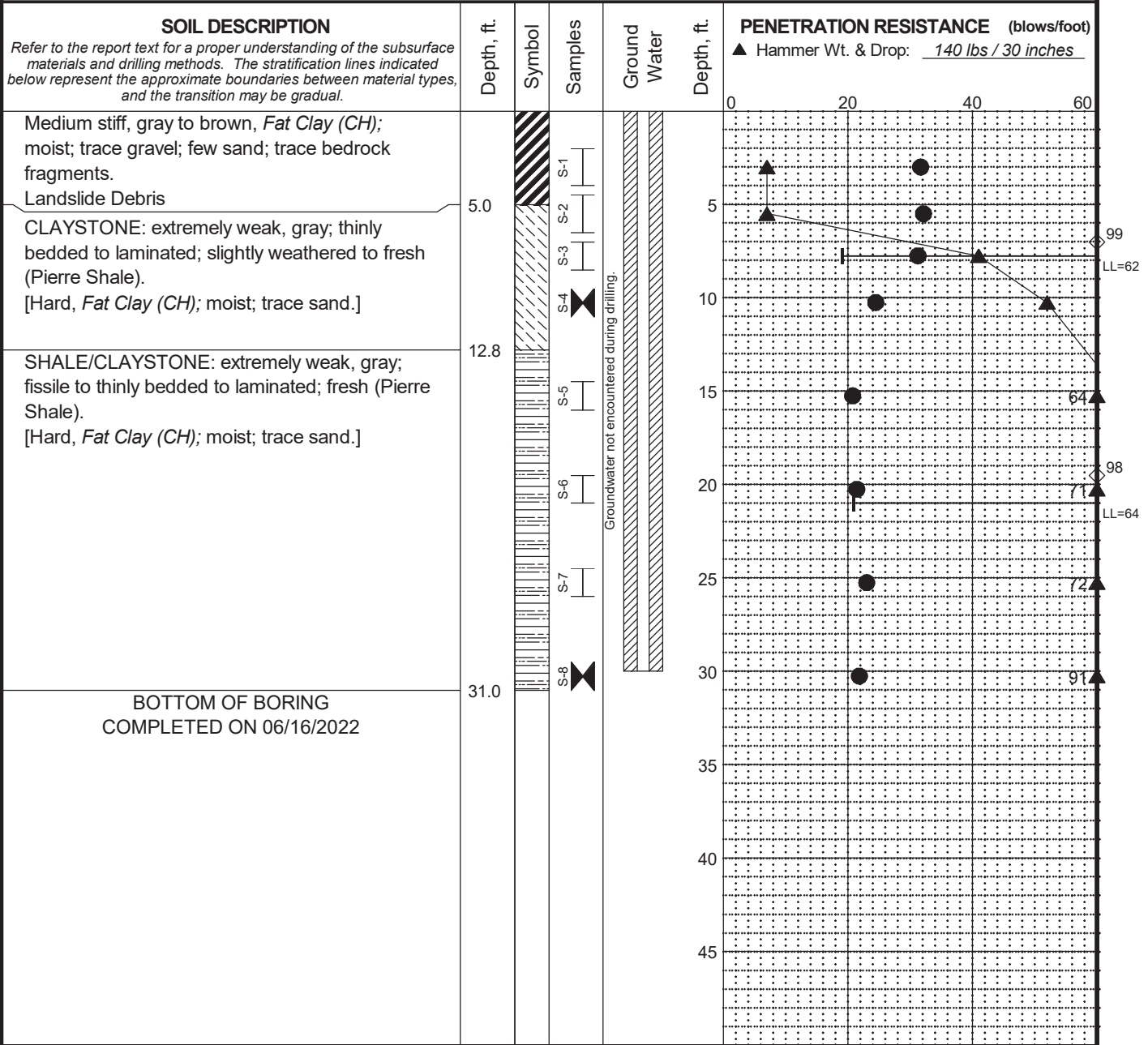
LOG OF BORING SW-09

March 2023 107877-001

SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants **FIG. A-11**

MASTER LOG_E_POCKETPEN_INCLINOMETER_107877-001.GPJ 11/18/22

Total Depth: 31 ft. Northing: 463,636 ft. Drilling Method: Hollow-Stem Auger Hole Diam.: 8 in.
 Top Elevation: 1295.2 ft. Easting: 2,583,610 ft. Drilling Company: Interstate Drilling Rod Type.: AWJ
 Vert. Datum: NAVD88 Station: _____ Drill Rig Equipment: Diedrich D-50 Track Hammer Type: Automatic
 Horiz. Datum: NAD83 (Conus) Offset: - Other Comments: Inclinometer installed in bore hole.



- LEGEND**
- * Sample Not Recovered
 - ┆ Standard Penetration Test
 - ⊠ Modified California Sampler
 - ▨ Inclinometer with Bentonite-Cement Grout
 - ▩ Inclinometer and VWP with Bentonite-Cement Grout
 - ▧ VWP with Bentonite-Cement Grout
 - ◇ % Fines (<0.075mm)
 - % Water Content
 - Plastic Limit
 - Liquid Limit
 - Natural Water Content

- NOTES**
- Refer to Figures A-1 and A-2 for explanation of symbols, codes, abbreviations and definitions.
 - The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
 - Groundwater level, if indicated above, is for the date specified and may vary.
 - USCS designation is based on visual-manual classification and selected lab testing.
 - KLJ Engineering surveyed the boring location for horizontal position and elevation and provided the coordinates to Shannon & Wilson on July 8, 2022.

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LOG OF BORING SW-10

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SHANNON & WILSON, INC.
 Geotechnical and Environmental Consultants **FIG. A-12**

MASTER LOG E. POCKETPEN INCLINOMETER 107877-001.GPJ 11/18/22

Appendix B

Laboratory Test Results

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Figure B-4:	Torsional Ring Shear, Drained Residual Shear Strength Test Results, Sample Composite 2
Figure B-5:	Torsional Ring Shear, Drained Residual Shear Strength Envelope
Figure B-6:	Torsional Ring Shear, Drained Residual Shear Strength Envelope (Detail)

B.1 INTRODUCTION

Laboratory tests were completed on soil and bedrock samples retrieved from the borings in general accordance with American Association of State Highway and Transportation Officials (AASHTO) test methods, ASTM International (ASTM) test methods, and Colorado Department of Transportation (CDOT) CP-L test methods. The laboratory testing program was performed to classify the materials into similar geologic groups and provide data that can be used for design of the project. The geotechnical laboratory testing was performed at Shannon & Wilson laboratories in Denver, Colorado and St. Louis, Missouri, and at Cooper Testing Labs in Palo Alto, California. A summary of the laboratory test results is presented in Table B-1. The following sections describe the laboratory testing procedures.

B.2 GEOTECHNICAL INDEX TESTS

B.2.1 Water Content

Water content was determined for selected samples in general accordance with AASHTO T265, Laboratory Determination of Moisture Content in Soils. To perform this test, a sample was weighed before and after oven-drying, and the water content was calculated. Water content determinations are shown graphically on the boring logs in Appendix A and are also summarized in Table B-1.

B.2.2 Unit Weight

The unit weights or in-place densities of selected modified California (MC) samples were determined in general accordance with ASTM D7263, Standard Test Methods for Laboratory Determination of Density and Unit Weight of Soil Specimens. To perform this test, the dimensions of the sample were measured, the sample was weight, and the moist unit weight was calculated. The results are summarized in Table B-1.

B.2.3 Grain Size Analysis

The grain size distribution of selected samples was determined in general accordance with AASHTO T311, Standard Method of Test for Grain-Size Analysis of Granular Soil Materials and AASHTO T88, Standard Method of Test for Particle Size Analysis of Soils for samples where a hydrometer analysis was completed. Results of these analyses are presented as grain size distribution curves on Figure B-1 and summarized in Table B-1.

Selected samples were tested for the percentage of material passing the No. 200 sieve in general accordance with ASTM D1140, Standard Test Method for Amount of Material in Soils Finer than the No. 200 (75- μ m) Sieve.

Where applicable, the percent fines (silt- and clay-sized particles passing the No. 200 sieve) are shown graphically on the boring logs in Appendix A and are also summarized in Table B-1.

B.2.4 Atterberg Limits

Soil plasticity was determined by performing Atterberg limits tests on selected fine-grained samples. The tests were completed in general accordance with AASHTO T89, Standard Method of Test for Determining the Liquid Limit of Soils and AASHTO T90, Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils. The Atterberg limits include liquid limit (LL), plastic limit (PL), and plasticity index (PI equals LL minus PL) and are generally used to assist in classification of soils, to indicate soil consistency (when compared to natural water content), and to provide correlation to soil properties. The results of the Atterberg limits tests are plotted on a plasticity chart in Figure B-2, shown graphically on the boring logs in Appendix A, and summarized in Table B-1.

B.3 GEOTECHNICAL ENGINEERING PROPERTY TESTS

B.3.1 Corrosion

Corrosion testing of select samples was performed for pH, resistivity, sulfate content, and chloride content. Testing for pH was completed in general accordance with AASHTO T289, Standard Test Method for Measuring pH of Soil for Use in Corrosion Testing. Resistivity testing was completed in accordance with ASTM G 57, Field Measurement of Resistivity Using the Wenner Four-Electrode Method. Sulfate and chloride content testing were completed in general accordance with CP-L 2013, Sulfate Ion Content in Soil and CP-L 2104, Determining the Chloride Ion Content in Water or Water-Soluble Chloride Ion Content in Soil, respectively.

Test results for sulfate and chloride content are given in units of percent by weight. The test results are summarized in Table B-1.

B.3.2 Torsional Ring Shear

Two composite soil/bedrock samples were sent to Cooper Testing Labs, Inc. of Palo Alto, California for torsional ring shear testing in accordance with ASTM D6467, Standard Test Method for Torsional Ring Shear Test to Determine Drained Residual Shear Strength of Cohesive Soils.

The two samples tested were reconstituted from the following samples:

- Composite 1 - SW-01, S-8; SW-03, S-3
- Composite 2 - SW-01, S-11; SW-02, S-9; SW-02, S-10

Detailed test results from Cooper Testing Labs, Inc. are included as Figures B-3 and B-4. Test results are summarized in Table B-2. Using the data provided by Cooper Testing Labs, Inc., we plotted figures of shear stress as a function of effective normal stress for the samples referenced above. Figures B-5 and B-6 compare the torsional ring shear test results to the correlation developed by Stark and Fernandez (2020).

Table B-1 - Summary of Laboratory Test Results

Boring	Sample	Depth (feet)		USCS Symbol ¹	AASHTO Soil Classification	Natural Moisture Content (%)	Moist Unit Weight (pcf)	GRAIN-SIZE ANALYSES ²			ATTERBERG LIMITS			CORROSION			
		Top	Bottom					Gravel (%)	Sand (%)	Fines (%) Silt ³ / Clay ³	LL (%)	PL (%)	PI (%)	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)
	S-1	0.0	2.0			14.6											
	S-2	4.5	6.5	SM	A-1-b (0)	9.3		13	74	13	NV	NP	NP				
	S-3	9.5	10.4			12.9											
	S-4	14.5	16.5			29.7											
	S-5	19.5	21.5			32.1											
	S-6	24.5	26.5	CH	A-7-6 (41)	28.8	100.8	2	10	53 / 35	70	29	41				
	S-7	29.5	31.5			37.9											
SW-01	S-8	34.5	36.5	CH	A-7-6 (117)	34.6				98	129	27	102				
	S-9	39.5	41.5			38.0	115.0										
	S-10	44.5	46.5			40.8											
	S-11	49.5	51.5	CH	A-7-6 (221)	33.6		0	0	28 / 72	215	27	188				
	S-12	54.5	56.5			34.9											
	S-13	59.5	61.4			31.5	125.6										
	S-14	64.5	66.0			27.2											
	S-15	69.5	70.4			40.0											

Table B-1 - Summary of Laboratory Test Results

Boring	Sample	Depth (feet)		USCS Symbol ¹	AASHTO Soil Classification	Natural Moisture Content (%)	Moist Unit Weight (pcf)	GRAIN-SIZE ANALYSES ²			ATTERBERG LIMITS			CORROSION				
		Top	Bottom					Gravel (%)	Sand (%)	Fines (%) Silt ³ / Clay ³	LL (%)	PL (%)	PI (%)	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)	
SW-02	S-1	0.0	2.0			30.7												
	S-2	4.5	6.5			30.3												
	S-3	9.5	11.5			30.4												
	S-4	14.5	16.5	CH	A-7-5 (56)	27.6		0	0	31 / 69	80	34	46					
	S-5	19.5	21.5			28.5												
	S-6	24.5	26.5	CH	A-7-5 (135)	30.8				99	146	31	115					
	S-7	29.5	31.5			28.3												
	S-8	34.5	36.5			30.2												
	S-9	39.5	41.5			42.5												
	S-10	44.5	46.5	CH	A-7-6 (211)	29.7		0	2	18 / 80	209	24	185					
	S-11	49.5	51.5			29.7												
	S-12	54.5	55.0	CH	A-7-6 (45)	18.9				99	62	22	40					
	S-13	59.5	60.5			20.2												
SW-03	S-1	2.0	4.0			41.4												
	S-2	4.5	6.5			39.7												
	S-3	7.0	9.0	CH	A-7-6 (162)	32.9				100	161	22	139					
	S-4	9.5	11.4			22.4	123.9											
	S-5	14.5	16.5			20.7												
	S-6	19.5	21.5	CH	A-7-6 (41)	20.7		0	2	28 / 70	55	17	38					
	S-7	24.5	26.5			24.5												

Table B-1 - Summary of Laboratory Test Results

Boring	SAMPLE DATA		USCS Symbol ¹	AASHTO Soil Classification	Natural Moisture Content (%)	Moist Unit Weight (pcf)	GRAIN-SIZE ANALYSES ²			ATTERBERG LIMITS			CORROSION				
	Sample	Depth (feet) Top Bottom					Gravel (%)	Sand (%)	Fines (%) Silt ³ / Clay ³	LL (%)	PL (%)	PI (%)	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)	
	S-1	2.0 4.0			25.7								7.7	440	1.7	0.04	
	S-2	4.5 6.5	CH	A-7-6 (42)	30.8	119.3			93		61	20	41				
	S-3	7.0 8.5			25.2												
SW-04	S-4	9.5 11.0	CH	A-7-6 (56)	24.2		0	1	29 / 70		69	19	50				
	S-5	14.5 16.0			22.6												
	S-6	19.5 20.9			21.4	123.8											
	S-7	24.5 26.0			20.5												
	S-1	0.0 2.0			15.4												
	S-2	4.5 6.5			14.8												
	S-3	9.5 11.5	SC	A-4 (0)	17.7		6	52	42		29	24	5				
	S-4	14.5 16.5	SC	A-2-4 (0)	17.8		9	57	34		26	16	10				
SW-05	S-7	29.5 31.0	CH	A-7-5 (71)	32.0				98		96	35	61				
	S-8	34.5 35.3			33.0												
	S-9	39.5 41.0			35.2												
	S-10	44.5 46.0	CH	A-7-5 (144)	31.1		0	1	37 / 61		154	31	123				
	S-11	49.5 51.0			28.1												
	S-1	0.0 2.0			44.7												
	S-2	4.5 6.5	CH	A-7-5 (79)	43.5	106.4			96		107	39	68				
	S-3	9.5 11.5	CH	A-7-6 (91)	30.6		0	0	23 / 77		101	23	78				
SW-06	S-4	14.5 16.5			20.7									7.6	490	0.26	0.02
	S-5	19.5 21.0			21.0												
	S-6	24.5 26.0	CH	A-7-6 (41)	22.5	114.1			99		58	21	37				
	S-7	29.5 31.0			23.7												

Table B-1 - Summary of Laboratory Test Results

Boring	SAMPLE DATA		USCS Symbol ¹	AASHTO Soil Classification	Natural Moisture Content (%)	Moist Unit Weight (pcf)	GRAIN-SIZE ANALYSES ²			ATTEBERG LIMITS			CORROSION						
	Sample	Depth (feet)					Top	Bottom	Gravel (%)	Sand (%)	Fines (%) Silt ³ / Clay ³	LL (%)	PL (%)	PI (%)	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)	
SW-07	S-1	0.0	2.0	CH	A-7-6 (12)	20.9			58			51	26	25					
	S-2	4.5	6.5	CL	A-7-6 (11)	21.0	4	36	36 / 24			42	19	23					
	S-3	9.5	11.5			21.8													
	S-5	17.0	19.0			31.9													
	S-6	19.5	21.5	SC	A-7-5 (13)	31.1			41			85	33	52					
	S-7	24.5	26.5			29.0													
	S-8	29.5	31.0			20.6	117.3												
	S-9	34.5	35.9			19.6													
	S-10	39.5	41.0	CH	A-7-6 (48)	21.3		0	1	28 / 71		62	19	43					
	S-11	44.5	46.0			21.7													
	S-12	49.5	50.5			22.8	121.2												
	S-13	54.5	56.0			21.0													
	SW-08	S-1	2.0	4.0			28.4												
S-2		4.5	6.5	CH	A-7-6 (41)	23.4	124.0	0	6	32 / 62	60	20	40						
S-3		9.5	11.5			22.6													
S-4		14.5	16.0			22.9													
S-5		19.5	20.9	CH	A-7-6 (45)	21.2			100			62	23	39					
S-6		24.5	25.9			21.3													

Table B-1 - Summary of Laboratory Test Results

Boring	SAMPLE DATA		USCS Symbol ¹	AASHTO Soil Classification	Natural Moisture Content (%)	Moist Unit Weight (pcf)	GRAIN-SIZE ANALYSES ²			ATTERBERG LIMITS			CORROSION			
	Sample	Depth (feet) Top Bottom					Gravel (%)	Sand (%)	Fines (%) Silt ³ / Clay ³	LL (%)	PL (%)	PI (%)	pH	Resistivity (ohm-cm)	Sulfate Content (%)	Chloride Content (%)
SW-09	S-1	2.0 4.0	CH	A-7-6 (61)	32.2				89	82	20	62				
	S-2	4.5 6.5			34.8											
	S-3	7.0 9.0			35.8	104.6										
	S-4	9.5 11.5	CH	A-7-6 (86)	38.7		0	0	27 / 73	101	29	72				
	S-5	14.5 16.5			42.2											
SW-10	S-6	19.5 21.0	CH	A-7-6 (66)	21.3		0	0	27 / 73	76	18	58				
	S-7	24.5 25.9	CH	A-7-6 (41)	20.3				100	59	23	36				
	S-8	29.5 31.0			20.5											
	S-9	34.5 36.0			20.8											
	S-1	2.0 4.0			31.7											
	S-2	4.5 6.5			32.1											
	S-3	7.0 8.5	CH	A-7-6 (48)	31.2		0	1	27 / 72	62	19	43				
	S-4	9.5 11.0			24.5											
	S-5	14.5 16.0			20.8											
S-6	19.5 21.0	CH	A-7-6 (48)	21.4				98	64	21	43					
S-7	24.5 26.0			23.0												
S-8	29.5 31.0			21.8												

Notes:

1. Refer to Appendix A, Figure A-1 for definitions.
2. Gravel is defined as particles larger than the No. 4 sieve opening size (4.75 mm), sand as particles between the No. 4 and No. 200 (0.075 mm) sieve opening sizes, and fines as silt- and clay-size particles passing the No. 200 sieve.
3. Silt is defined as particles passing the No. 200 sieve opening size larger than 0.002 mm, and clay size particles finer than 0.002 mm. USCS classification is based on fines content and the results of Atterberg limits testing.

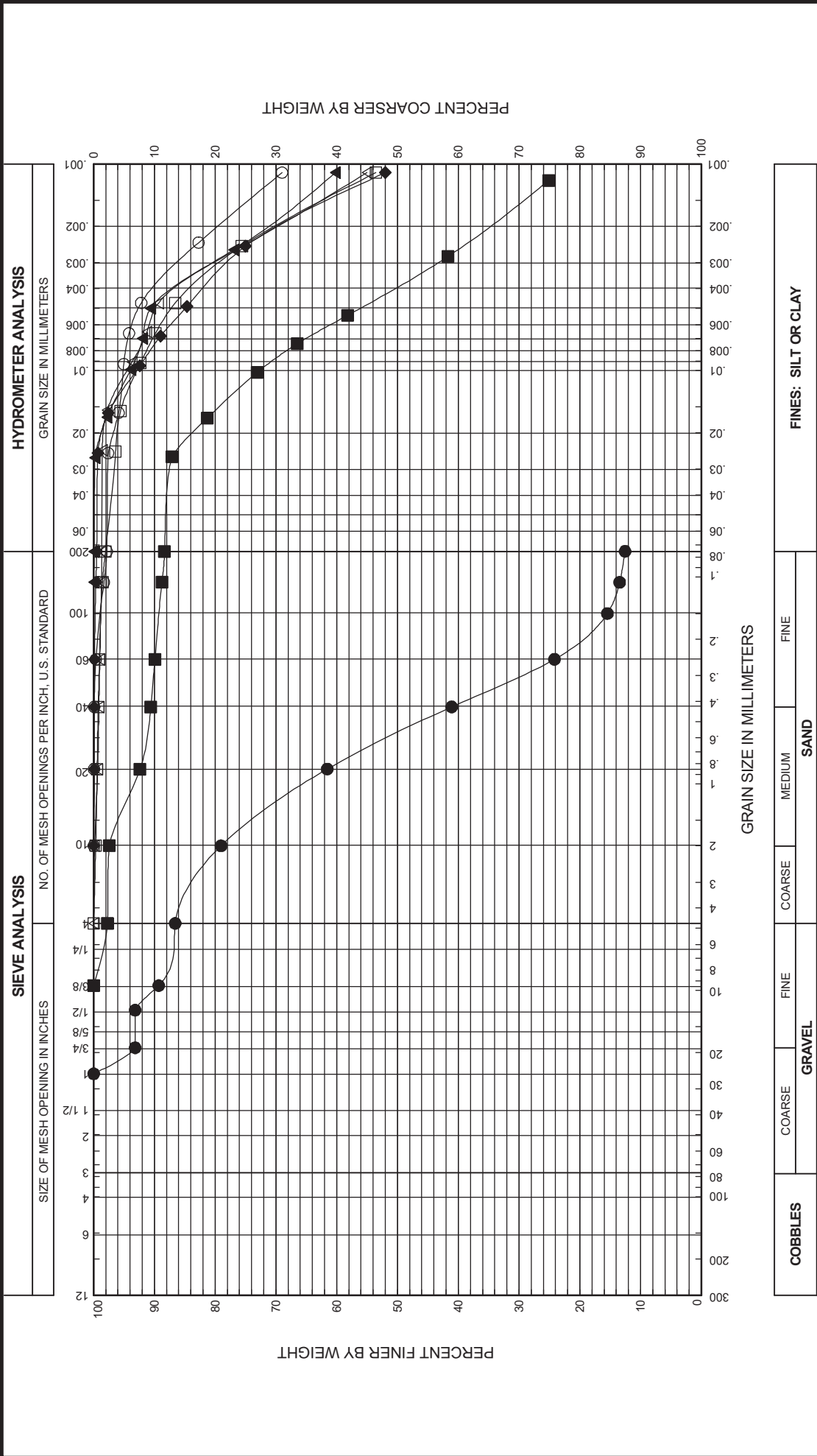
LL = Liquid Limit; NP = Non-Plastic; NV = No Value; PI = Plasticity Index; PL = Plastic Limit; pcf = pounds per cubic foot; USCS = Unified Soil Classification System; ohm-cm = ohm centimeters

Table B-2 - Summary of Torsional Ring Shear Test Results

Sample ID	SAMPLE DATA Borings/Samples	Liquid Limit (%)	Plasticity Index (%)	Clay Fraction (%)	Effective Normal Stress (psf)	Drained Residual Shear Strength (psf)	Drained Residual Secant Phi (deg)
Composite 1	SW-01, S-8, 34.5 to 36.5 ft	129	102	> 50 ¹	522	51	6
	SW-03, S-3; 7.0 to 9.0 ft	161	139		1,044	128	7
					3,133	303	6
					7,832	719	5
Composite 2	SW-01, S-11, 49.5 to 51.5 ft	215	188	72	522	37	4
	SW-02, S-9, 39.5 to 41.5 ft	- ²	- ²	- ²	1,044	67	4
	SW-02, S-10, 44.5-46.5 ft	209	185	80	3,133	249	5
					7,832	794	6

Notes:

- 1 Clay fraction was not directly measured from Composite 1 samples, and is estimated based on results of hydrometer testing on samples from the same soil/rock unit.
 - 2 Grain size and Atterberg limit tests were not completed for boring SW-02, sample S-9.
- ft = feet; psf = pounds per square foot; deg = degrees



SAMPLE ID	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %		NAT. W.C. %	LL %	PL %	PI %
				12.5	88.4				
● SW-01, S-2	4.5	SM	Silty Sand; few gravel	12.5	88.4	9.3	NV	NP	NP
■ SW-01, S-6	24.5	CH	DISPLACED SHALE [Fat Clay; trace gravel; few sand]	99.9	99.9	28.8	70	29	41
▲ SW-01, S-11	49.5	CH	CLAYSTONE/SHALE [Fat Clay; trace sand]	99.5	99.5	33.6	215	27	188
◆ SW-02, S-4	14.5	CH	DISPLACED CLAYSTONE [Fat Clay; trace sand]	97.8	97.8	27.6	80	34	46
○ SW-02, S-10	44.5	CH	DISPLACED CLAYSTONE [Fat Clay; trace sand]	98.1	98.1	29.7	209	24	185
□ SW-03, S-6	19.5	CH	CLAYSTONE [Fat Clay; trace sand]	98.6	98.6	20.7	55	17	38
△ SW-04, S-4	9.5	CH	CLAYSTONE [Fat Clay; trace sand]	98.6	98.6	24.2	69	19	50

Valley City SW Backslope Slide Repair
2-094(185)290, PCN 23338
Valley City, North Dakota

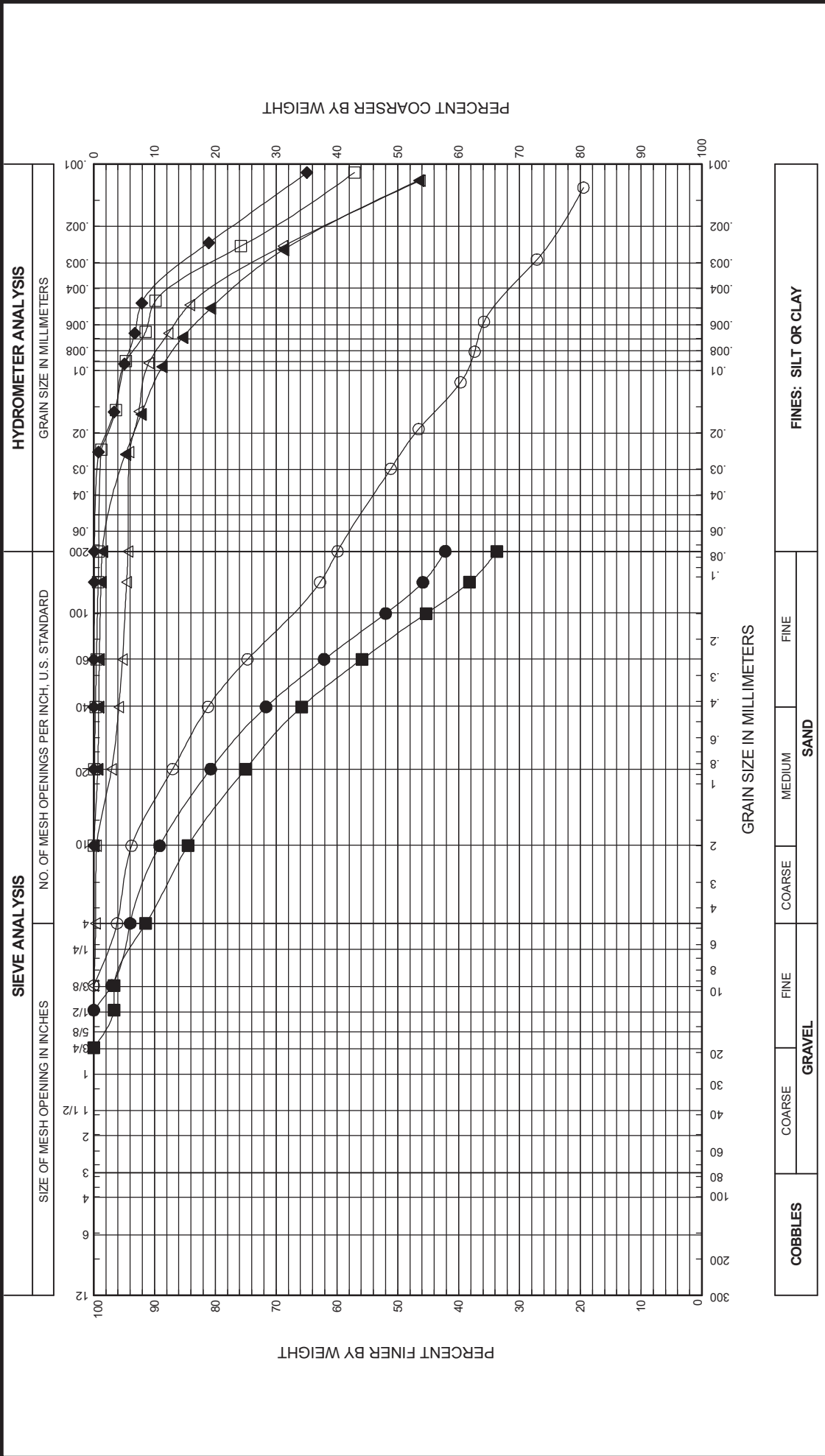
GRAIN SIZE DISTRIBUTION

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FIG. B-1
Sheet 1 of 3

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SAMPLE ID	DEPTH (feet)	U.S.S. SYMBOL	SAMPLE DESCRIPTION	FINES %		NAT. W.C. %	LL %	PL %	PI %
				COARSE	FINE				
● SW-05, S-3	9.5	SC	Clayey Sand; few gravel	42.2	57.8	17.7	29	24	5
■ SW-05, S-4	14.5	SC	Clayey Sand; few gravel	33.7	66.3	17.8	26	16	10
▲ SW-05, S-10	44.5	CH	SHALE [Fat Clay; trace sand]	98.6	1.4	31.1	154	31	123
◆ SW-06, S-3	9.5	CH	CLAYSTONE [Fat Clay]	99.9	0.1	30.6	101	23	78
○ SW-07, S-2	4.5	CL	Sandy Lean Clay; trace gravel	59.9	40.1	21.0	42	19	23
□ SW-07, S-10	39.5	CH	SHALE [Fat Clay; trace sand]	99.2	0.8	21.3	62	19	43
△ SW-08, S-2	4.5	CH	CLAYSTONE [Fat Clay; trace sand]	94.4	5.6	23.4	60	20	40

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Valley City, North Dakota

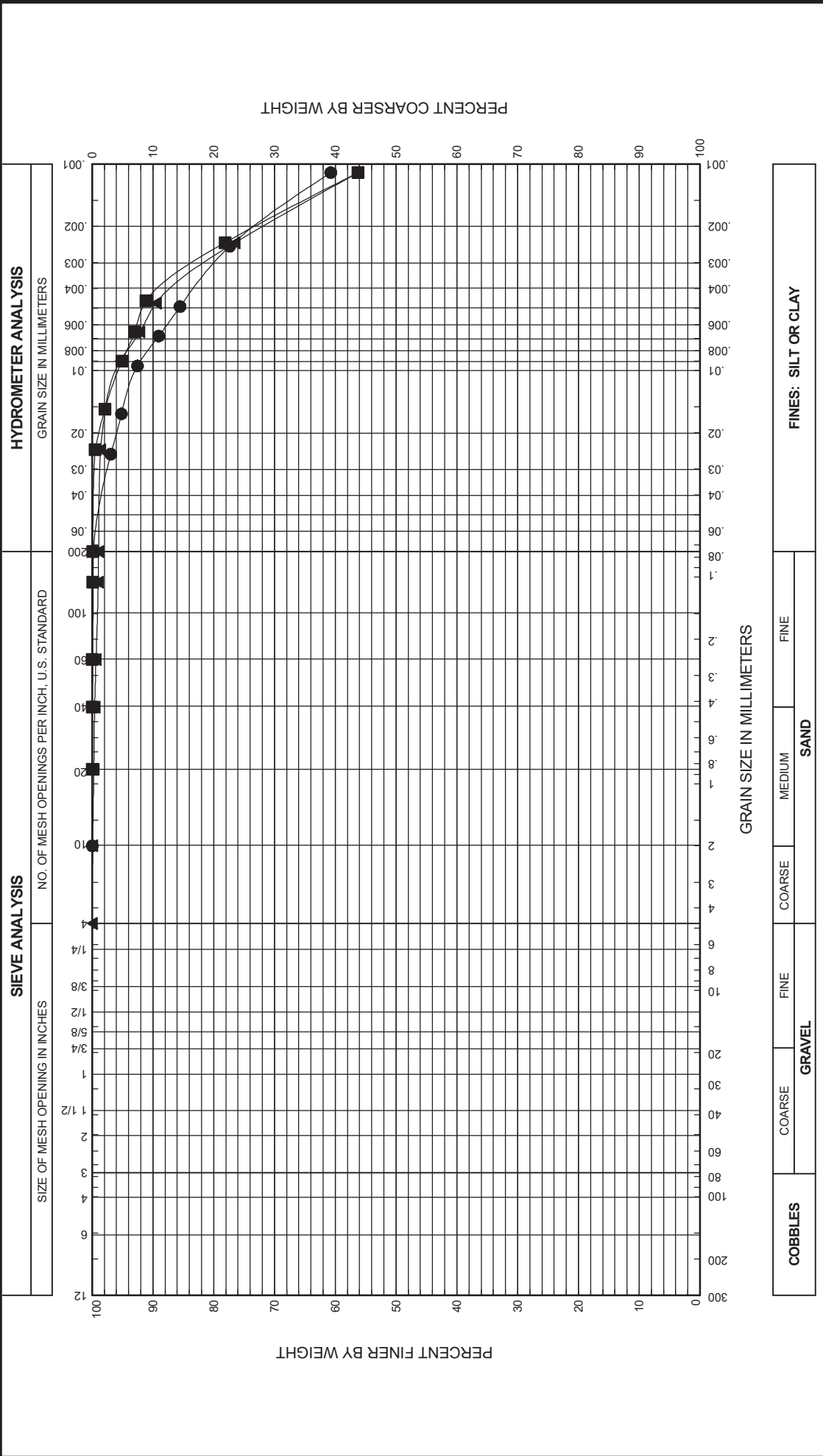
GRAIN SIZE DISTRIBUTION

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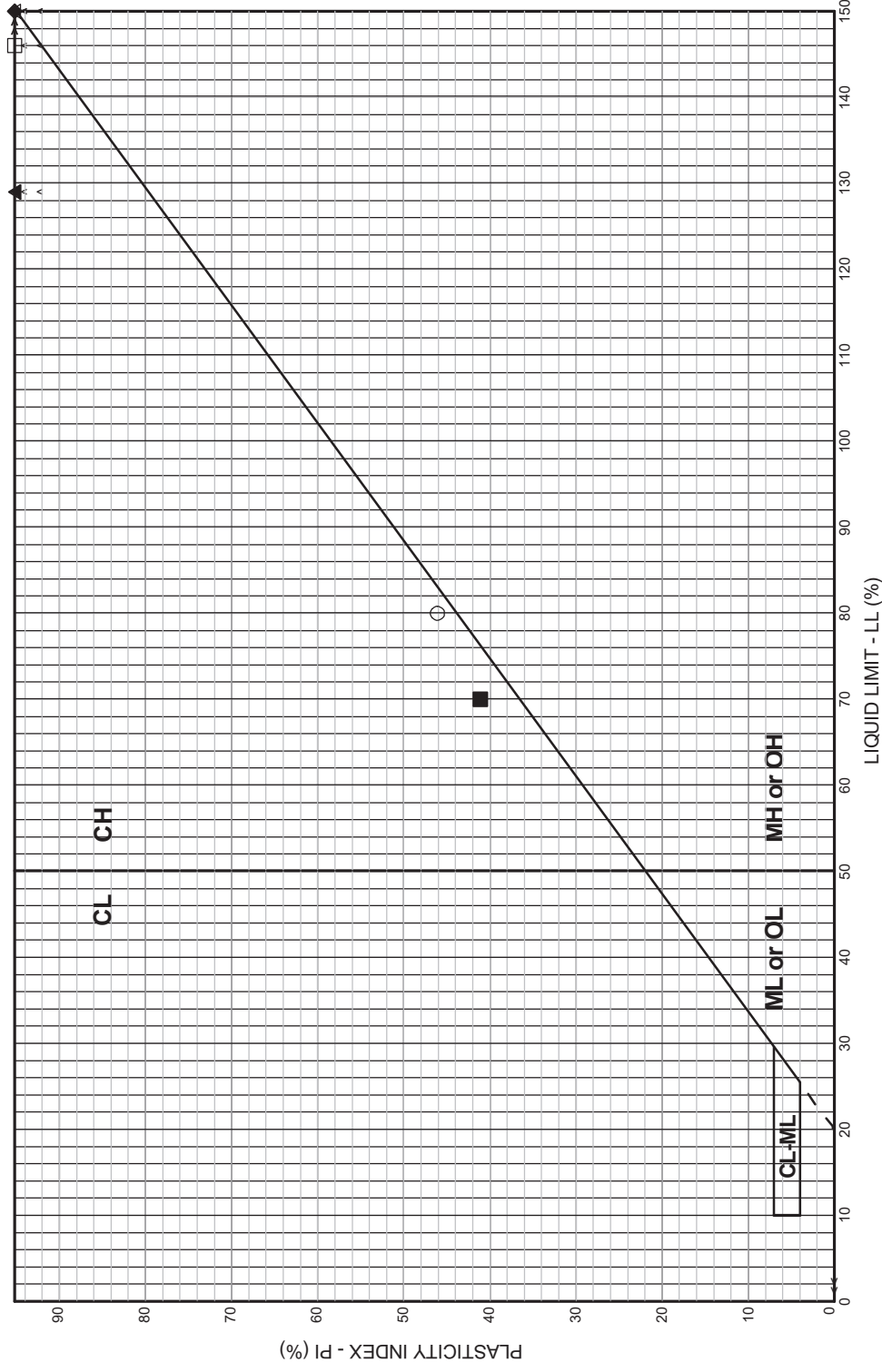
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FIG. B-1
Sheet 2 of 3



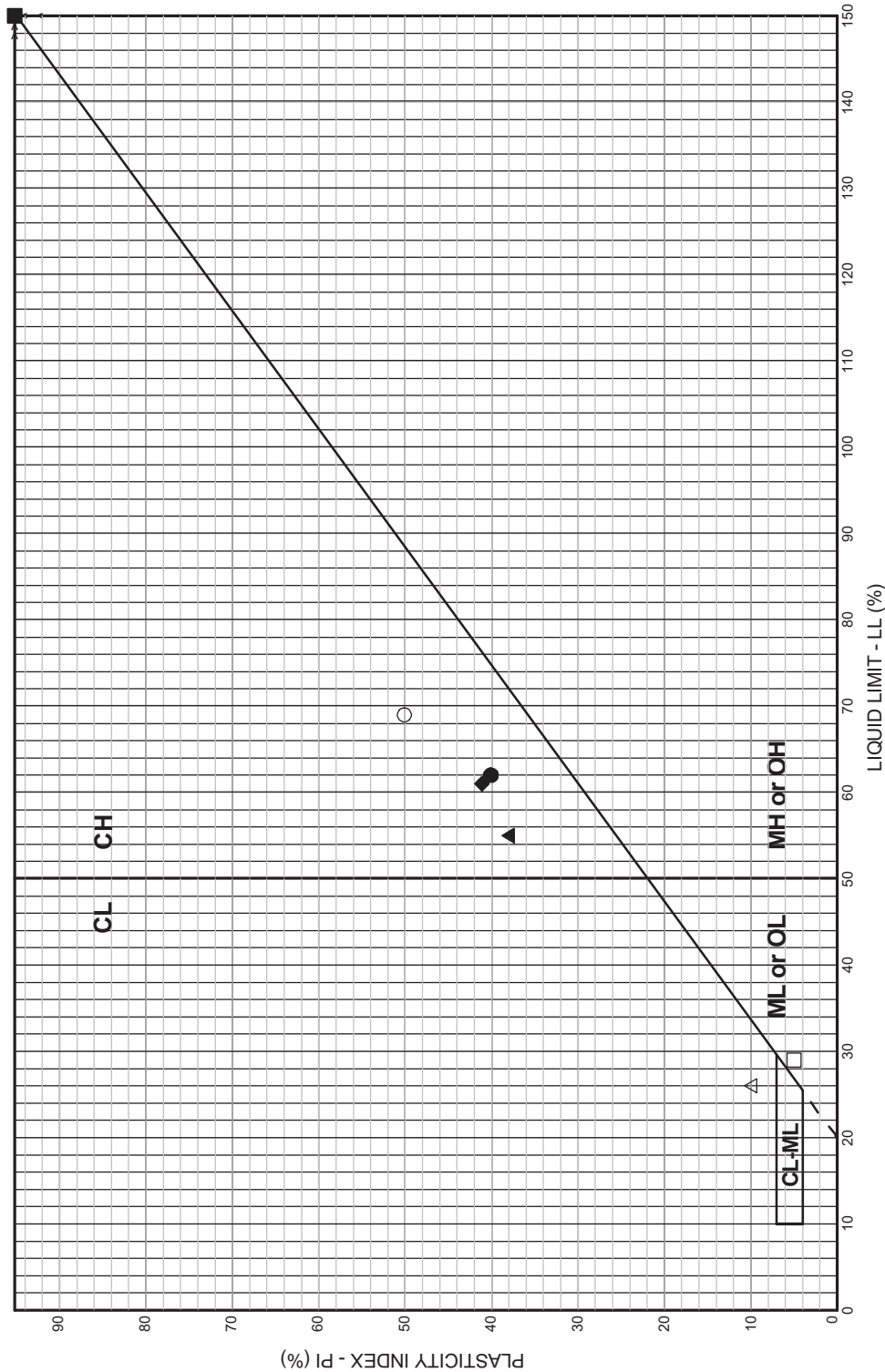
SAMPLE ID	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %		NAT. W.C. %	LL %	PL %	PI %
				99.8	99.9				
● SW-09, S-4	9.5	CH	DISPLACED CLAYSTONE [Fat Clay; trace sand]	99.8	99.9	38.7	101	29	72
■ SW-09, S-6	19.5	CH	CLAYSTONE [Fat Clay]	99.9	99.9	21.3	76	18	58
▲ SW-10, S-3	7.0	CH	CLAYSTONE [Fat Clay; trace sand]	98.9	98.9	31.2	62	19	43

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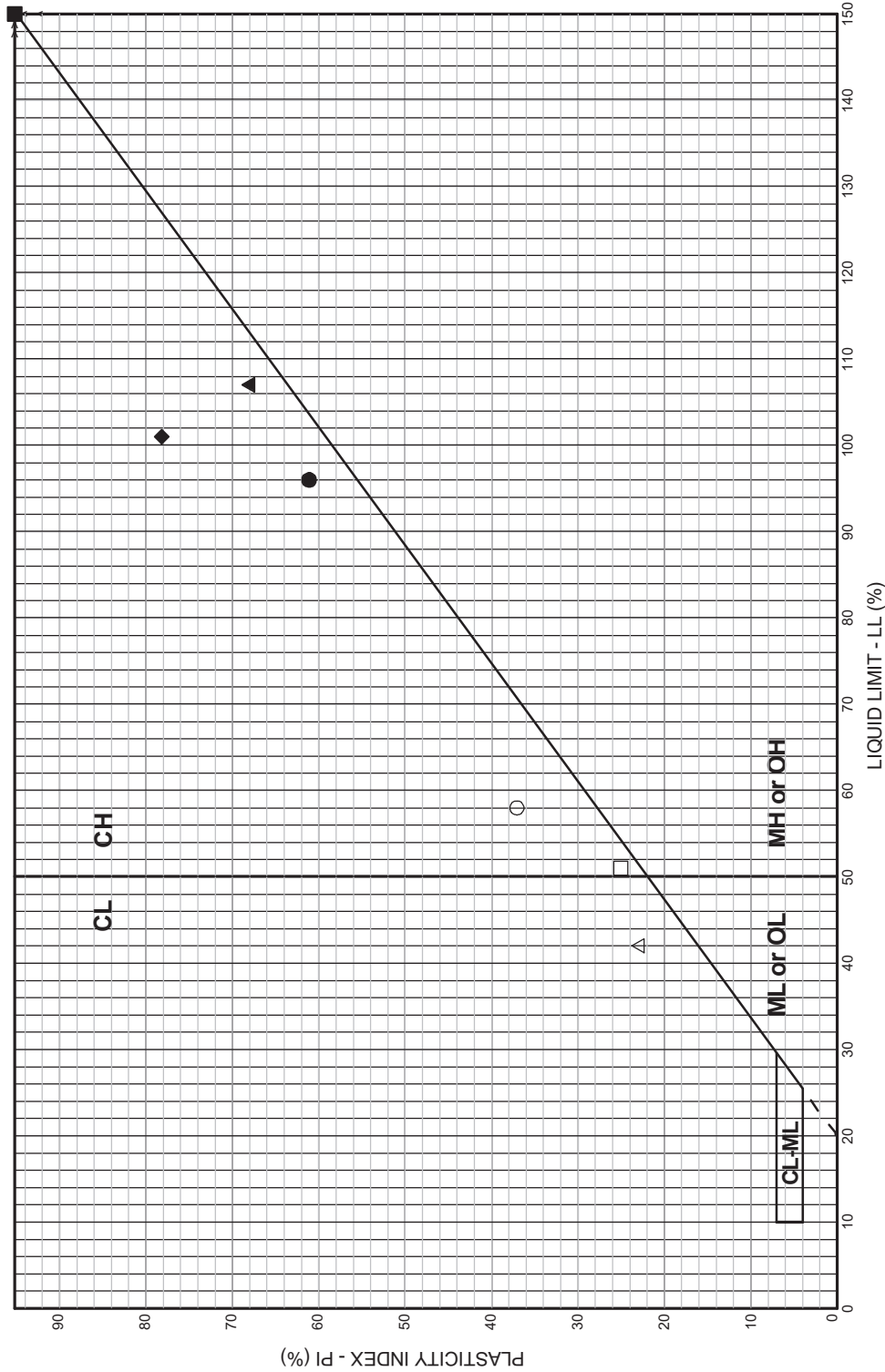
Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
PLASTICITY CHART	
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FIG. B-2



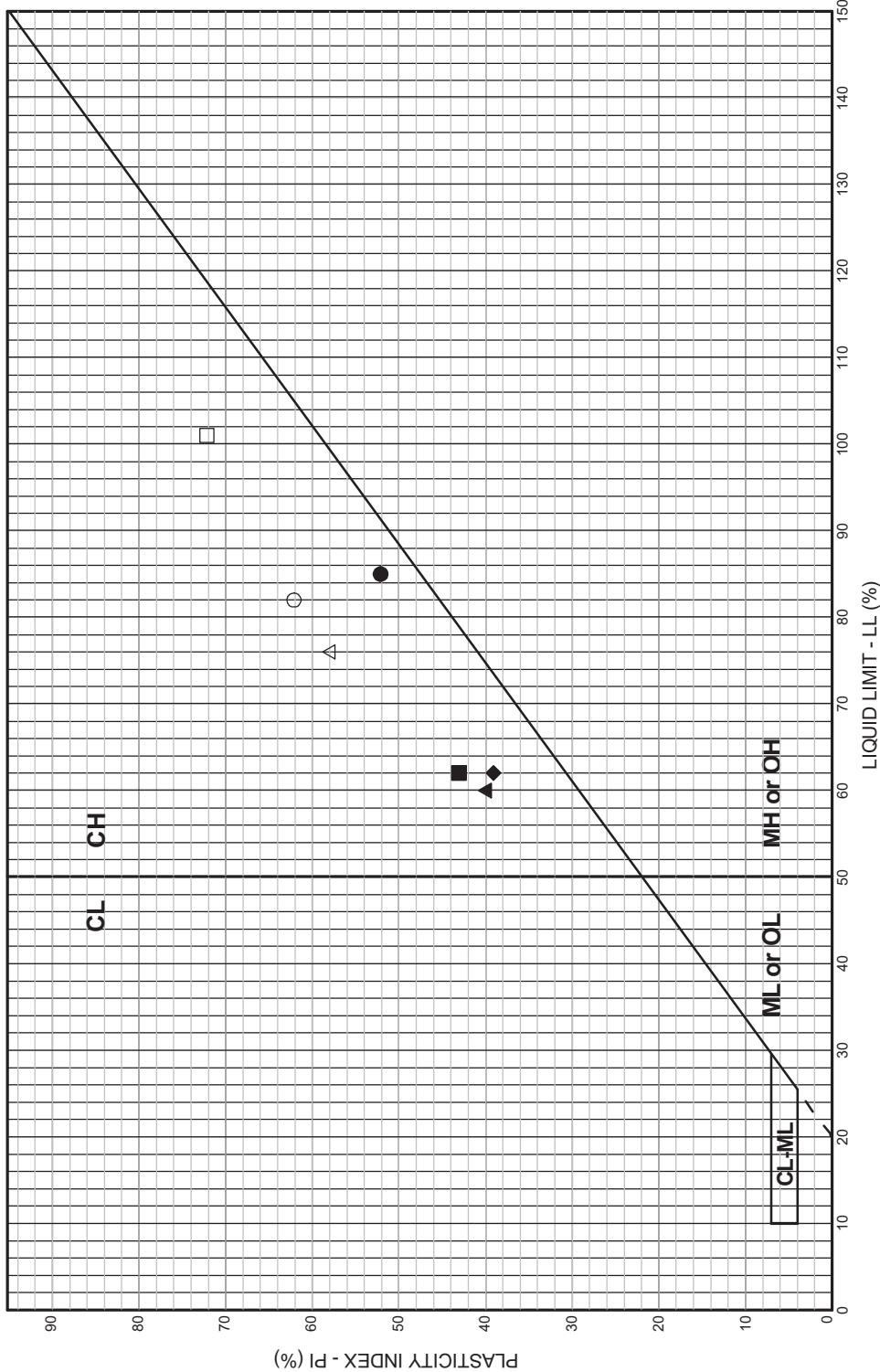
Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
PLASTICITY CHART	
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FIG. B-2



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FIG. B-2

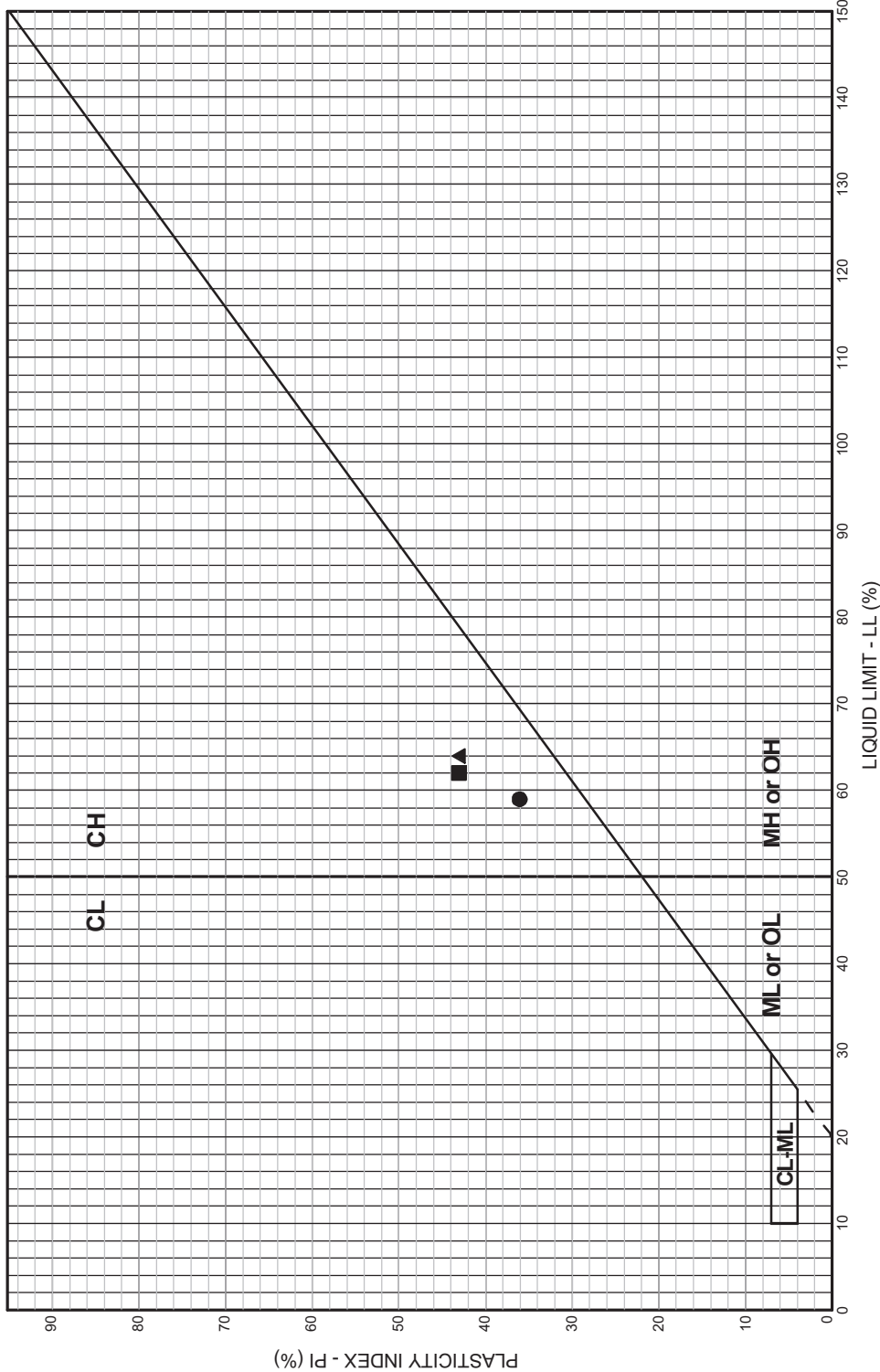


LEGEND

- CL:** Low plasticity inorganic clays; sandy and silty clays
- CH:** High plasticity inorganic clays
- ML or OL:** Inorganic and organic silts and clayey silts of low plasticity
- MH or OH:** Inorganic and organic silts and clayey silts of high plasticity
- CL-ML:** Silty clays and clayey silts

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PLASTICITY CHART	
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FIG. B-2 Sheet 4 of 5	

FIG. B-2



LEGEND

- CL:** Low plasticity inorganic clays; sandy and silty clays
- CH:** High plasticity inorganic clays
- ML or OL:** Inorganic and organic silts and clayey silts of low plasticity
- MH or OH:** Inorganic and organic silts and clayey silts of high plasticity
- CL-ML:** Silty clays and clayey silts

BORING AND SAMPLE NO.		SW-09, S-7	SW-10, S-3	SW-10, S-6
U.S.C.S. SYMBOL		CH	CH	CH
DEPTH (feet)		24.5	7.0	19.5
SOIL CLASSIFICATION		SHALE [Fat Clay]	CLAYSTONE [Fat Clay; trace sand]	SHALE/CLAYSTONE [Fat Clay; trace sand]
LL %	PL %	PI %	NAT. W.C. %	PASS #200 %
59	23	36	20.3	99.6
62	19	43	31.2	98.9
64	21	43	21.4	97.7
Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota				
PLASTICITY CHART				
March 2023		107877-001		
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FIG. B-2



Drained Residual Torsional Shear Strength (ASTM D6467)

CTL Job No.:	645-116	Boring:	SW-01 & SW-03	Date:	10/5/2022	Clay, %:	
Client:	Shannon & Wilson	Sample:	S-8 & S-3	By:	PJ	LL:	
Project Name:	Valley City	Depth (ft):	34.5' & 7'	Checked:	DC	PL:	
Project Number:	107877-001	Test Type:	Reconstituted Residual				
Soil Type: Gray CLAY		Remarks: A small friction correction was applied to each point. The shear loads encountered in this testing were at the lower limits of the equipment, and in this range (25kpa normal load) the resolution of the equipment can result in up to 4psf (~1 degree) of uncertainty.					
Normal Load (psf):	522	1044	3133	7832			
Secant Phi (deg):	6	7	6	5			

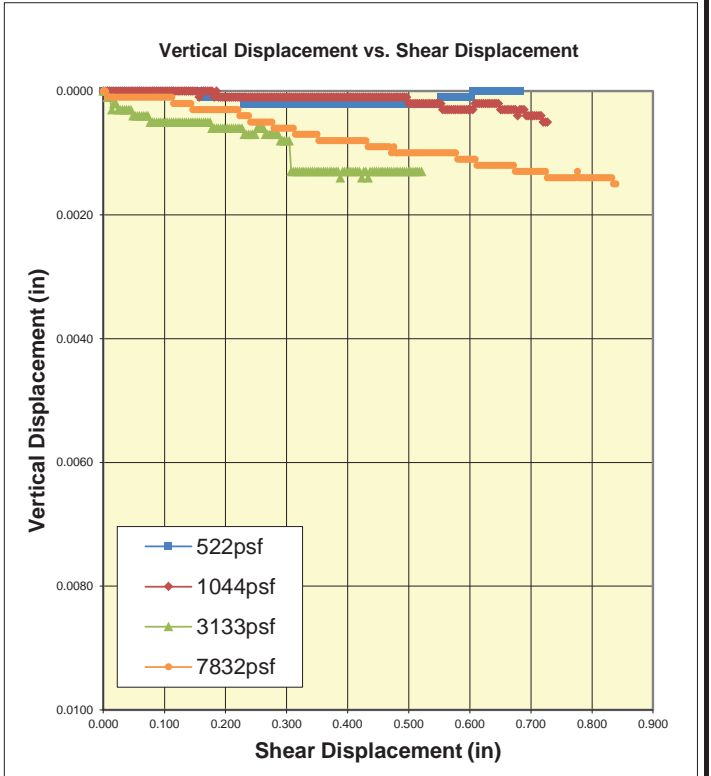
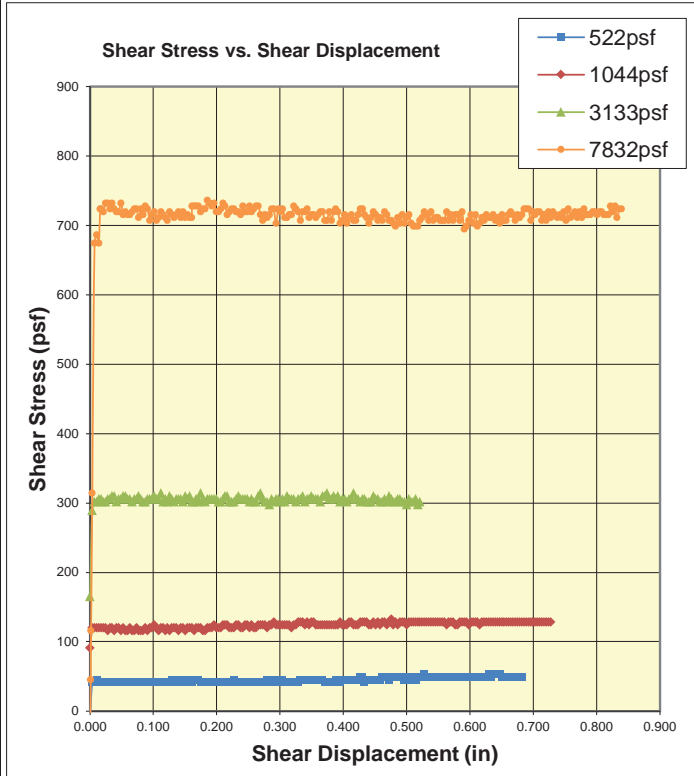
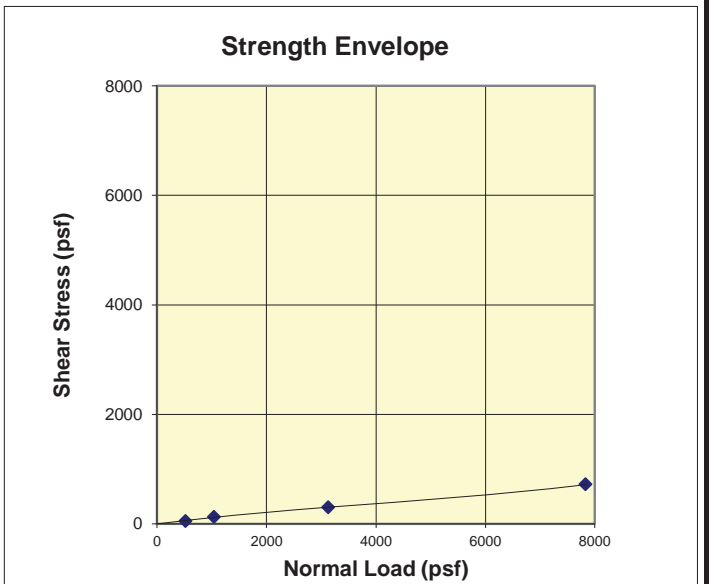
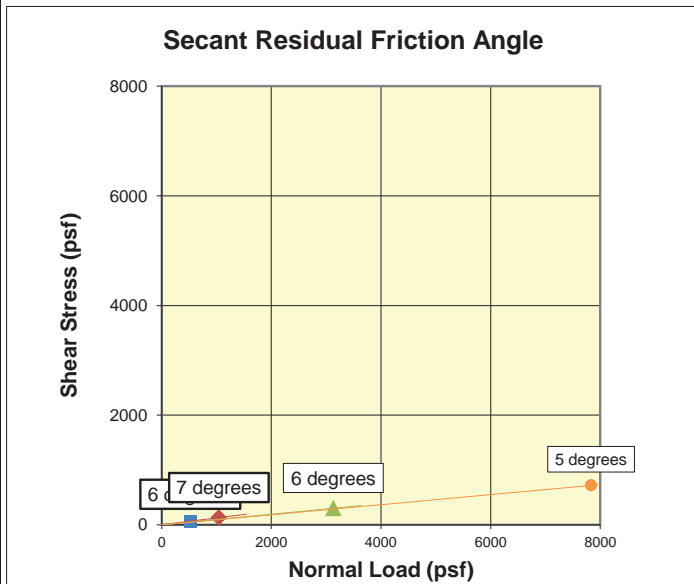


Figure B-3



Drained Residual Torsional Shear Strength (ASTM D6467)

CTL Job No.:	645-116	Boring:	SW-01 & SW-02	Date:	10/17/2022	Clay, %:	
Client:	Shannon & Wilson	Sample:	S-11, S-9, & S-10	By:	PJ	LL:	
Project Name:	Valley City	Depth (ft):	49.5', 39.5', & 44.5'	Checked:	DC	PL:	
Project Number:	107877-001	Test Type:	Reconstituted Residual				
Soil Type: Gray CLAY		Remarks: A small friction correction was applied to each point. The shear loads encountered in this testing were at the lower limits of the equipment, and in this range (25kpa normal load) the resolution of the equipment can result in up to 4psf (~1 degree) of uncertainty.					
Normal Load (psf):	522	1044	3133	7832			
Secant Phi (deg):	4	4	5	6			

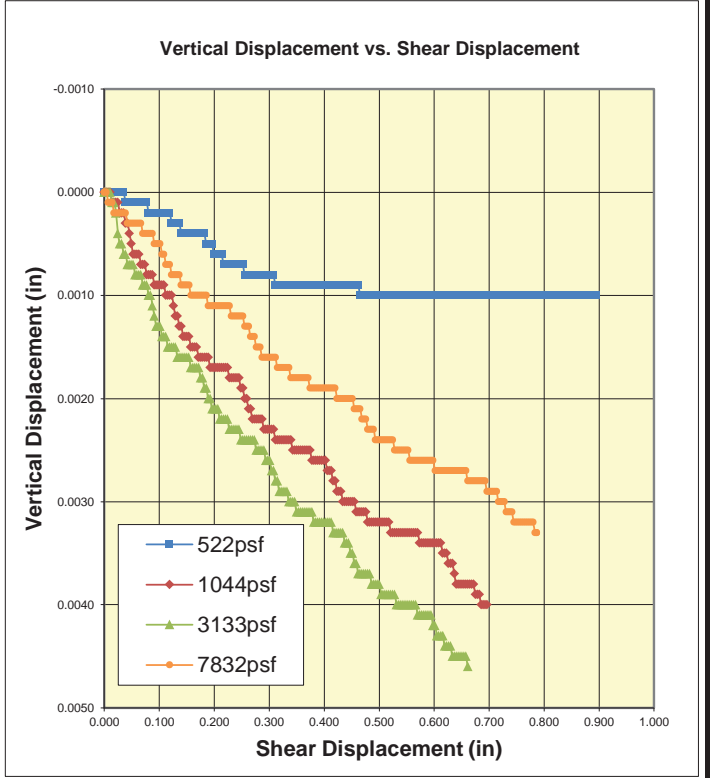
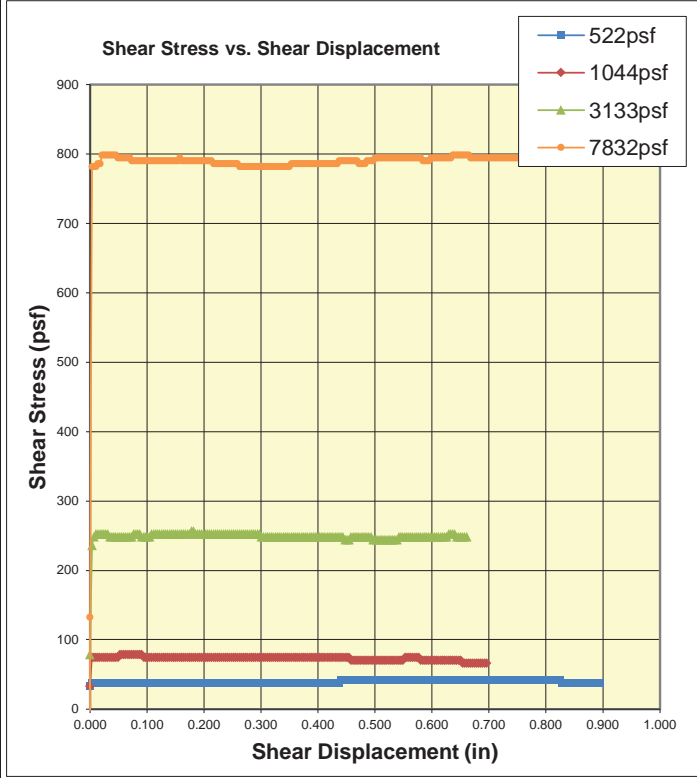
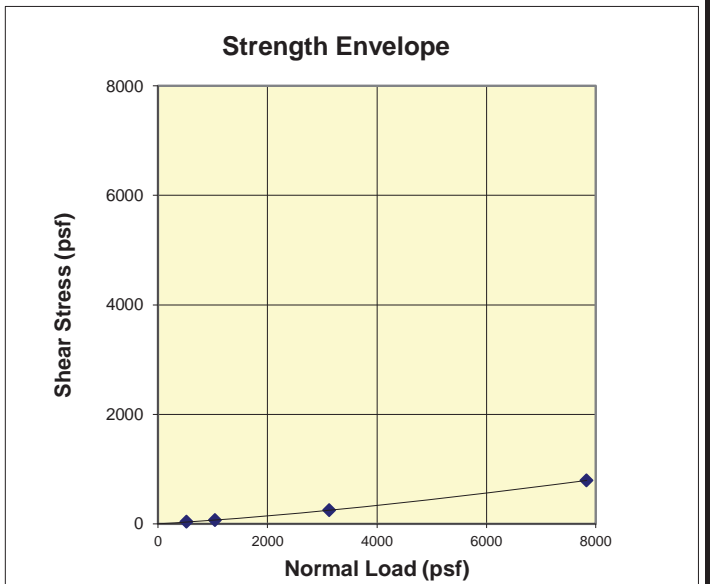
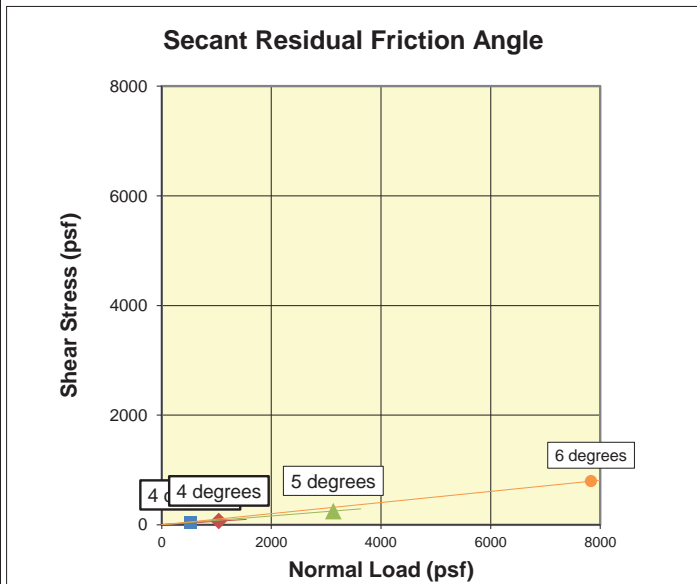
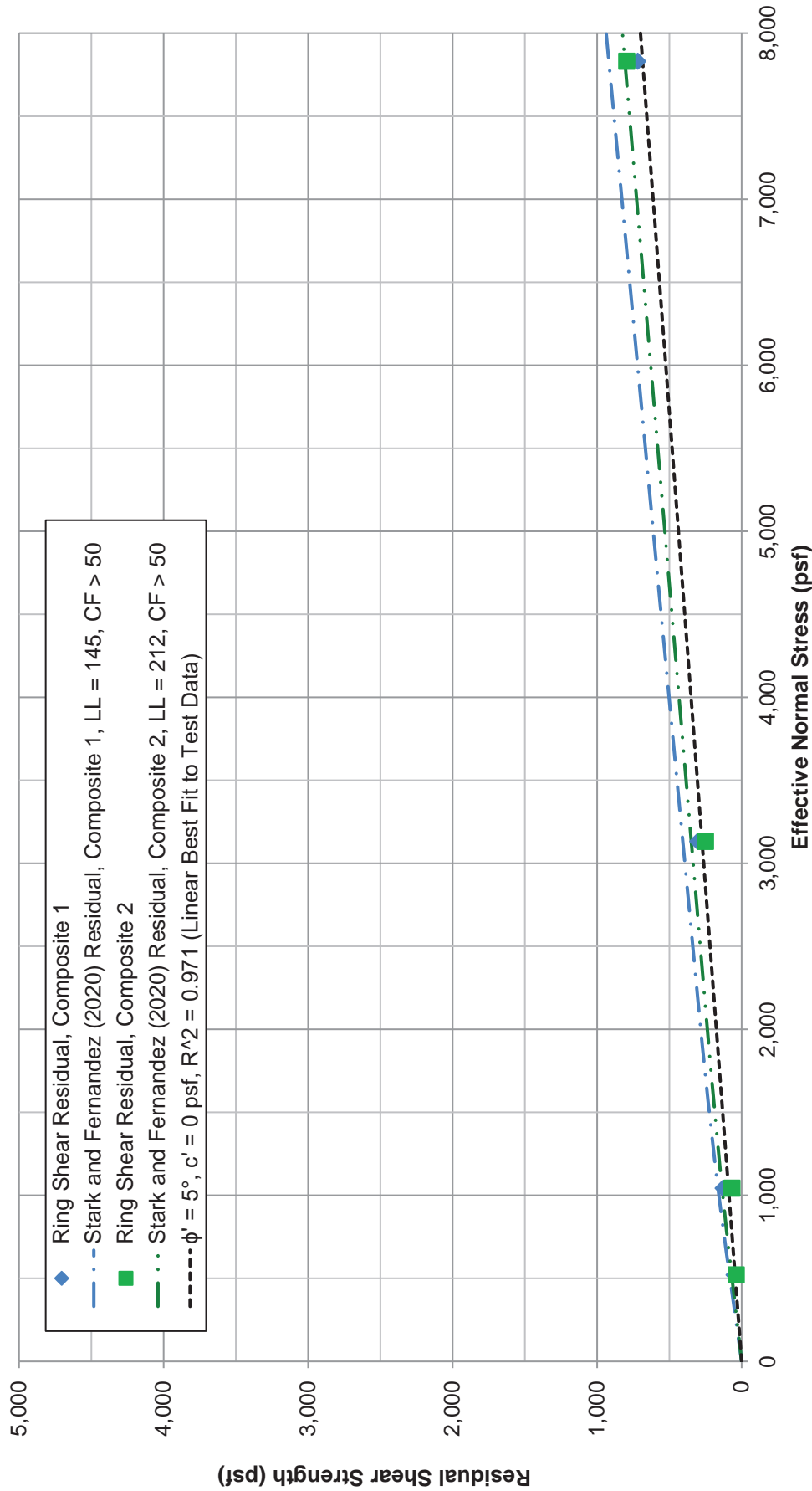


Figure B-4



Note:
Liquid Limit (LL) values used in Stark and Fernandez (2020) correlation were estimated by averaging individual LL test results from the composite samples.

Boring & Sample	Point	Normal Stress (psf)	Shear Stress (psf)
Composite 1 (Table B-2)	A	522	51
	B	1,044	128
	C	3,133	303
	D	7,832	719
Composite 2 (Table B-2)	A	522	37
	B	1,044	67
	C	3,133	249
	D	7,832	794

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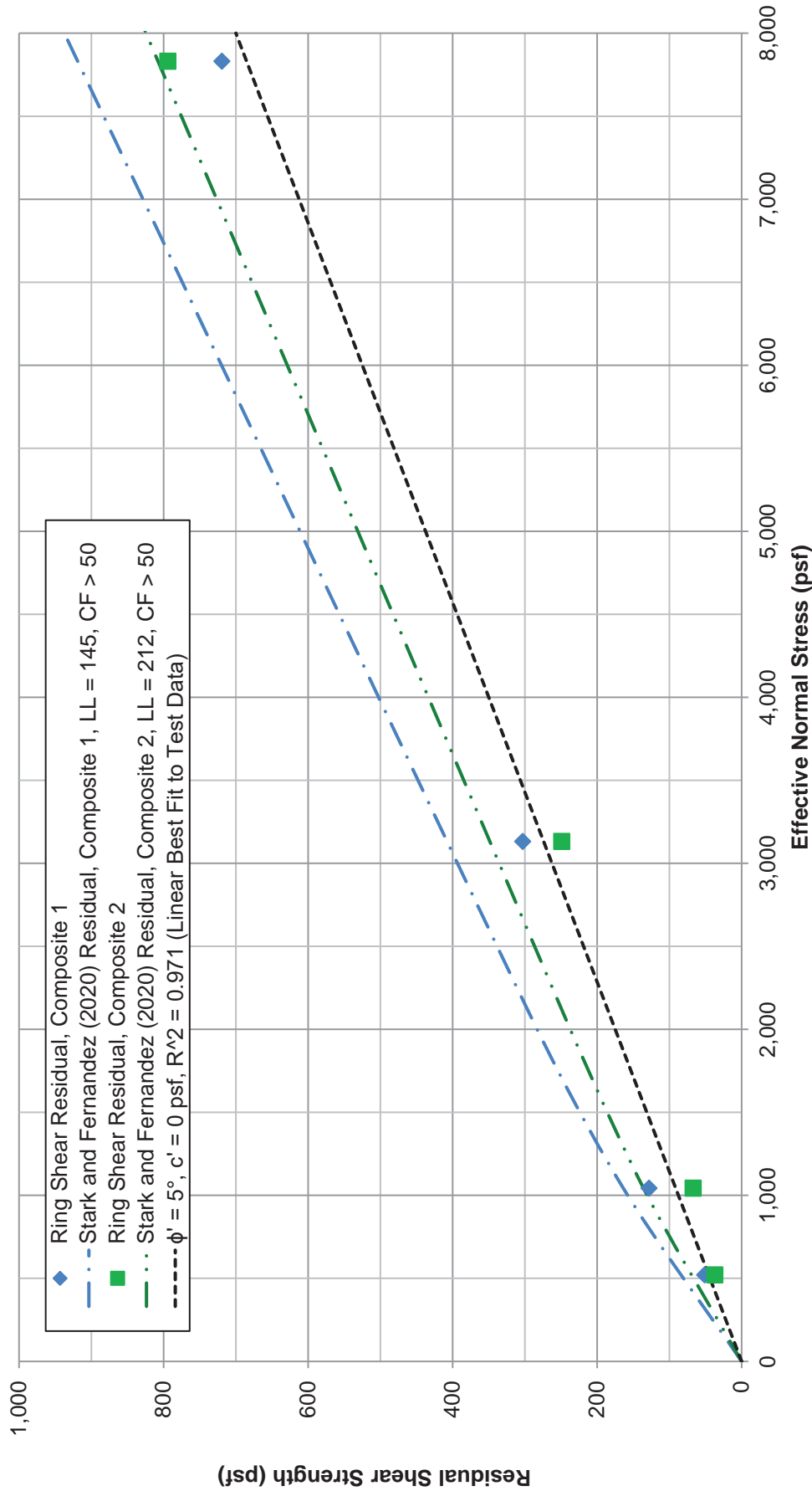
**TORSIONAL RING SHEAR,
DRAINED RESIDUAL SHEAR
STRENGTH ENVELOPE**

March 2023 107877-001

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FIG. B-5

FIG. B-5



Note:
Liquid Limit (LL) values used in Stark and Fernandez (2020) correlation were estimated by averaging individual LL test results from the composite samples.

Boring & Sample	Point	Normal Stress (psf)	Shear Stress (psf)
Composite 1 (Table B-2)	A	522	51
	B	1,044	128
	C	3,133	303
	D	7,832	719
Composite 2 (Table B-2)	A	522	37
	B	1,044	67
	C	3,133	249
	D	7,832	794

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Valley City, North Dakota

**TORSIONAL RING SHEAR,
DRAINED RESIDUAL SHEAR
STRENGTH ENVELOPE (DETAIL)**

March 2023 107877-001

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FIG. B-6

FIG. B-6

Appendix C

Instrumentation

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C.3	Inclinometer Readings.....	C-1
C.4	Vibrating Wire Piezometer Installation	C-1
C.5	Groundwater Elevation Calculation	C-2
C.6	Time-Domain Reflectometer Installation.....	C-3
C.7	Time-Domain Reflectometer Readings	C-3

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Figure C-3: Inclinator Profile Change SW-03

Figure C-4: Inclinator Profile Change SW-04

Figure C-5: Inclinator Profile Change SW-06

Figure C-6: Inclinator Profile Change SW-07

Figure C-7: Inclinator Profile Change SW-08

Figure C-8: Inclinator Profile Change SW-09

Figure C-9: Inclinator Profile Change SW-10

Figure C-10: Groundwater Measurements, SW-01 Vibrating Wire Piezometer

Figure C-11: Groundwater Measurements, SW-02 Vibrating Wire Piezometer

Figure C-12: Groundwater Measurements, SW-03 Vibrating Wire Piezometer

Figure C-13: Groundwater Measurements, SW-05 (40') Vibrating Wire Piezometer

Figure C-14: Groundwater Measurements, SW-05 (25') Vibrating Wire Piezometer

Figure C-15: Groundwater Measurements, SW-07 Vibrating Wire Piezometer

Figure C-16: Groundwater Measurements, SW-09 Vibrating Wire Piezometer

Figure C-17: TDR-Inclinator Comparison, Boring SW-03

Figure C-18: TDR-Inclinator Comparison, Boring SW-07

Figure C-19: TDR-Inclinator Comparison, Boring SW-09

C.1 INTRODUCTION

As part of our subsurface explorations, Shannon & Wilson installed instrumentation consisting of inclinometers, vibrating wire piezometers (VWPs), and time-domain reflectometers (TDRs) at select boring locations as indicated in Figure 2. Installation of the instruments was observed by a Shannon & Wilson field representative. The following sections present details of instrumentation installation, readings, and data reduction.

C.2 INCLINOMETER INSTALLATION

Shannon & Wilson installed nine inclinometer casings at the locations indicated in Figure 2. The inclinometer installations were completed using 2.75-inch diameter Slope Indicator casing with VWPs and TDRs secured to the outside of the casing, where used. After the casing was lowered to the bottom of the borehole, a bentonite-cement grout mixture was pumped to the bottom of the borehole through a hose that was secured to the outside of the inclinometer casing. To counter buoyancy, water was poured into the inclinometer casing before grouting. Following installation, the instrumentation was completed with a 4-inch square stick up monument.

C.3 INCLINOMETER READINGS

On June 22, 2022, a representative from NDDOT initialized the inclinometers, using a NDDOT probe. After initialization, NDDOT has continued to collect periodic readings for all inclinometers at the site. Plots of inclinometer profile change are provided in Figures C-1 through C-10. Bias corrections were applied to individual inclinometers if bias error was apparent and are indicated on each plot (where used).

C.4 VIBRATING WIRE PIEZOMETER INSTALLATION

A total of seven VWPs were installed in borings SW-01, SW-02, SW-03, SW-05, SW-07, and SW-09. Each VWP consists of a pressure transducer contained in a stainless-steel housing. Water pressure acts against a low air entry filter at one end of the housing. Geokon Model No. 4500S VWPs with a pressure range of 350 kilopascals were used for the seven installations. Borehole SW-05 has two VWPs, one at a depth of 25 feet and the other at a depth of 40 feet below ground surface.

Boring locations where the VWP's were installed are shown in Figure 2, and the approximate elevations of VWP installation are shown on the subsurface profiles in Figures 3 through 5. The VWP's were secured to the inclinometer casing at the desired installation depth before the casing was lowered into the borehole as described in Section C.2. The fully grouted borehole installation method described in Section 4.2 of the manufacturer's manual (Geokon, 2021)¹ was used to complete the installation of the VWP's.

C.5 GROUNDWATER ELEVATION CALCULATION

To read the instrument, the VWP's were connected to a signal cable that was routed up the borehole to the ground surface. Prior to installation, each VWP transducer was saturated and then read (pressure and temperature) while under zero water head. These data were recorded along with the factory calibration data summarized in Table C-1 to interpret future readings.

Shannon & Wilson recorded the baseline readings for each of the VWP's prior to installation. Subsequent readings were completed by NDDOT personnel. The VWP's at borings SW-01, SW-03, and SW-05 were connected to dataloggers which stored readings taken every two hours. Personnel from NDDOT made periodic site visits to take discrete VWP readings and collect the readings stored in the dataloggers. These data were reported to Shannon & Wilson and then used to calculate the water pressure, which was converted to groundwater elevation. Plots of groundwater elevation versus time are included as Figures C-10 through C-16. The highest groundwater elevations measured are shown on individual boring logs in Appendix A.

¹ Geokon, 2021, Model 4500 Series Vibrating Wire Piezometer: Instruction Manual, Geokon, Inc., Lebanon, New Hampshire, Rev. PP, January 28, 2021.

C.6 TIME-DOMAIN REFLECTOMETER INSTALLATION

A total of three TDRs were installed, in borings SW-03, SW-07, and SW-09. Each TDR consists of a length of coaxial cable that runs along the inclinometer casing. The TDR was secured to the casing before it was lowered into the borehole, as described in Section C.2.

C.7 TIME-DOMAIN REFLECTOMETER READINGS

Time domain reflectometers (TDRs) measure reflections of a wave through an electrical conductor (i.e., the cable wire). These reflections can be correlated to displacements or discontinuities along the wire, which in turn can be interpreted to indicate locations of slope movement.

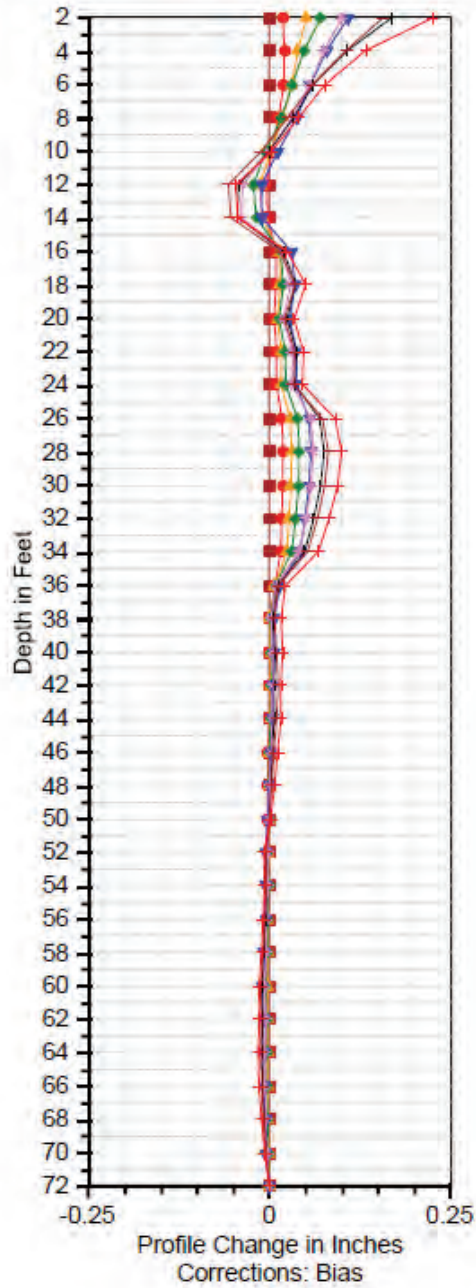
On June 16, 2022, a representative from NDDOT initialized the TDRs, using a portable TDR200 readout that S&W provided to the NDDOT. After initialization, NDDOT has continued to monitor and complete TDR readings. Plots of TDR reflection are shown on Figures C-17 through C-19, with plots of inclinometer resultant profile change superimposed for comparison.

Table C-1 - Calibration Data for Groundwater Calculation

Exploration Designation	Model Number	Serial Number	Sensor Depth (ft)	Linear Gage Factor (psi/digit)	Thermal Factor (psi/°C)	Factory Zero Reading (digits)	Temperature (°C)	Barometer (mbar)	Initial Field Zero Reading (digits)	Initial Field Temperature (°C)
SW-01	4500S-350kPa	2223056	33	-0.01765	0.00924	8690	24.4	1011	8742.5	24.8
SW-02	4500S-350kPa	2223057	45	-0.01717	0.02952	7275	24.4	1011	7330.7	23.7
SW-03	4500S-350kPa	2223101	19	-0.01686	0.005256	8808	22.8	993.5	8862.9	26.9
SW-05 (40')	4500S-350kPa	2223103	40	-0.01684	0.01779	8607	22.8	993.5	8639.2	21.8
SW-05 (25')	4500S-350kPa	2223099	25	-0.01623	0.004561	8924	22.8	993.5	8975.8	22.6
SW-07	4500S-350kPa	2223100	17.8	-0.01661	0.005645	8859	22.8	993.5	8916.9	22.8
SW-09	4500S-350kPa	2223098	17.5	-0.01765	0.002983	8934	22.8	993.5	8985.1	22.6

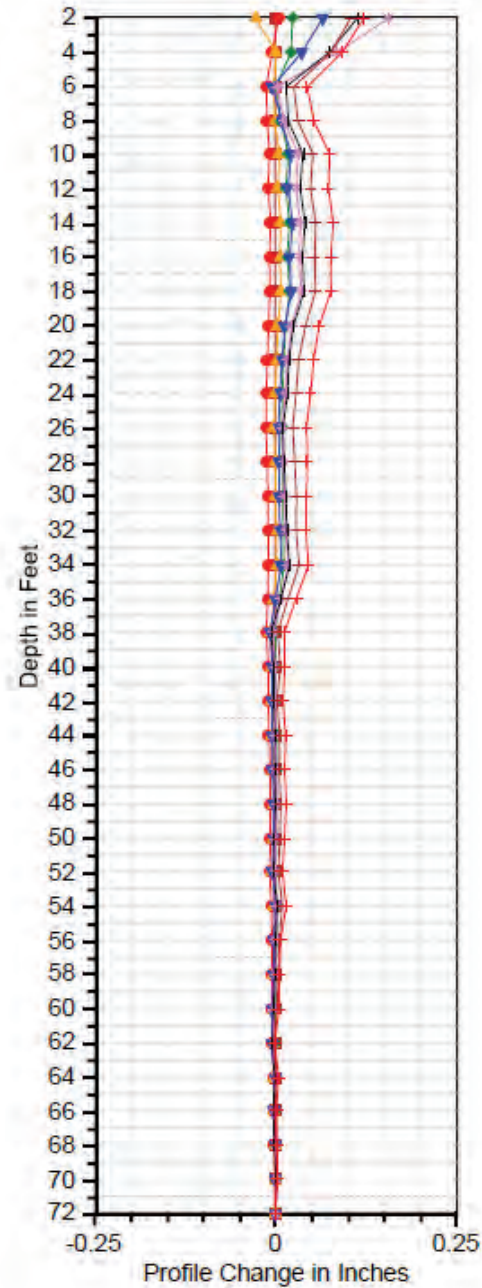
VCBS-SW-01 A

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◆ 7/12/2022 ▼ 7/19/2022 ◇ 8/18/2022
✕ 9/27/2022 + 11/3/2022 * 12/5/2022



VCBS-SW-01 B

■ 6/22/2022 ● 6/28/2022 ▲ 7/6/2022
◆ 7/12/2022 ▼ 7/19/2022 ◇ 8/18/2022
✕ 9/27/2022 + 11/3/2022 * 12/5/2022



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**INCLINOMETER PROFILE CHANGE,
BORING SW-01**

March 2023

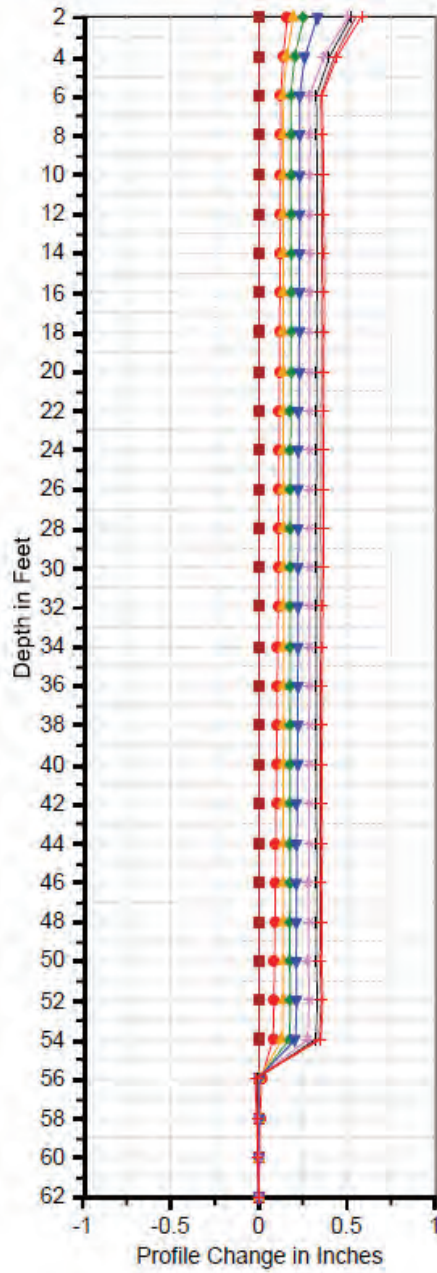
107877-001

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FIG. C-1

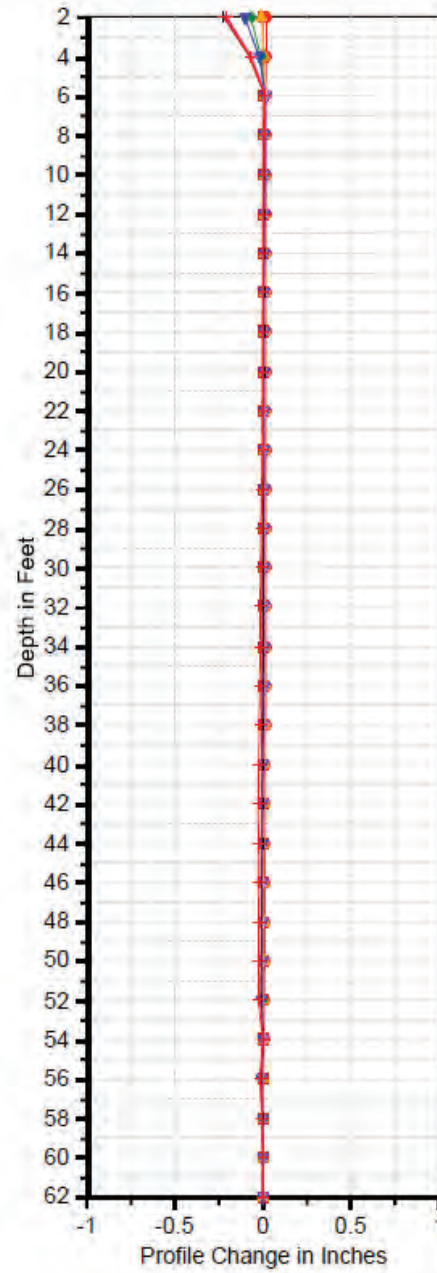
VCBS-SW-02 A

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■ 7/12/2022 ▼ 7/19/2022 ■ 8/18/2022
■ 9/27/2022 ■ 11/3/2022 ■ 12/5/2022



VCBS-SW-02 B

■ 6/22/2022 ● 6/28/2022 ▲ 7/6/2022
■ 7/12/2022 ▼ 7/19/2022 ■ 8/18/2022
■ 9/27/2022 ■ 11/3/2022 ■ 12/5/2022



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**INCLINOMETER PROFILE CHANGE,
BORING SW-02**

March 2023

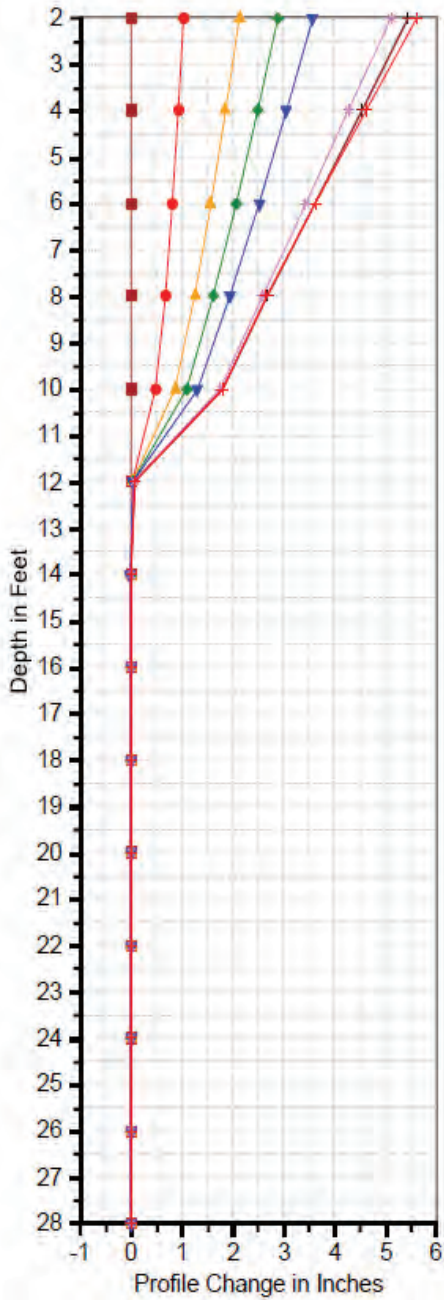
107877-001

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FIG. C-2

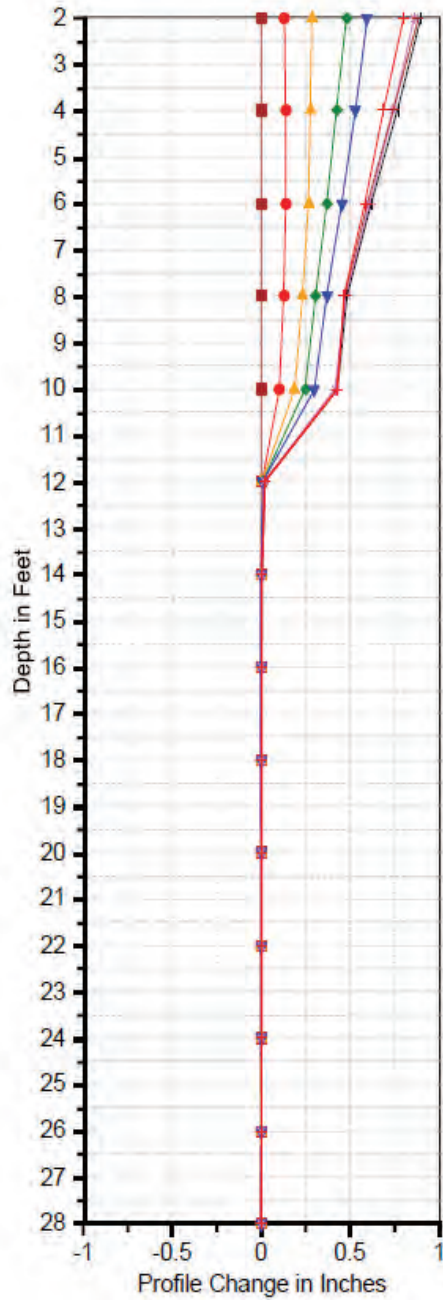
VCBS-SW-03 A

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◆ 7/12/2022 ▼ 7/19/2022 ✦ 8/18/2022
✦ 9/27/2022 ✦ 11/3/2022 ✦ 12/5/2022



VCBS-SW-03 B

■ 6/22/2022 ● 6/28/2022 ▲ 7/6/2022
◆ 7/12/2022 ▼ 7/19/2022 ✦ 8/18/2022
✦ 9/27/2022 ✦ 11/3/2022 ✦ 12/5/2022



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 Valley City, North Dakota

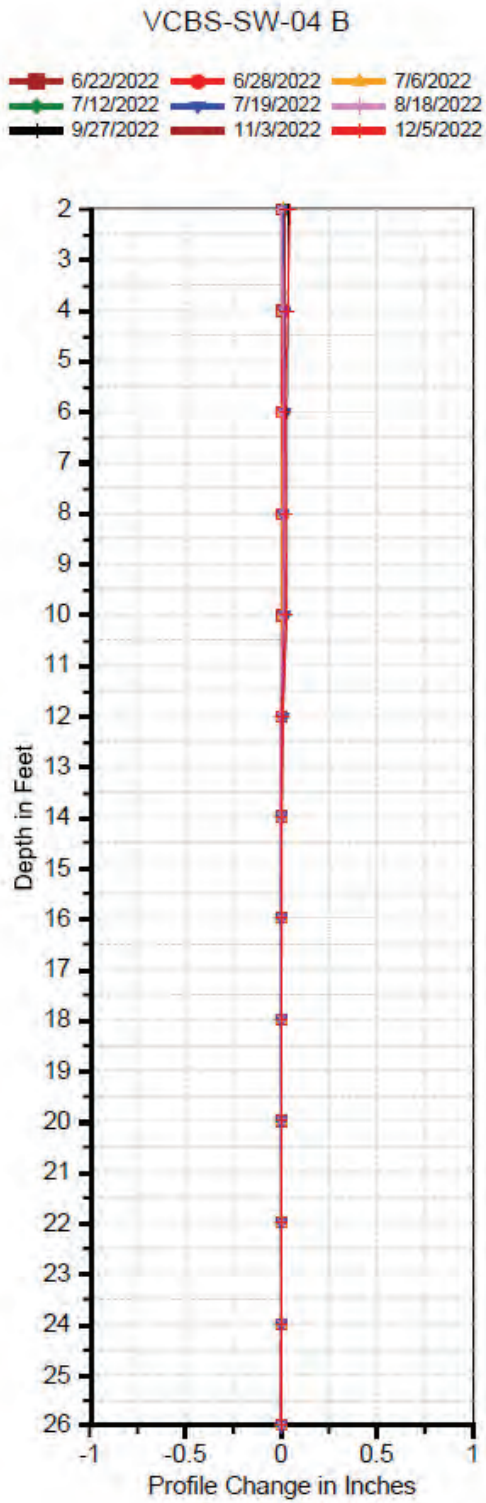
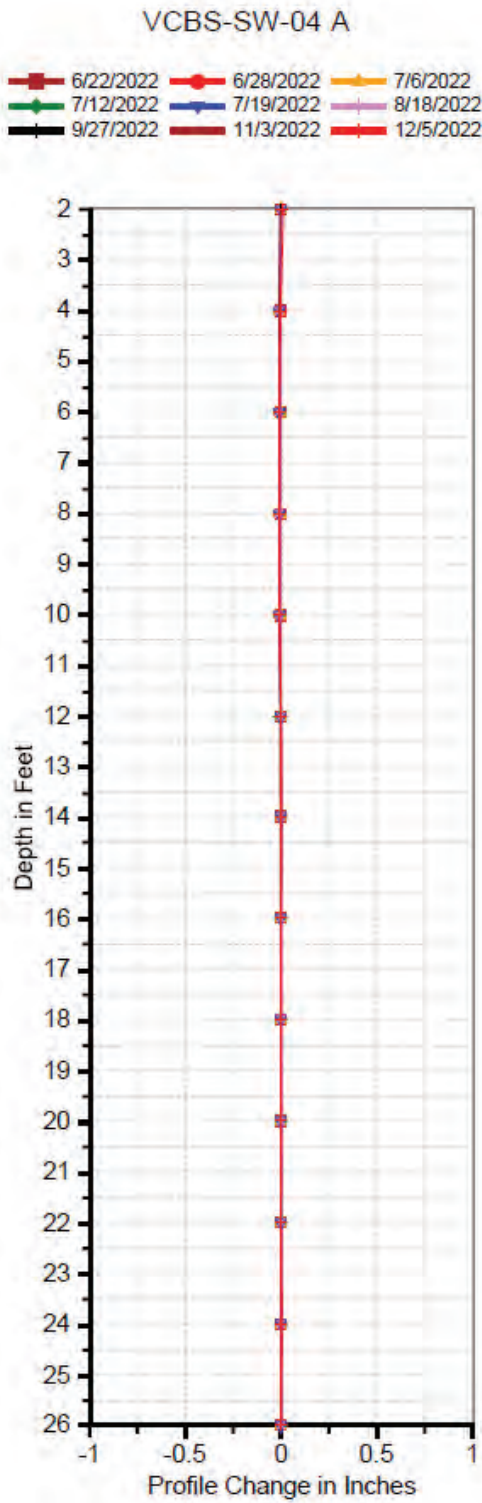
**INCLINOMETER PROFILE CHANGE,
 BORING SW-03**

March 2023

107877-001

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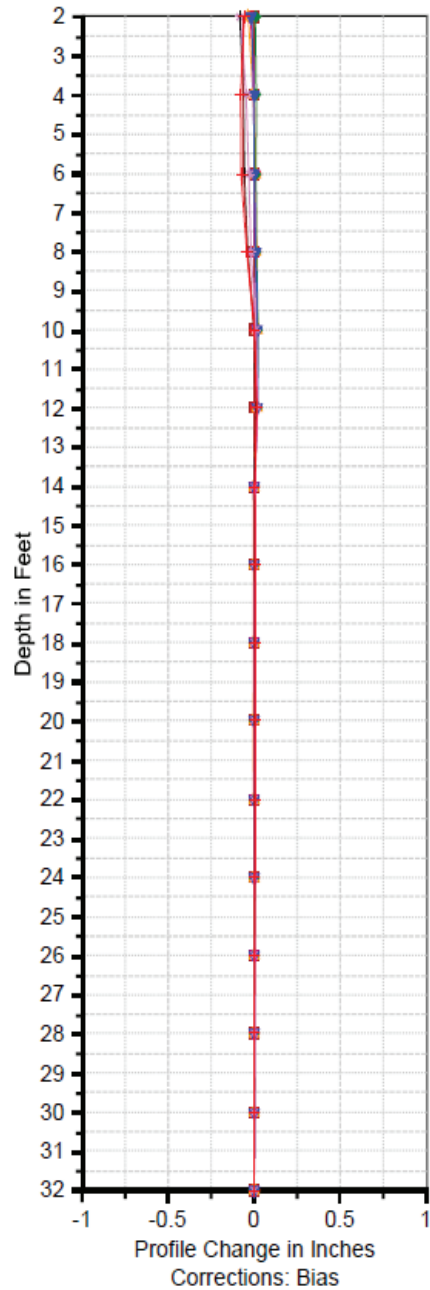
FIG. C-3



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
INCLINOMETER PROFILE CHANGE, BORING SW-04	
March 2023	107877-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. C-4

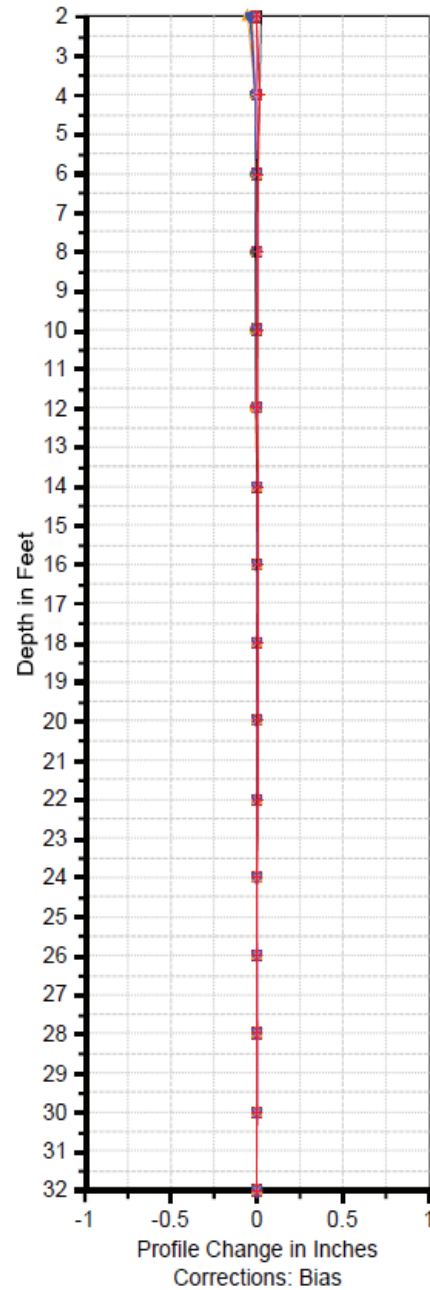
VCBS-SW-06 A

■ 6/22/2022 ● 6/28/2022 ▲ 7/6/2022
◆ 7/12/2022 ▼ 7/19/2022 + 8/18/2022
+ 9/27/2022 ■ 11/3/2022 + 12/5/2022



VCBS-SW-06 B

■ 6/22/2022 ● 6/28/2022 ▲ 7/6/2022
◆ 7/12/2022 ▼ 7/19/2022 + 8/18/2022
+ 9/27/2022 ■ 11/3/2022 + 12/5/2022



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Valley City, North Dakota

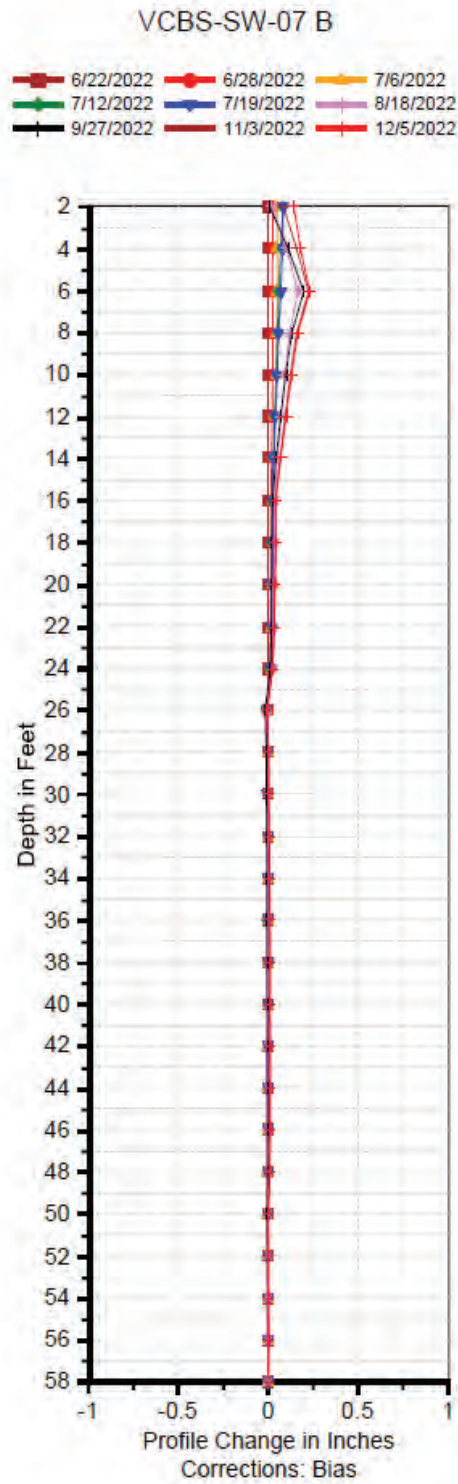
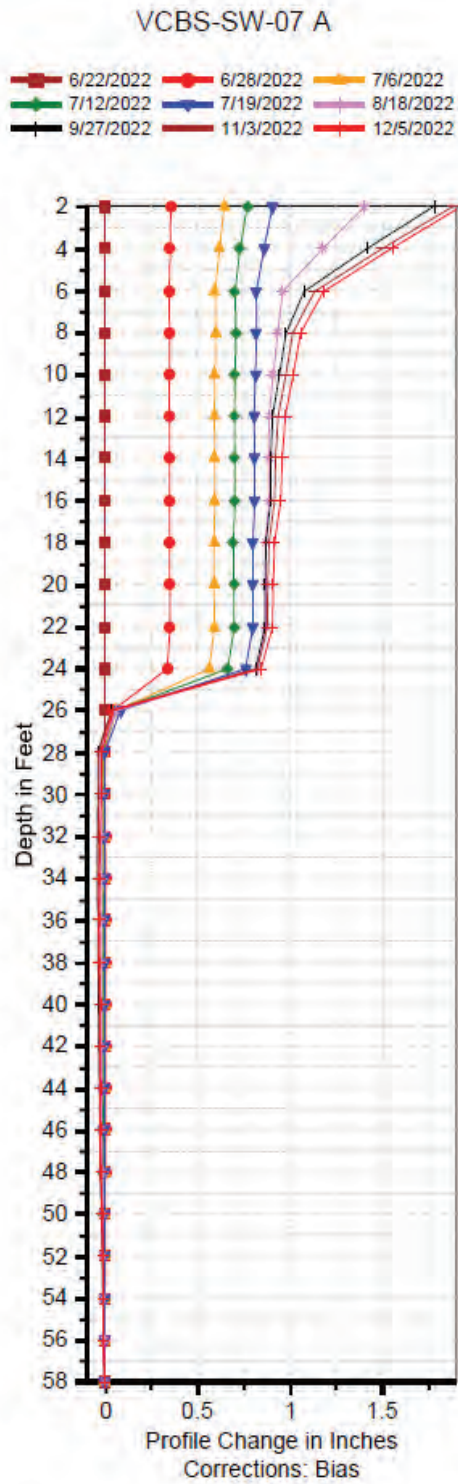
**INCLINOMETER PROFILE CHANGE,
BORING SW-06**

March 2023

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FIG. C-5

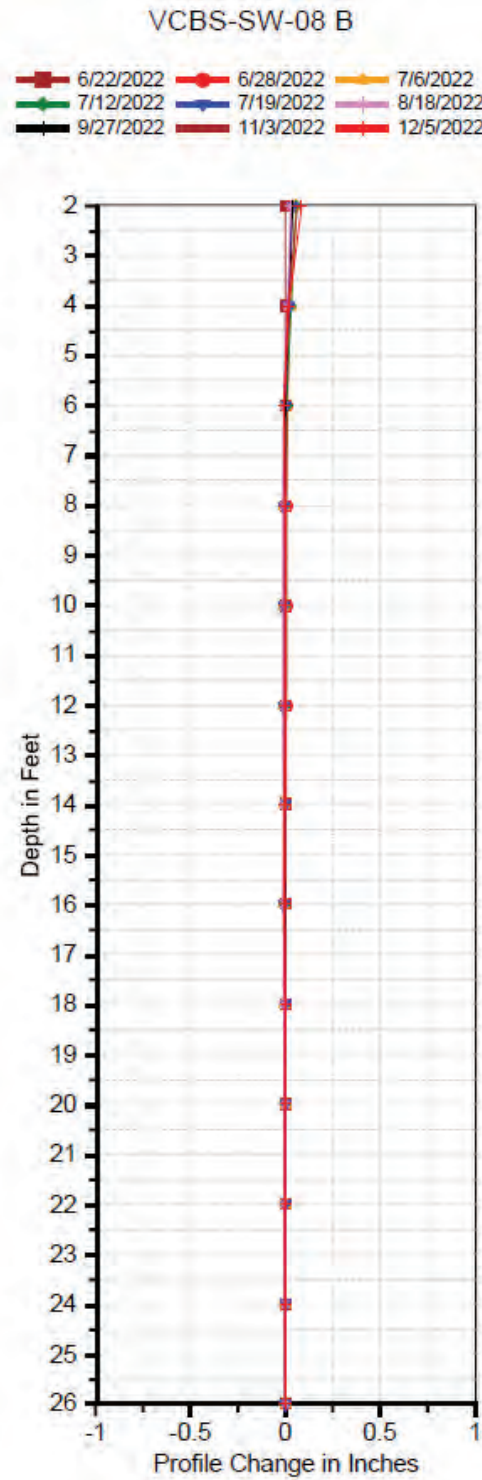
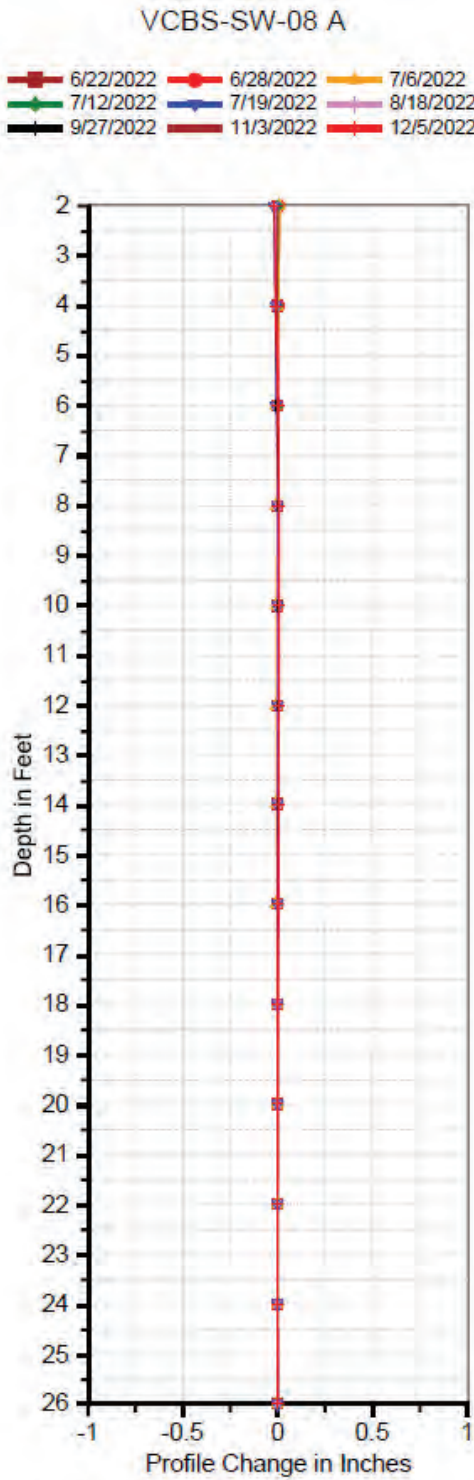


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**INCLINOMETER PROFILE CHANGE,
 BORING SW-07**

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Valley City SW Backslope Slide Repair
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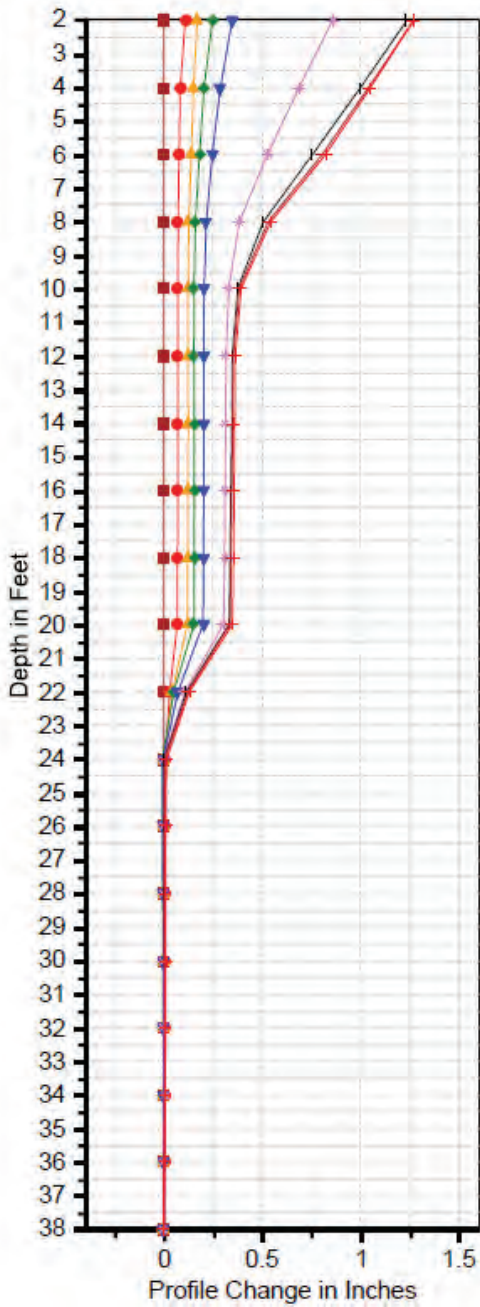
**INCLINOMETER PROFILE CHANGE,
 BORING SW-08**

March 2023
107877-001

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FIG. C-7

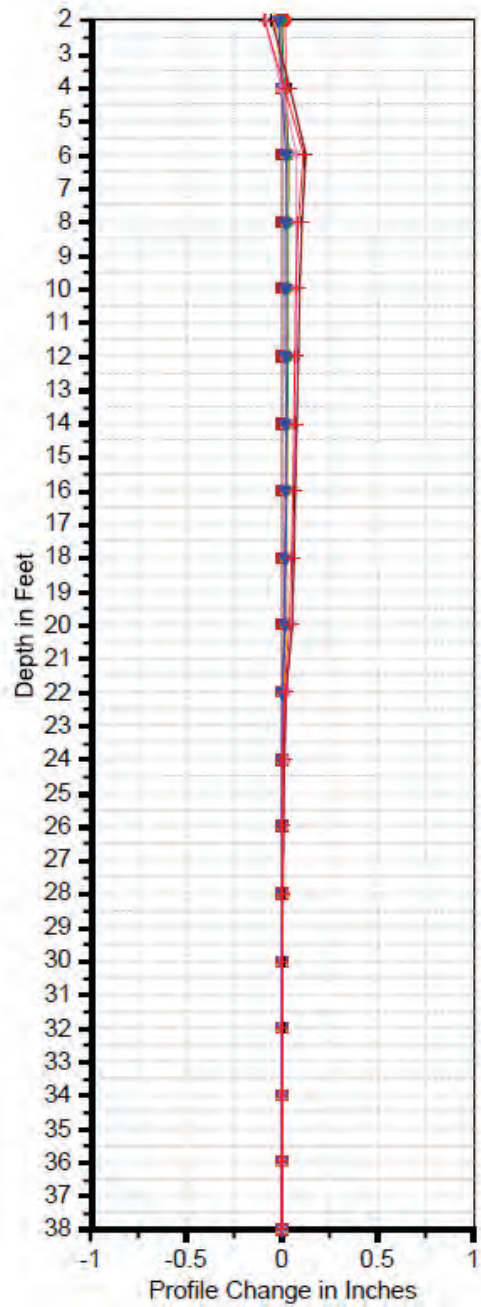
VCBS-SW-09 A

6/22/2022 6/28/2022 7/6/2022
 7/12/2022 7/19/2022 8/18/2022
 9/27/2022 11/3/2022 12/5/2022



VCBS-SW-09 B

6/22/2022 6/28/2022 7/6/2022
 7/12/2022 7/19/2022 8/18/2022
 9/27/2022 11/3/2022 12/5/2022



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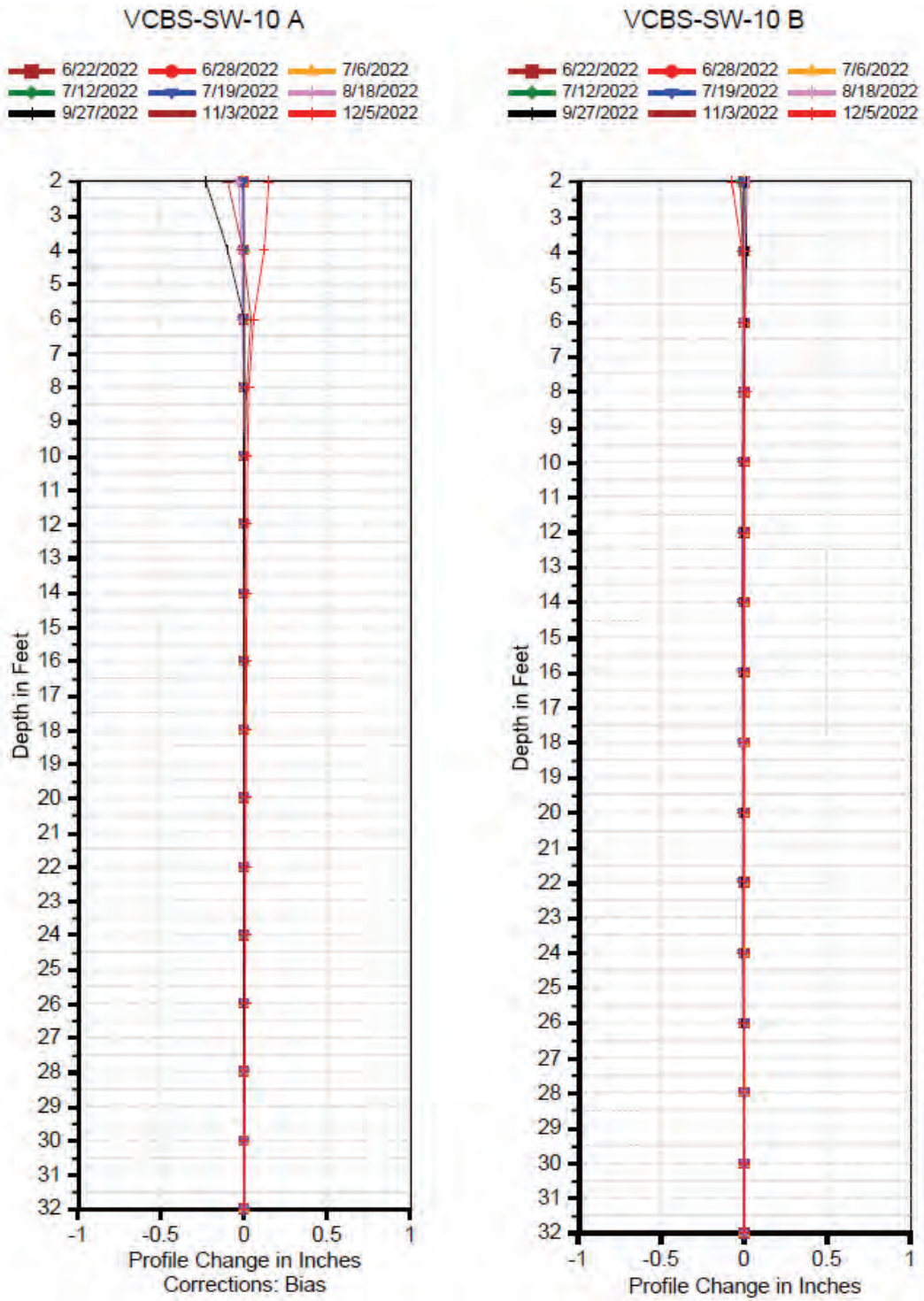
**INCLINOMETER PROFILE CHANGE,
 BORING SW-09**

March 2023

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FIG. C-8



Valley City SW Backslope Slide Repair
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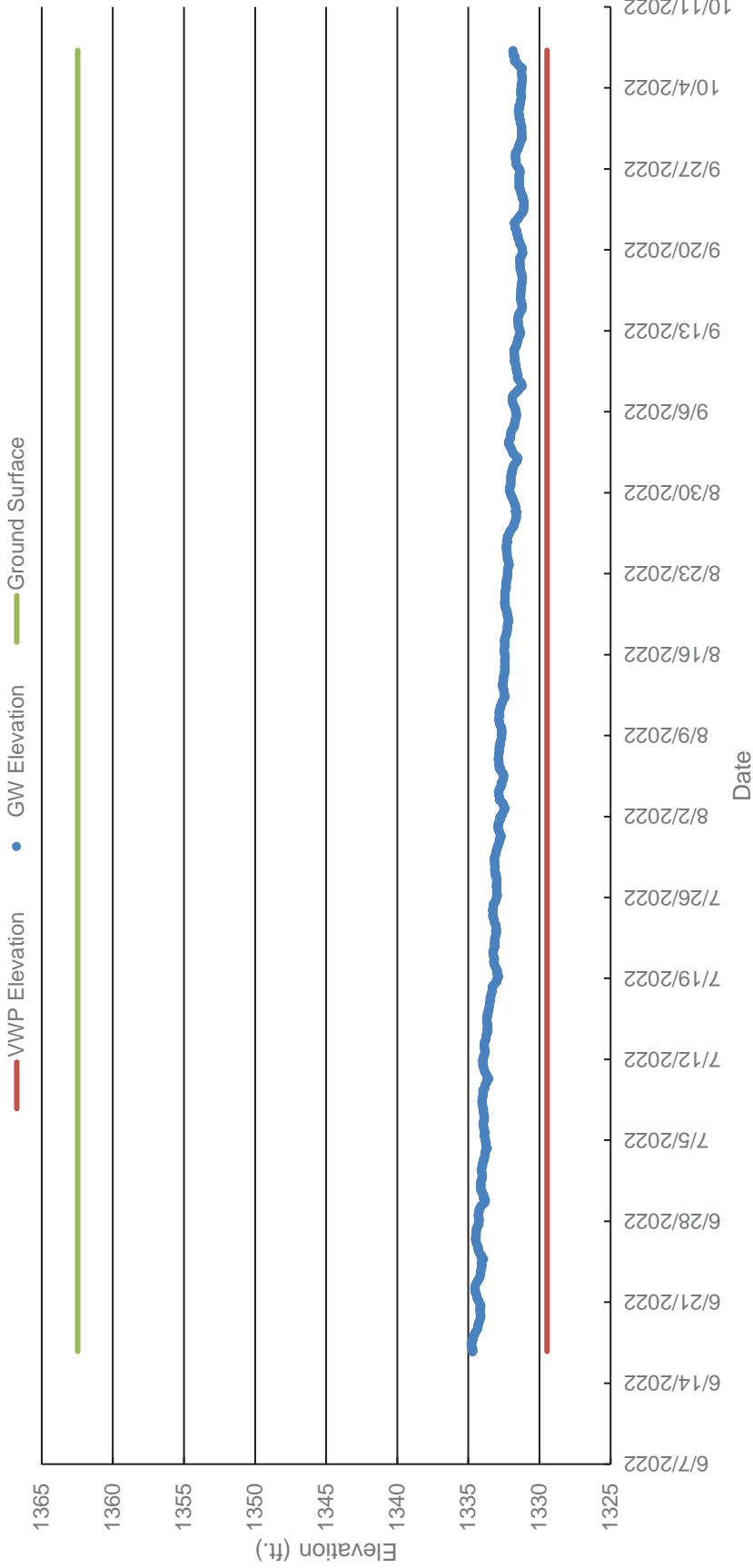
**INCLINOMETER PROFILE CHANGE,
 BORING SW-10**

March 2023 107877-001

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FIG. C-9

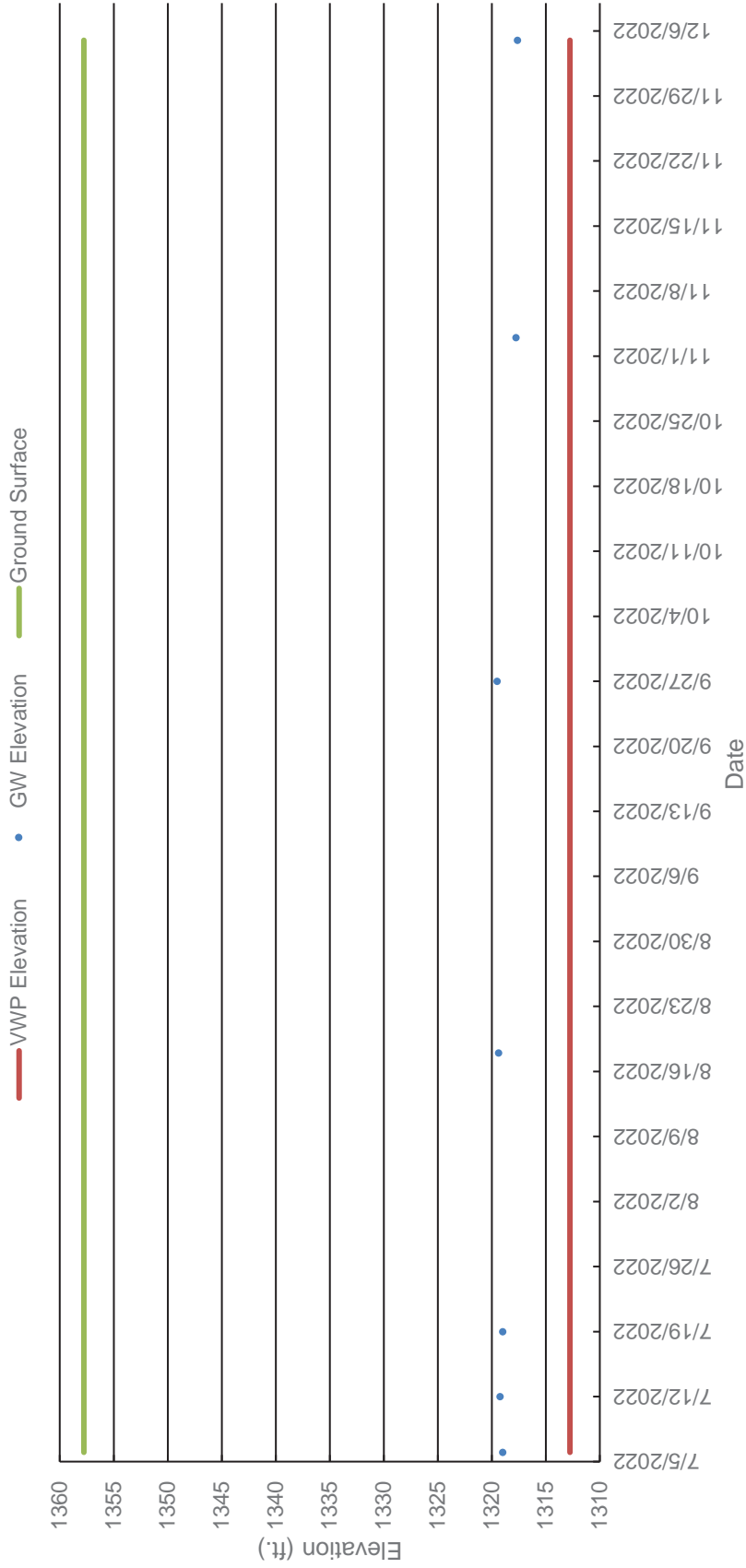
SW-01 VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-01 VIBRATING WIRE PIEZOMETER	
March 2023	107877-001
SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-10

FIG. C-10

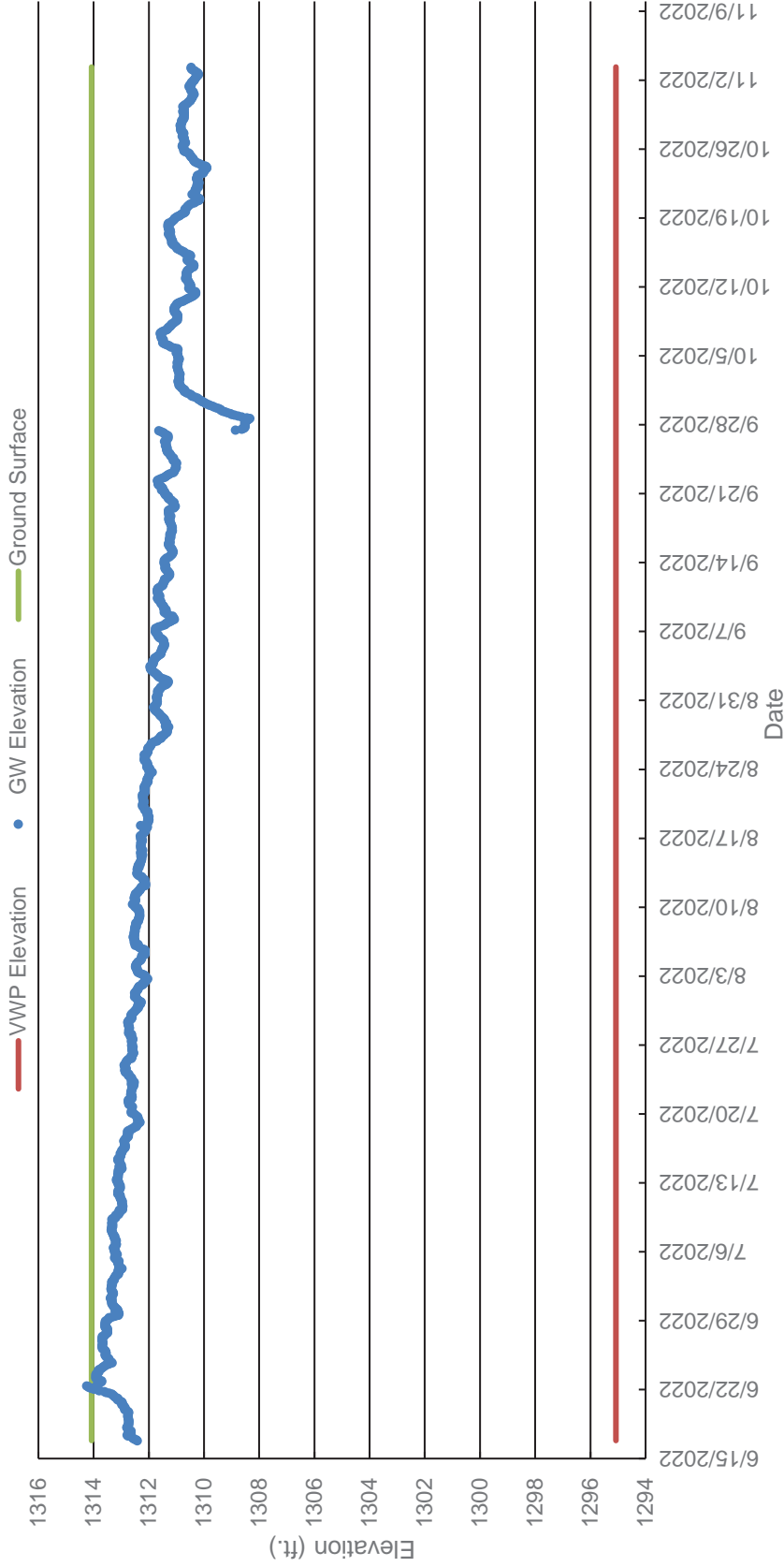
SW-02 VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-02 VIBRATING WIRE PIEZOMETER	
March 2023	107877-001
SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-11

FIG. C-11

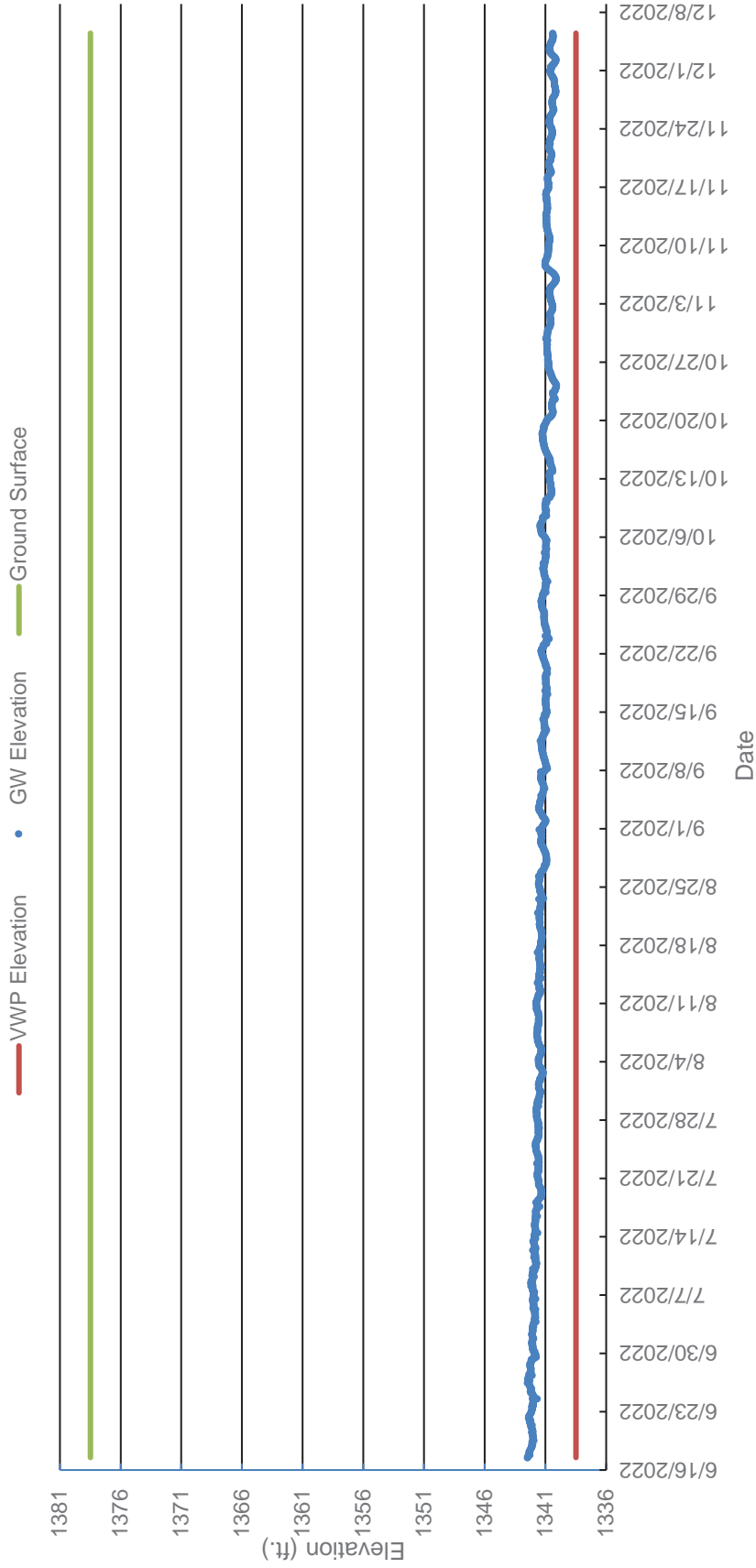
SW-03 VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-03 VIBRATING WIRE PIEZOMETER	
March 2023	107877-001
SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-12

FIG. C-12

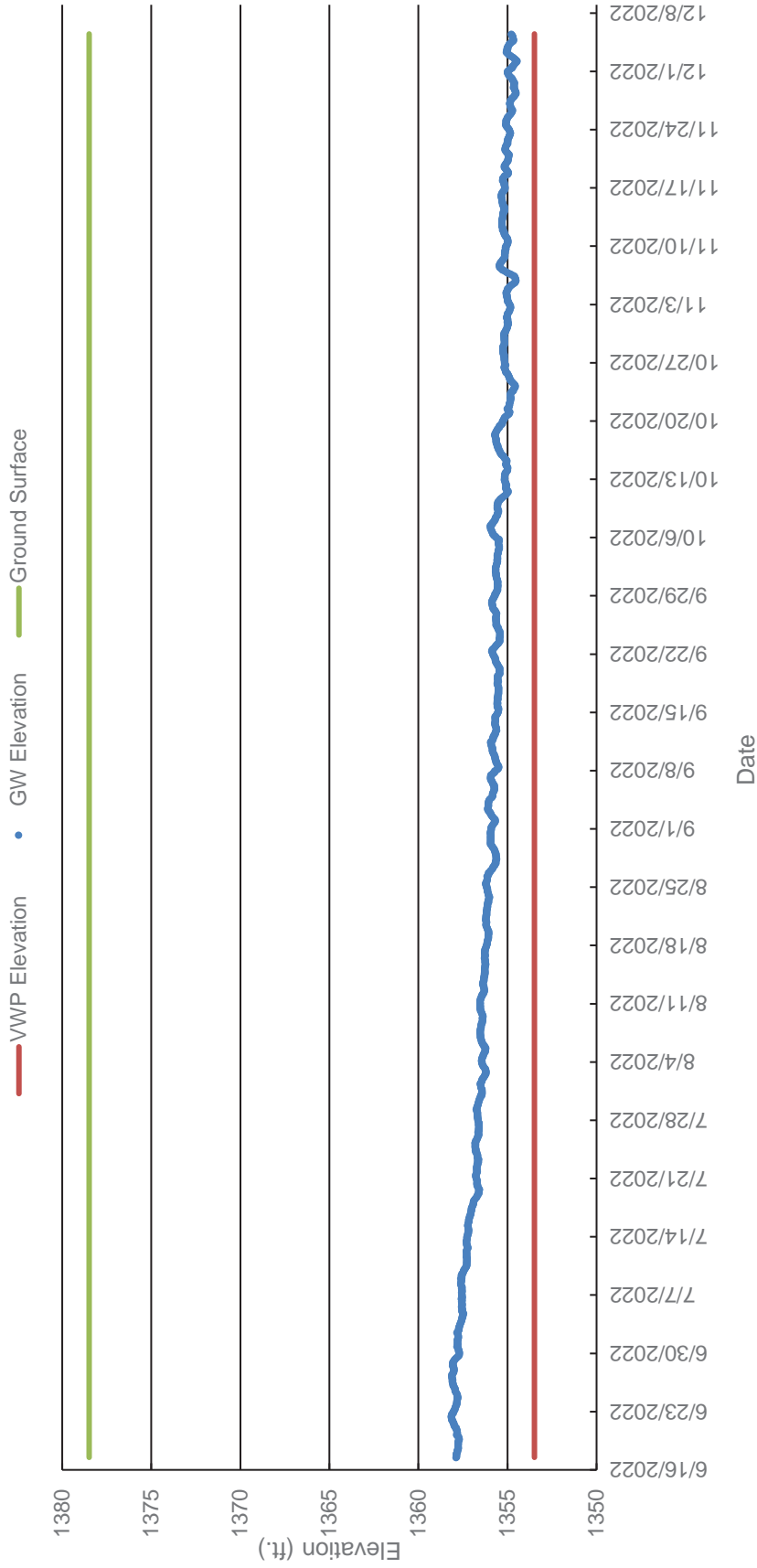
SW-05 (40') VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-05 (40') VIBRATING WIRE PIEZOMETER	
March 2023	107877-001
SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-13

FIG. C-13

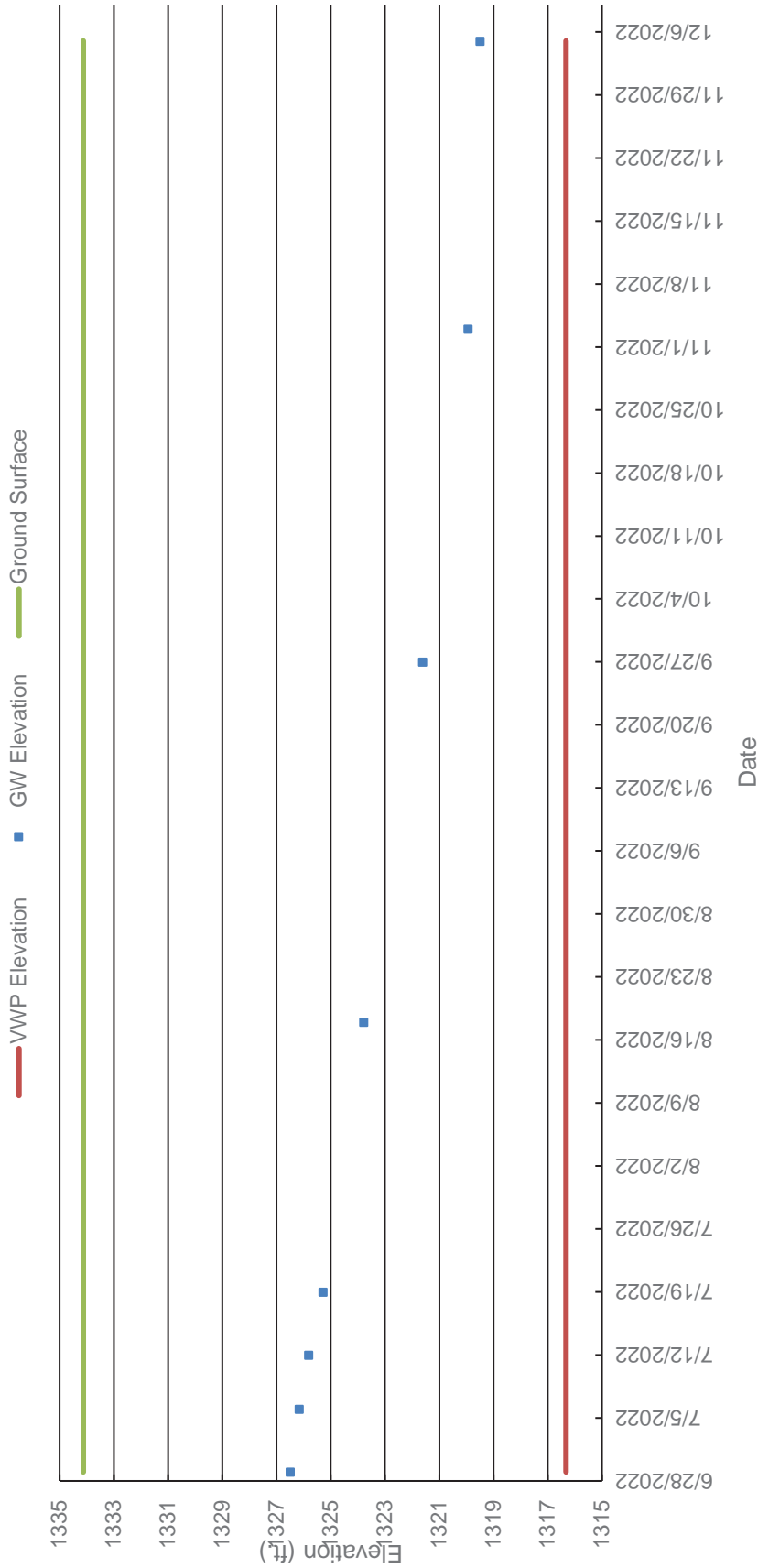
SW-05 (25') VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-05 (25') VIBRATING WIRE PIEZOMETER	
March 2023	107877-001
SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-14

FIG. C-14

SW-07 VWP GW



Valley City SW Backslope Slide Repair
 2-094(185)290, PCN 23338
 Valley City, North Dakota

GROUNDWATER MEASUREMENTS
SW-07 VIBRATING WIRE PIEZOMETER

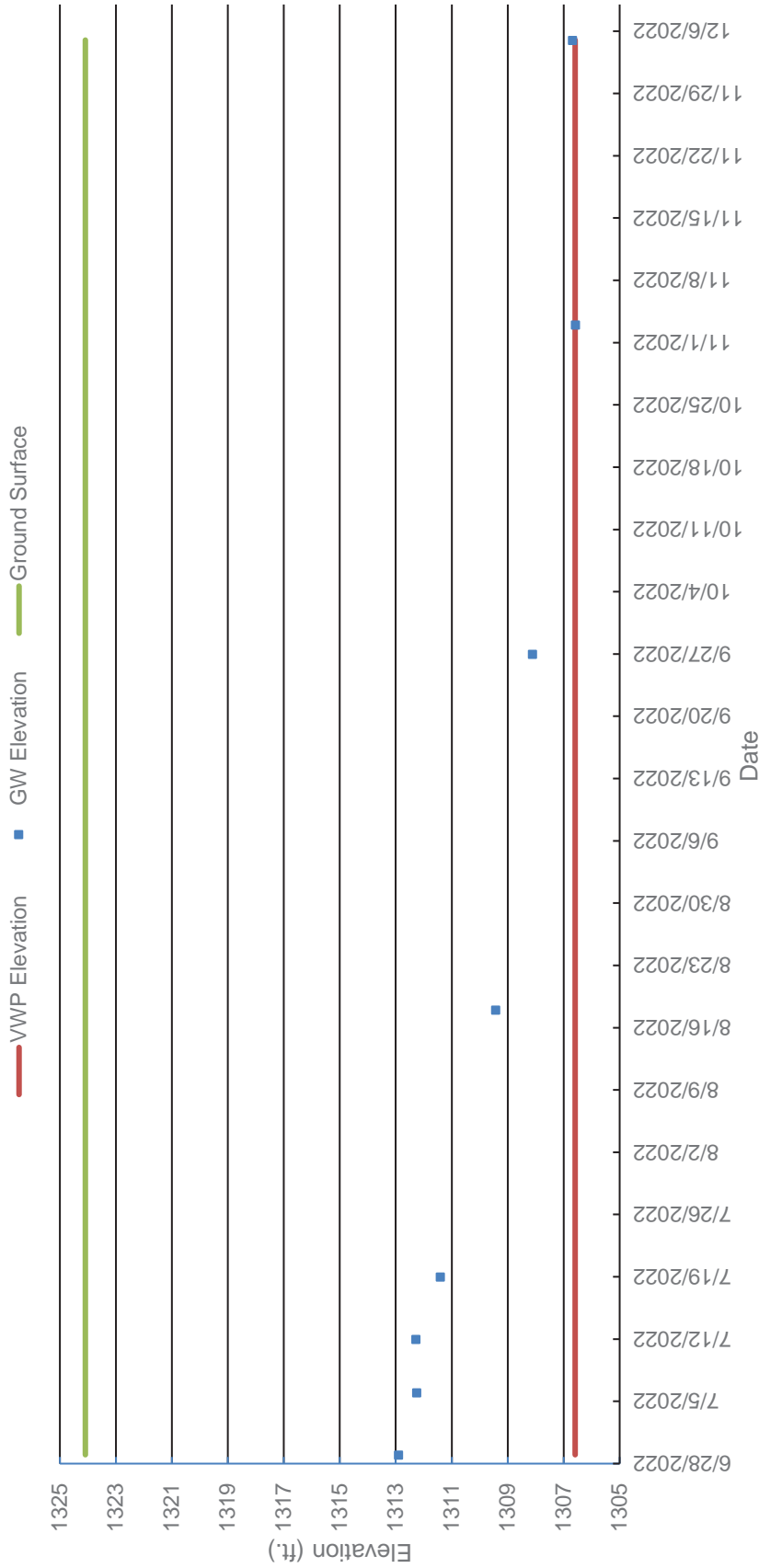
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FIG. C-15

FIG. C-15

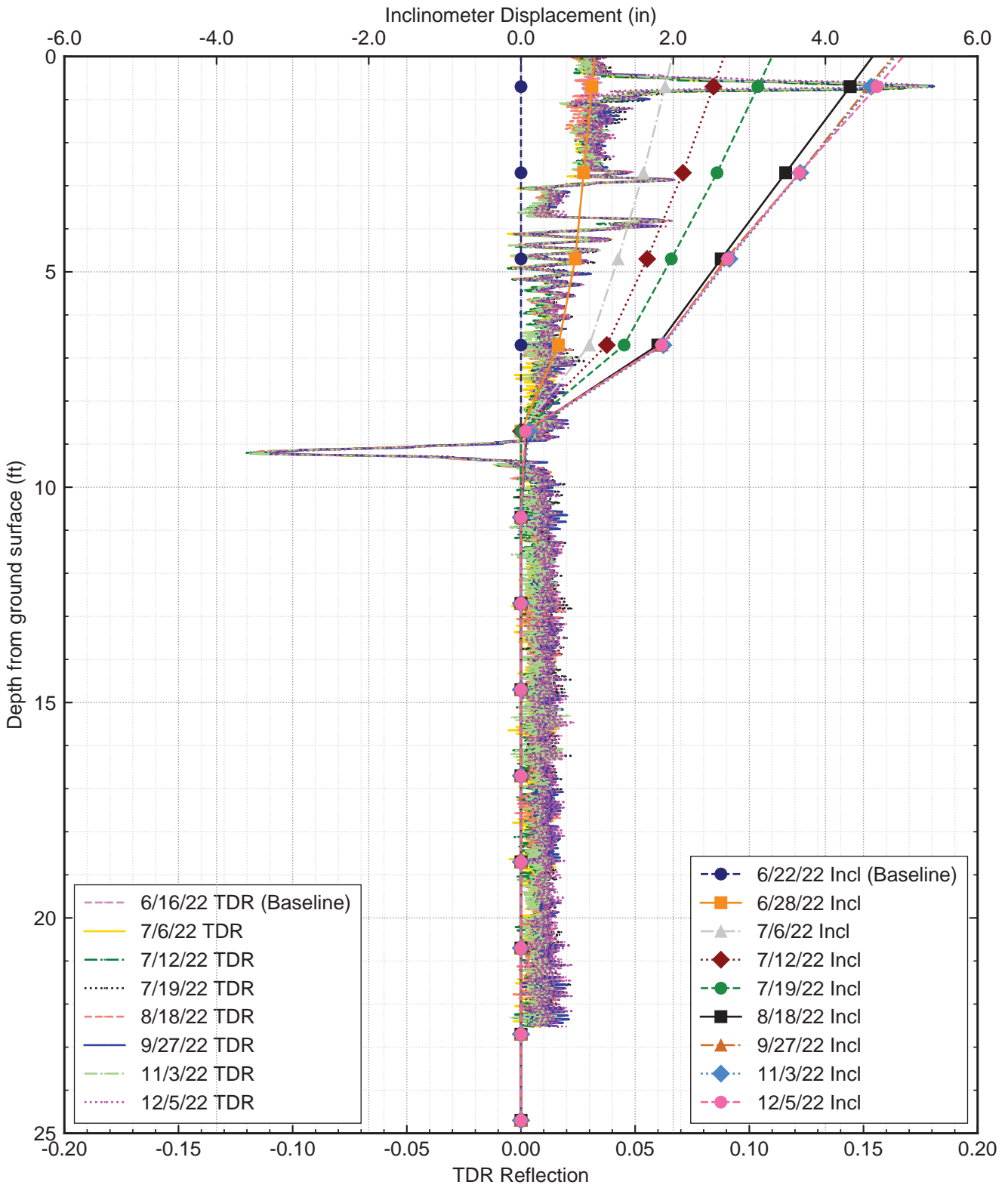
SW-09 VWP GW



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
GROUNDWATER MEASUREMENTS SW-09 VIBRATING WIRE PIEZOMETER	
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SHANNON & WILSON, Geotechnical and Environmental Consultants	FIG. C-16

FIG. C-16

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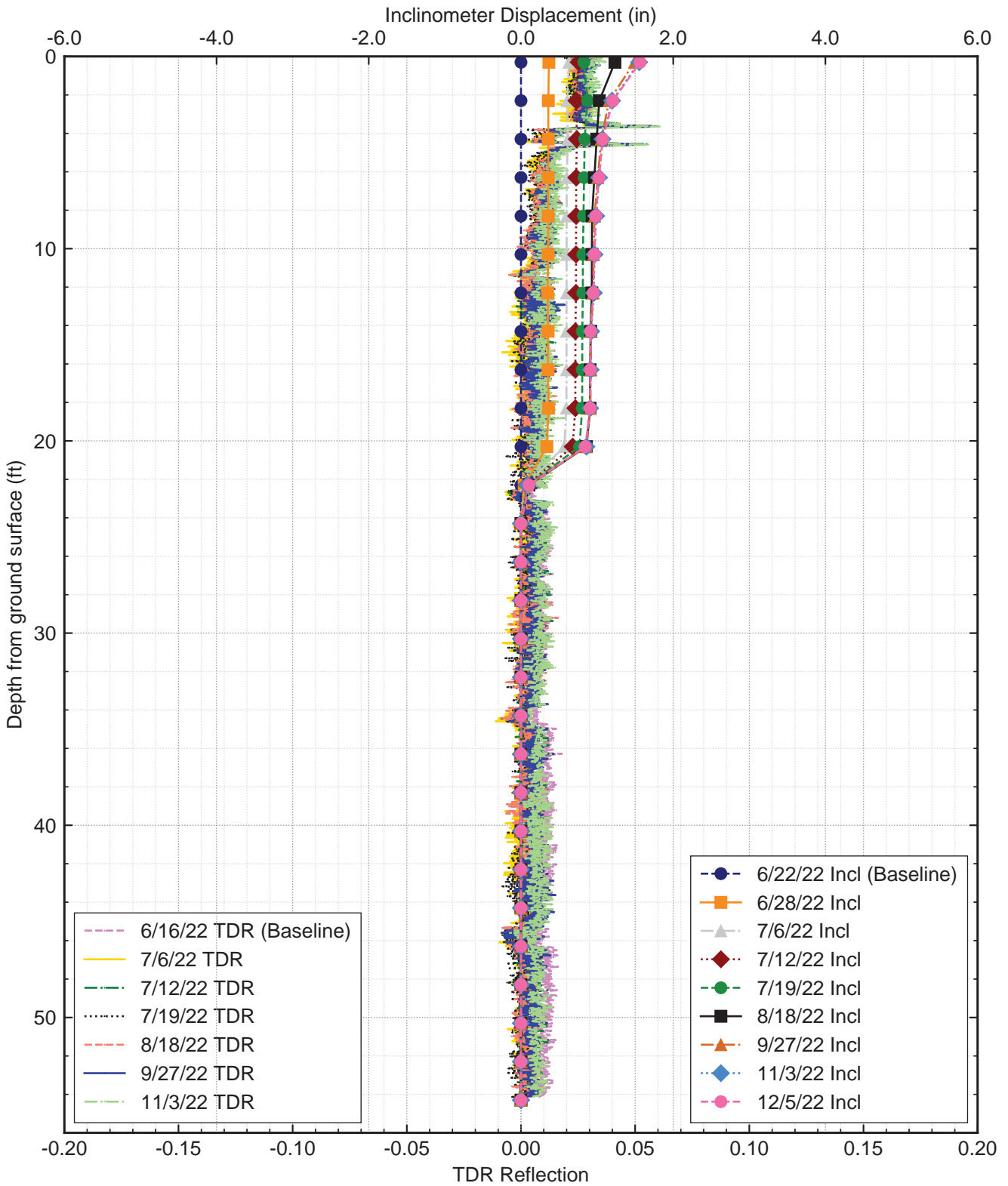
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**TDR-INCLINOMETER
COMPARISON,
BORING SW-03**

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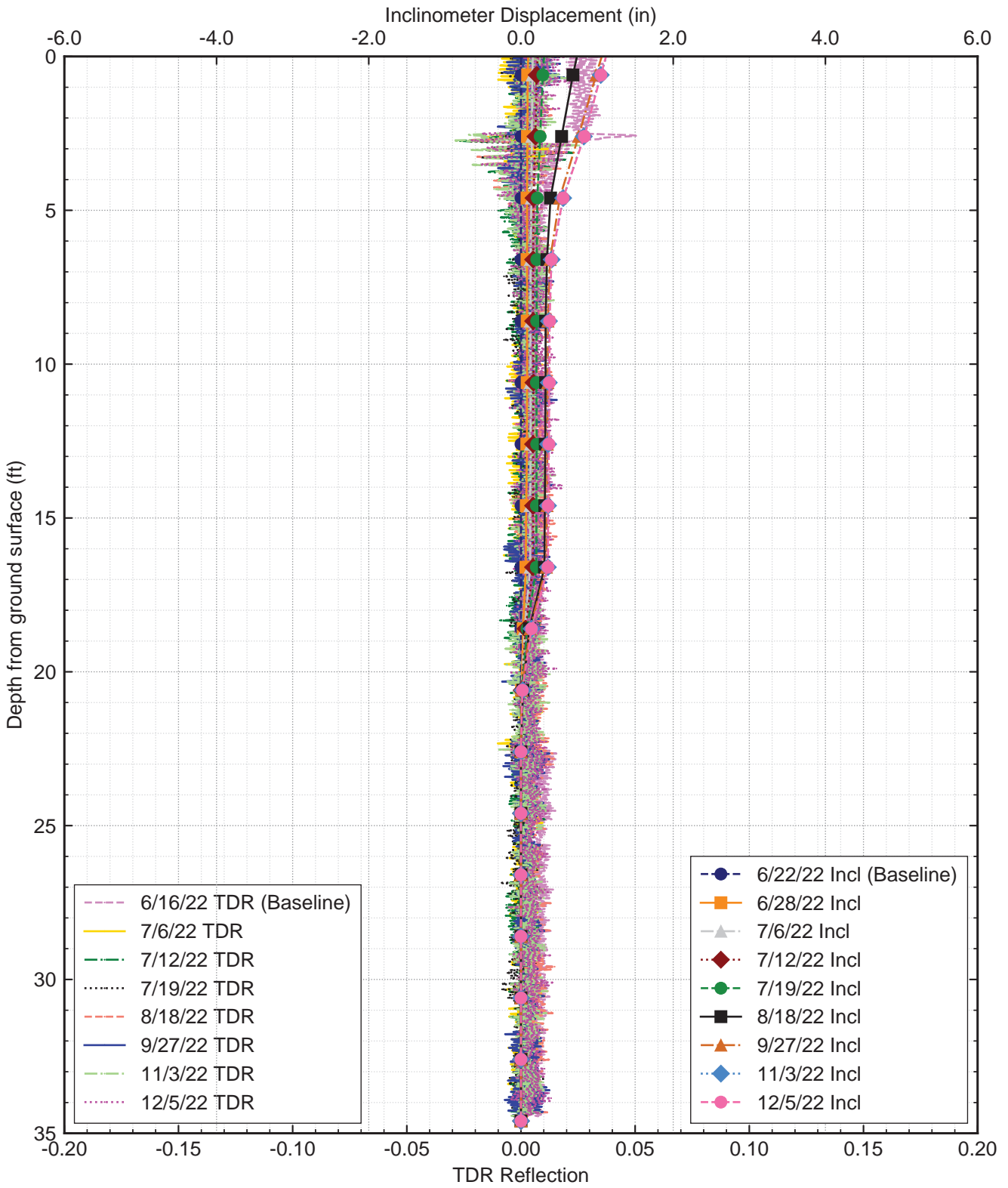
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FIG. C-17



Valley City SW Backslope Slide Repair 2-094(185)290, PCN 23338 Valley City, North Dakota	
TDR-INCLINOMETER COMPARISON, BORING SW-07	
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SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. C-18

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**TDR-INCLINOMETER
COMPARISON,
BORING SW-09**

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Appendix D

Slope Stability Analyses

Figures

Section A-A'

Figure D-1:	Back-Analysis
Figure D-2:	Grading (Global)
Figure D-3:	Grading (Intermediate)
Figure D-4:	Ground Anchors (Global)
Figure D-5:	Anchored Shafts (Global)

Section B-B'

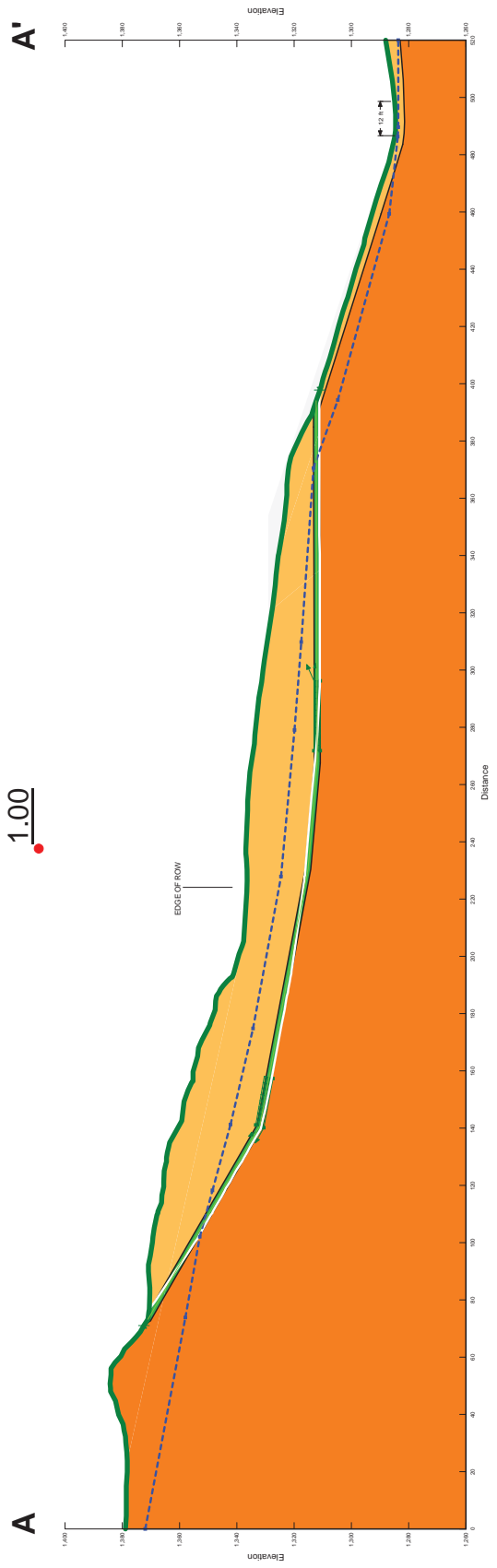
Figure D-6:	Back-Analysis
Figure D-7:	Grading (Global)
Figure D-8:	Grading (Intermediate)
Figure D-9:	Grading (Downslope)
Figure D-10:	Ground Anchors (Global)
Figure D-11:	Anchored Shafts (Global)

Section C-C'

Figure D-12:	Back-Analysis
Figure D-13:	Grading (Global)
Figure D-14:	Grading (Intermediate)
Figure D-15:	Grading (Downslope)
Figure D-16:	Ground Anchors (Global)
Figure D-17:	Anchored Shafts (Global)

Section D-D'

Figure D-18:	Back-Analysis
Figure D-19:	Grading (Global)
Figure D-20:	Grading (Intermediate)
Figure D-21:	Grading (Downslope)
Figure D-22:	Ground Anchors (Global)
Figure D-23:	Anchored Shafts (Global)



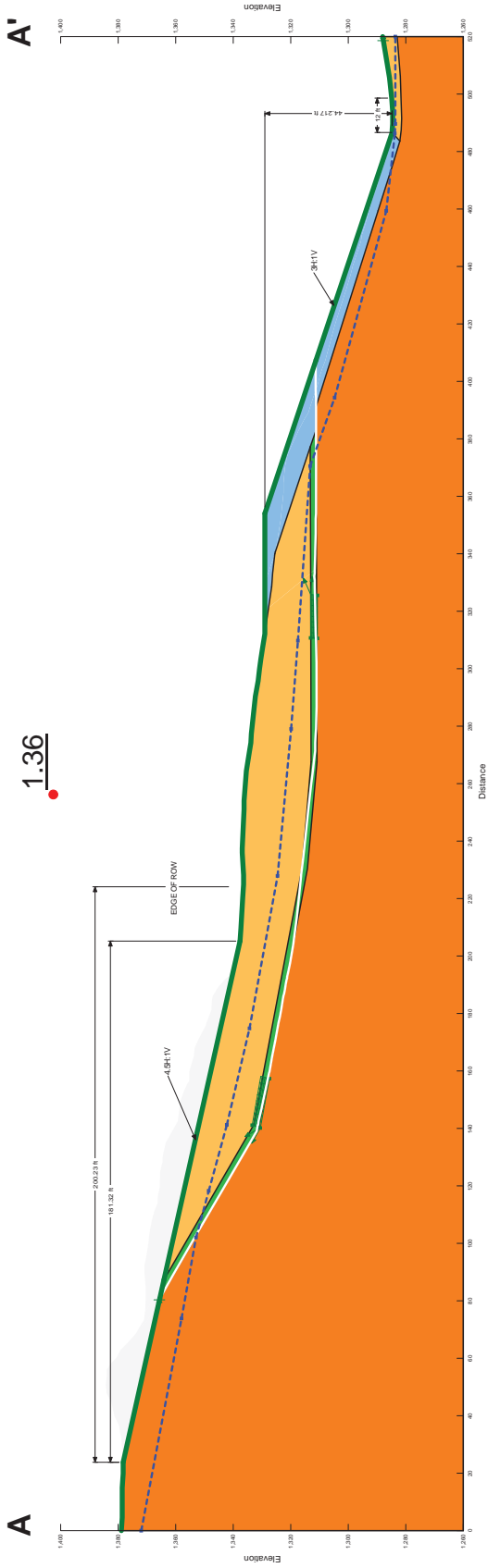
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C'; LL=80, CF>50		

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SECTION A-A'
BACK-ANALYSIS

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FIG. D-1



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Butress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50		

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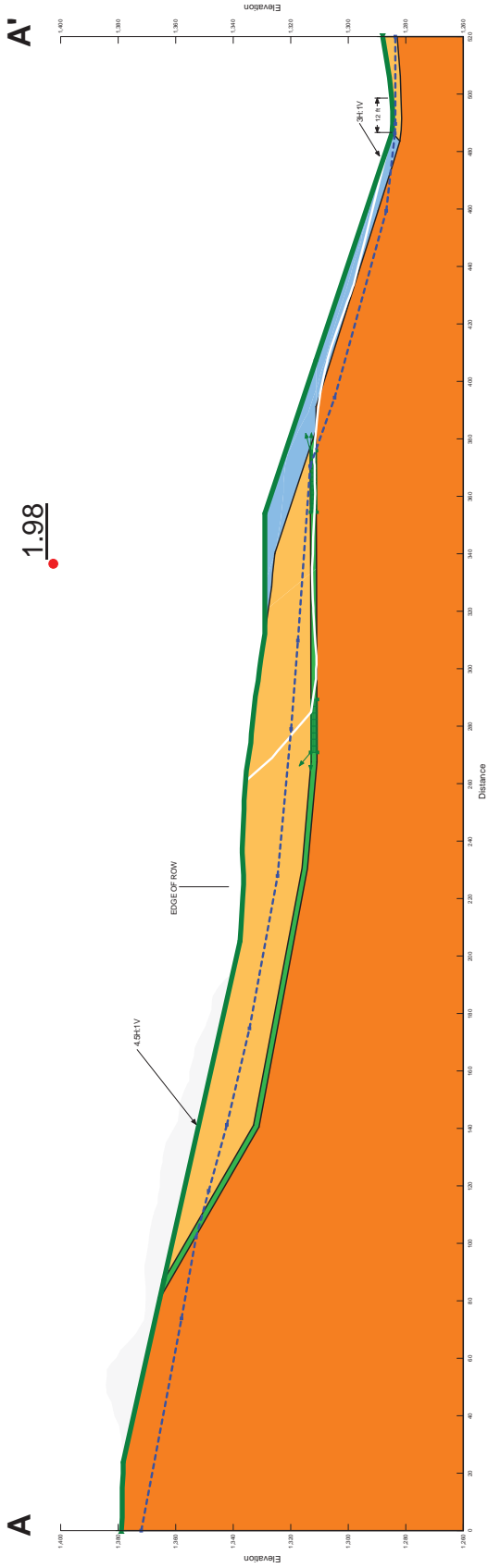
SECTION A-A'
GRADING (GLOBAL)

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FIG. D-2



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Butress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50		

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 2-094(185)290, PCN 23338
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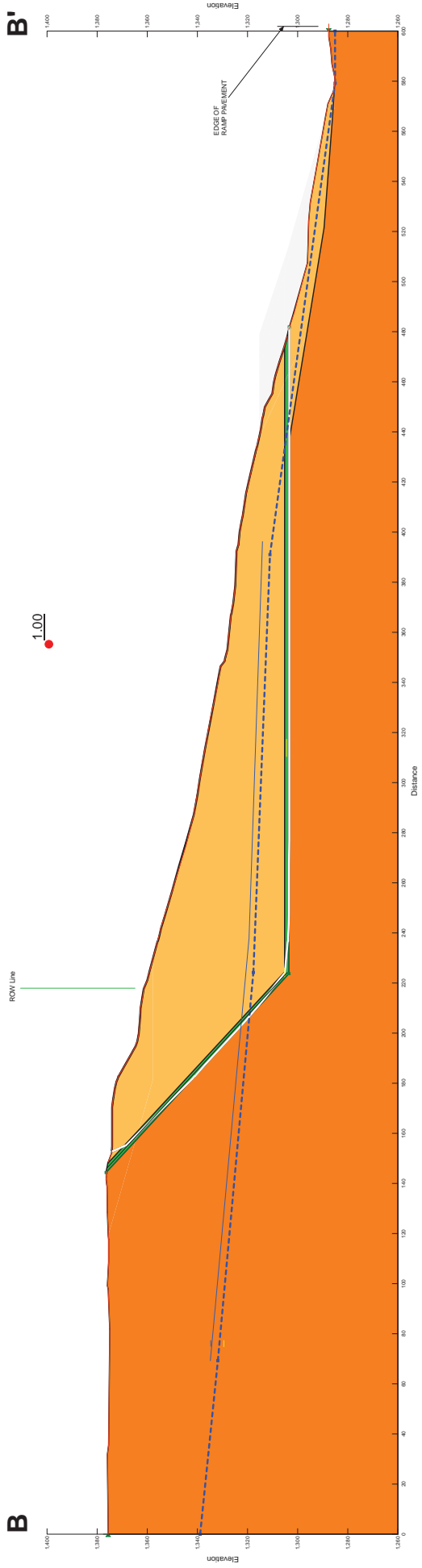
SECTION A-A'
GRADING (INTERMEDIATE)

March 2023

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FIG. D-3



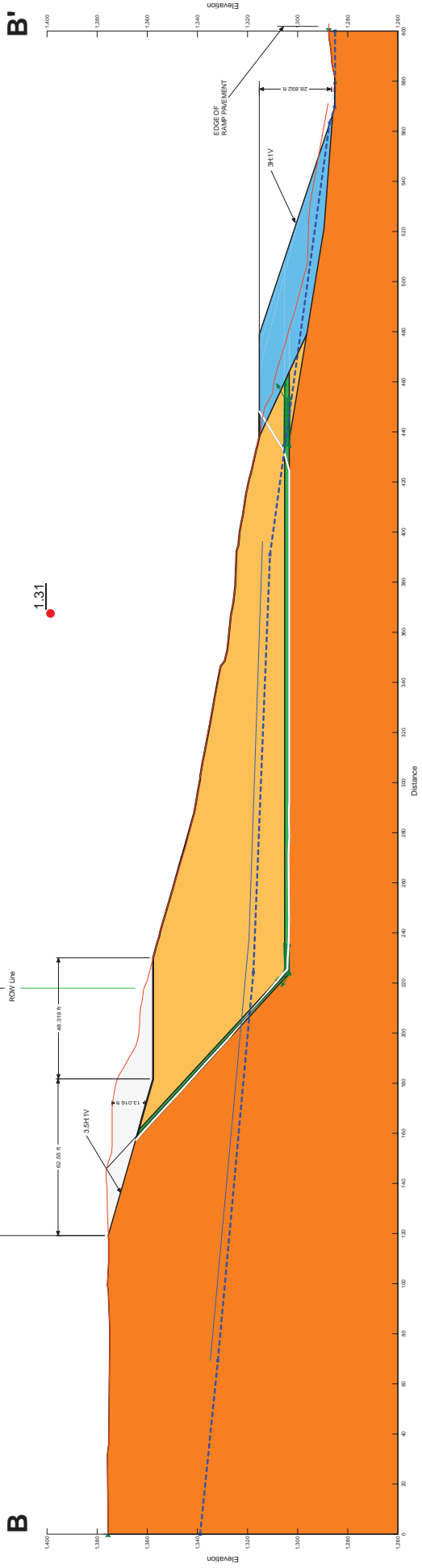
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

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 Valley City, North Dakota

SECTION B-B'
BACK-ANALYSIS

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FIG. D-6



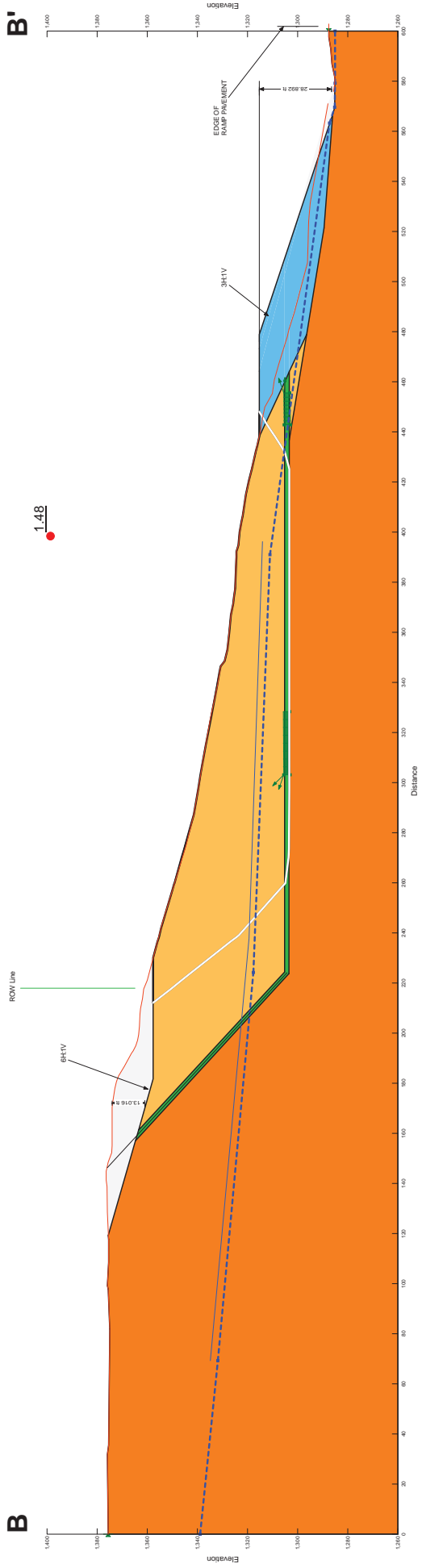
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

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SECTION B-B'
GRADING (GLOBAL)

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FIG. D-7



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

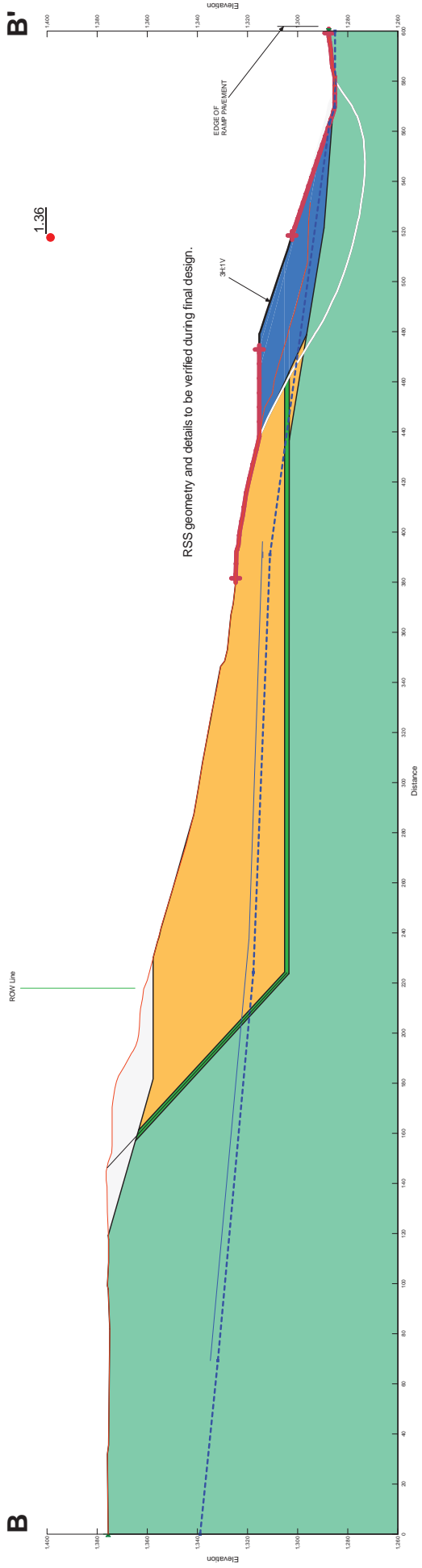
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SECTION B-B'
GRADING (INTERMEDIATE)

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FIG. D-8



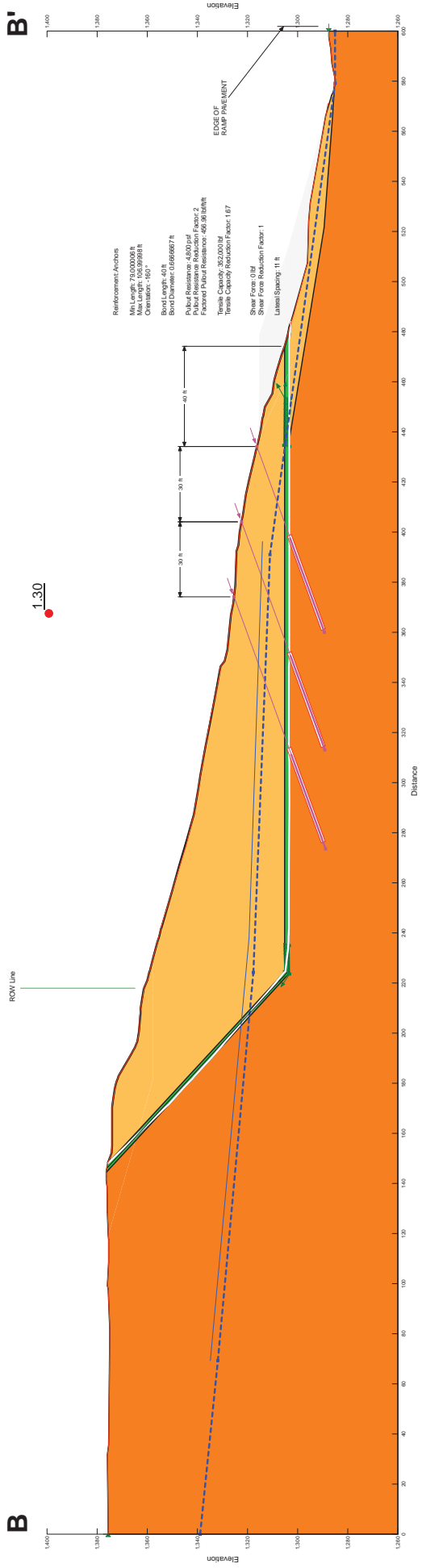
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)	C-Horizontal (psf)	C-Vertical (psf)	Phi-Horizontal (°)	Phi-Vertical (°)
Light Green	Bedrock (Anisotropic)	Anisotropic Strength	125				70	100	22	24
Orange	Overburden	Mohr-Coulomb	120		0	24				
Blue	Reinforced Butress Fill (RSS)	Mohr-Coulomb	125		1,500	28				
Dark Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50						

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SECTION B-B'
GRADING (DOWNSLOPE)

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FIG. D-9



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

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2-094 (165)290, PCN 23338
Valley City, North Dakota

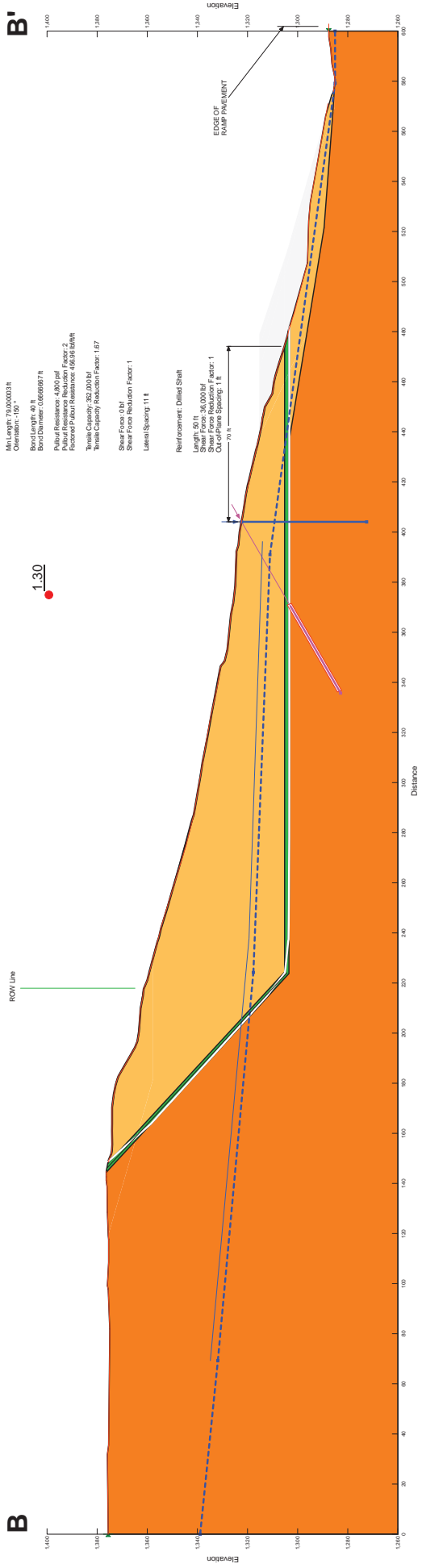
SECTION B-B'

GROUND ANCHORS (GLOBAL)

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FIG. D-10

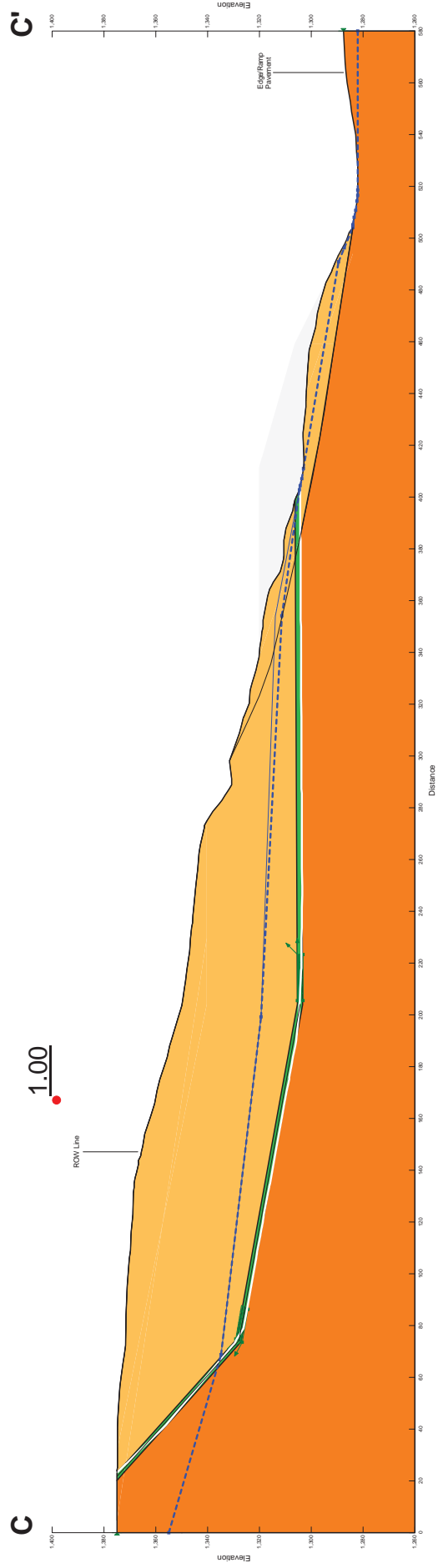


Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surface	Shear/Normal Fn.	120	Interp_LL=75,CF>50		

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SECTION B-B'
ANCHORED SHAFTS (GLOBAL)
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FIG. D-11



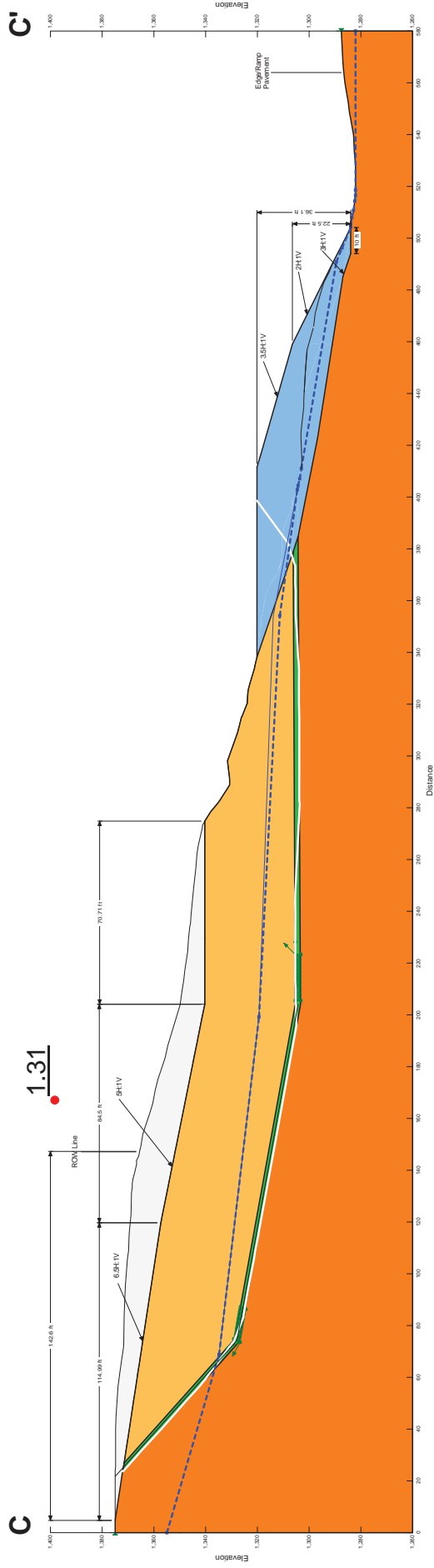
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C'; LL=80, CF>50		

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SECTION C-C'
BACK-ANALYSIS

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FIG. D-12



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50		

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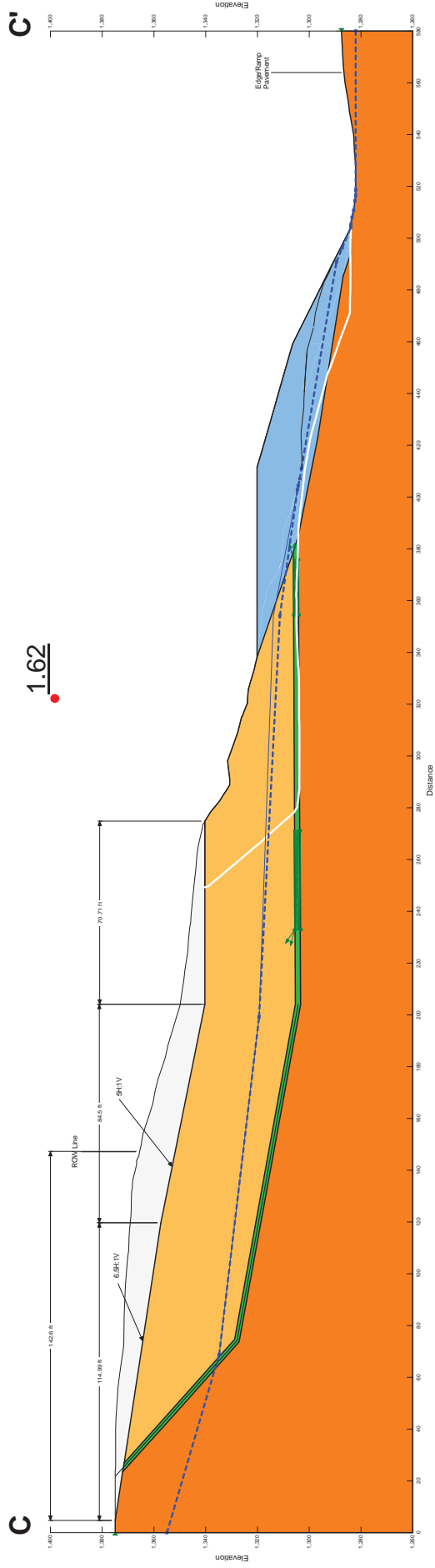
SECTION C-C'
GRADING (GLOBAL)

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FIG. D-13



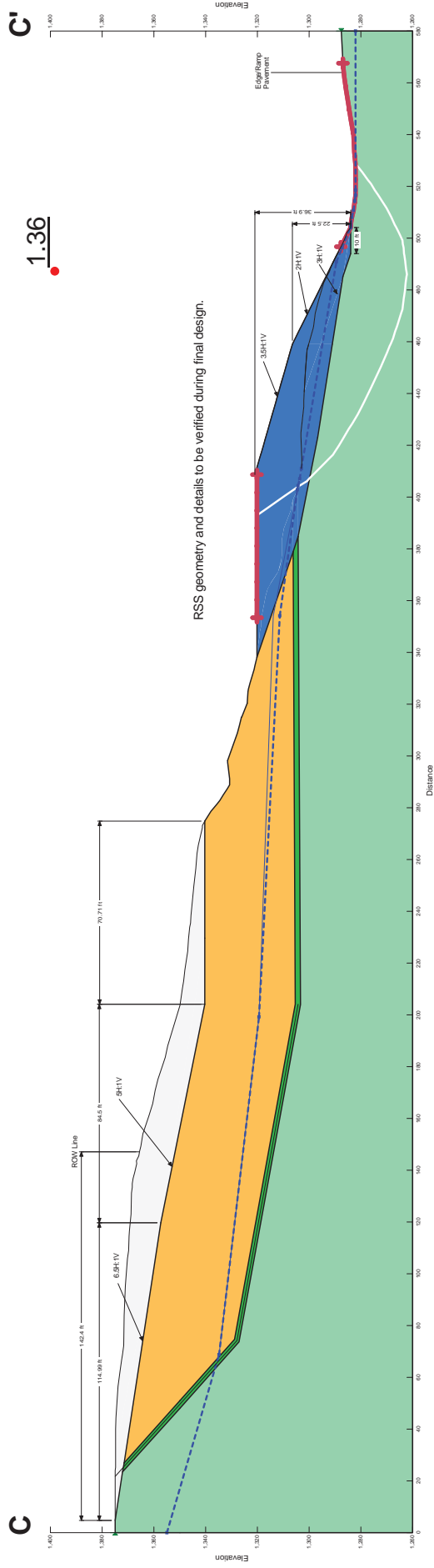
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50		

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SECTION C-C'
GRADING (INTERMEDIATE)

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FIG. D-14



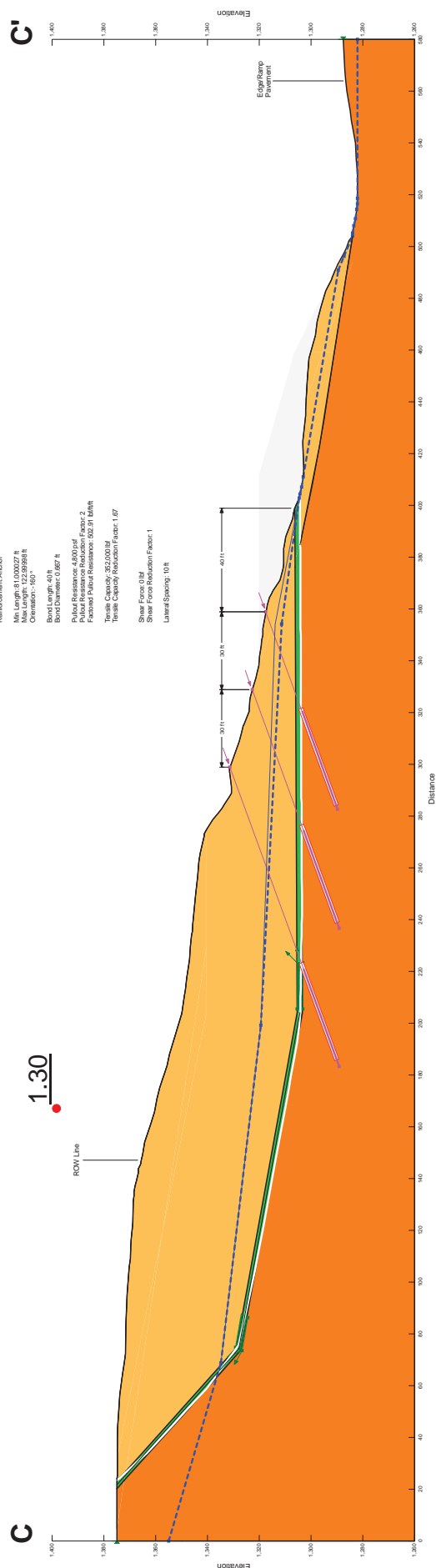
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)	C-Horizontal (psf)	C-Vertical (psf)	Phi-Horizontal (°)	Phi-Vertical (°)
Light Green	Bedrock (Anisotropic)	Anisotropic Strength	125				70	100	22	24
Orange	Overburden	Mohr-Coulomb	120		0	24				
Blue	Reinforced Butress Fill (RSS)	Mohr-Coulomb	125		1,500	28				
Dark Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50						

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SECTION C-C'
GRADING (BUTTRESS)

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FIG. D-15



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C'; LL=80, CF>50		

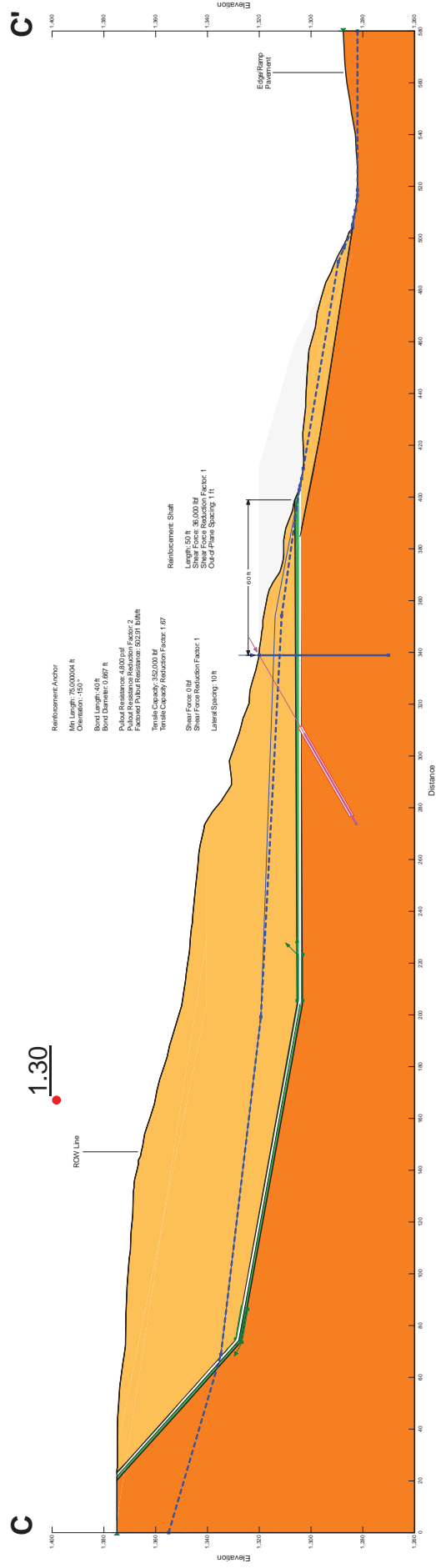
Valley City SW Backslope Repair
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 Valley City, North Dakota

SECTION C-C'
GROUND ANCHORS (GLOBAL)

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FIG. D-16



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	A-A' & C-C', LL=80, CF>50		

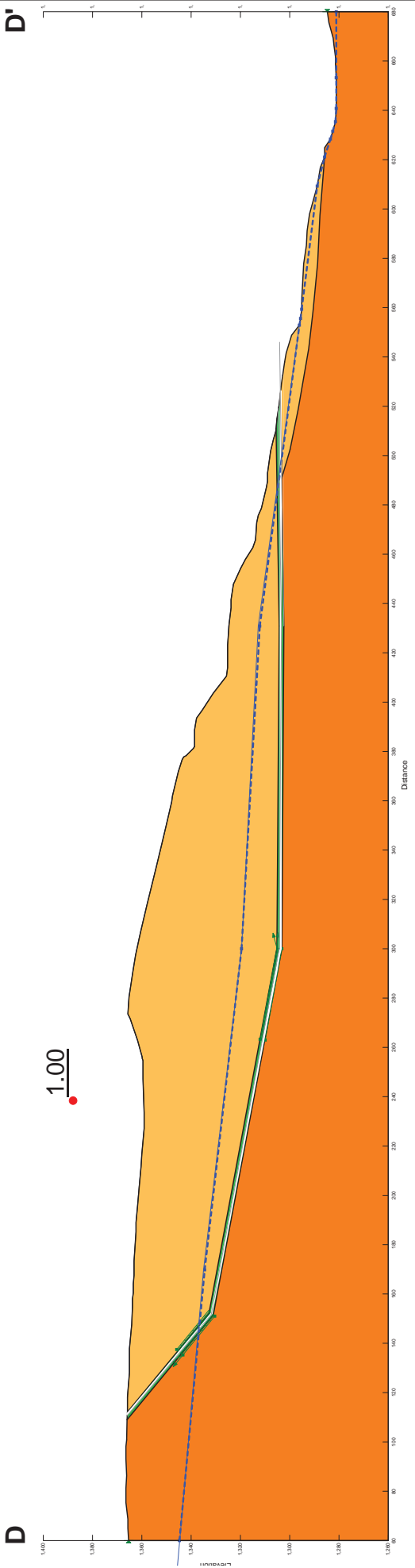
Valley City SW Backslope Repair
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SECTION C-C'
ANCHORED SHAFTS (GLOBAL)

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FIG. D-17



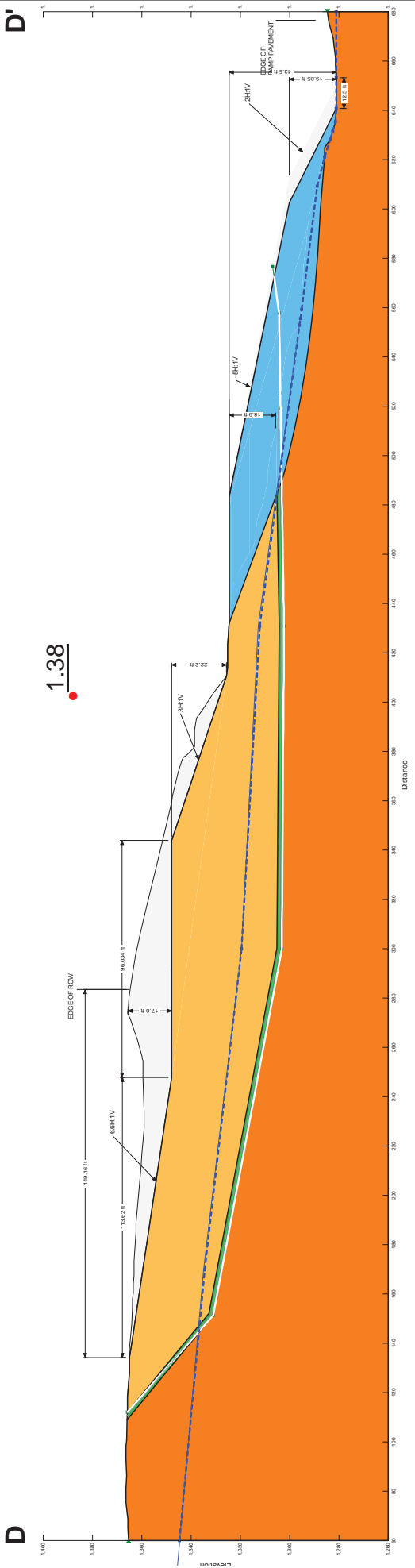
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	D-D', LL=100, CF>50		

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SECTION D-D'
BACK-ANALYSIS

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FIG. D-18



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Blue	Buttruss Fill	Mohr-Coulomb	125		0	28
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	D-D', LL=100, CF>50		

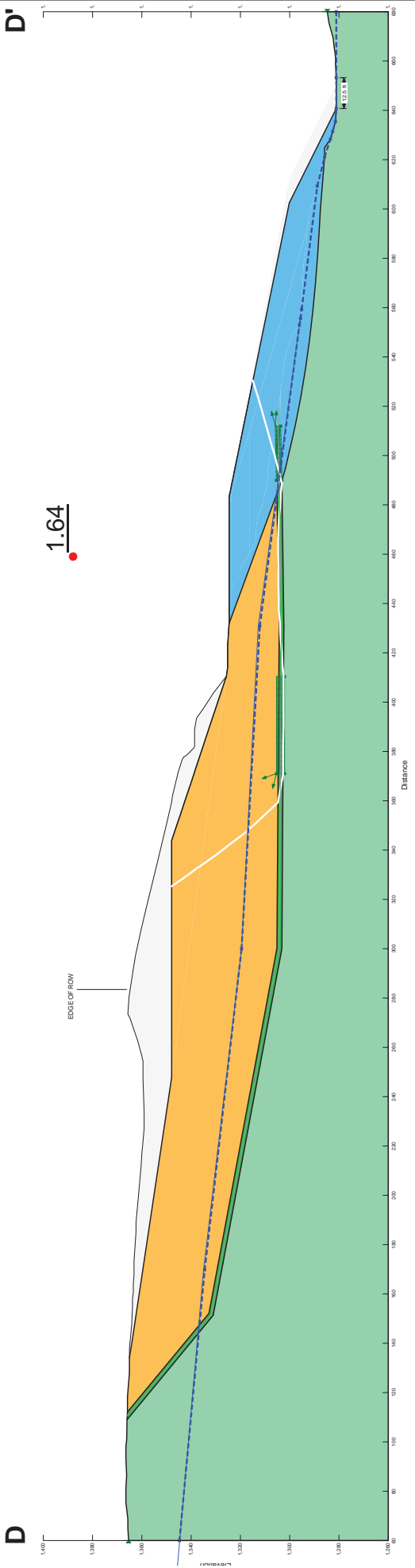
Valley City, SW Backslope Slide Repair
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SECTION D-D'
GRADING (GLOBAL)

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FIG. D-19



Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)	C-Horizontal (psf)	C-Vertical (psf)	Phi-Horizontal (°)	Phi-Vertical (°)
Light Green	Bedrock (Anisotropic)	Anisotropic Strength	125				70	100	22	24
Blue	Buttress Fill	Mohr-Coulomb	125		0	28				
Orange	Overburden	Mohr-Coulomb	120		0	24				
Dark Green	Slip Surf	Shear/Normal Fn.	120	D-D', LL=100, CF>50						

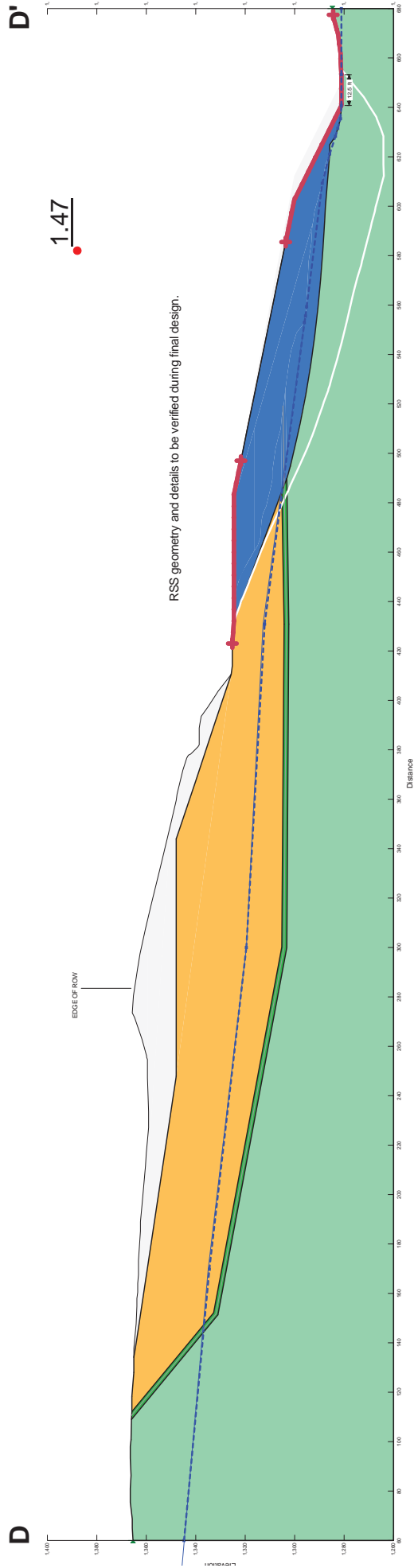
Valley City, SW Backslope Slide Repair
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SECTION D-D'
GRADING (INTERMEDIATE)

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FIG. D-20



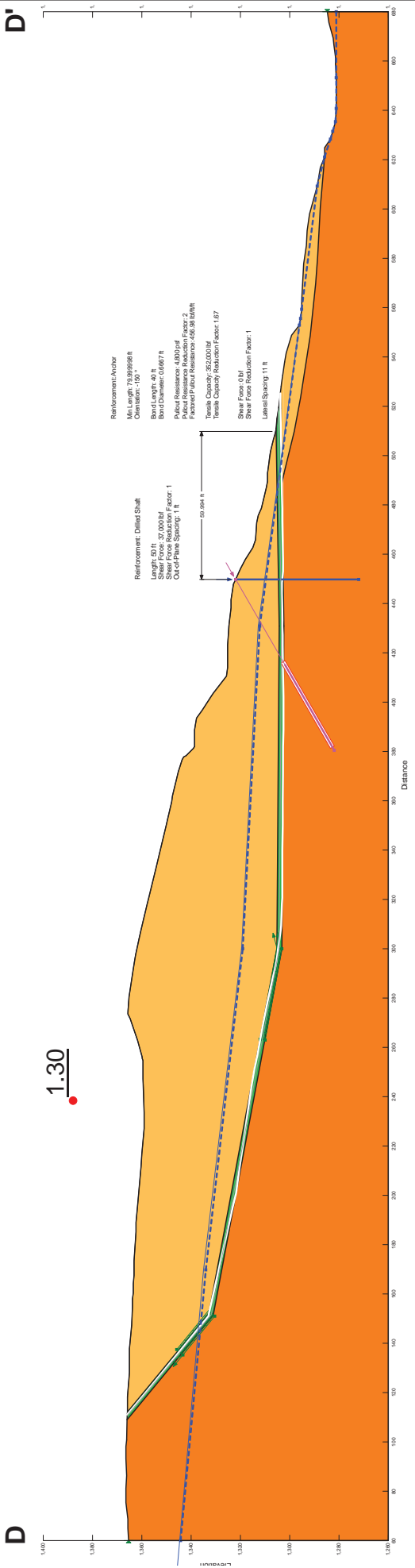
Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)	C-Horizontal (psf)	C-Vertical (psf)	Phi-Horizontal (°)	Phi-Vertical (°)
Light Green	Bedrock (Anisotropic)	Anisotropic Strength	125				70	100	22	24
Orange	Overburden	Mohr-Coulomb	120		0	24				
Blue	Reinforced Buttress Fill (RSS)	Mohr-Coulomb	125		1,500	28				
Dark Green	Slip Surf	Shear/Normal Fn.	120	D-D', LL=100, CF>50						

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SECTION D-D'
GRADING (DOWNSLOPE)

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FIG. D-21



P:\DEM17000s1\07877 NDOT Valley C1001 Geotechnical Seva\Analysis\Slope Stability Analysis for Fracture Section D-D-252

Color	Name	Material Model	Unit Weight (pcf)	Strength Function	Effective Cohesion (psf)	Effective Friction Angle (°)
Orange	Bedrock	Mohr-Coulomb	125		100	24
Yellow	Overburden	Mohr-Coulomb	120		0	24
Green	Slip Surf	Shear/Normal Fn.	120	D-D', LL=100, CF>50		

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SECTION D-D'
ANCHORED SHAFTS (GLOBAL)

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FIG. D-23

Appendix E

Stabilization Alternative Cost Estimates

TABLES

Table E-1: Cost Estimate Data

Table E-1 - Cost Estimate Data

Maintenance Alternative					
Item	Quantity	Unit	Unit Price	Amount	
Maintenance Work	1	LS	\$ 300,000.00	\$ 300,000.00	
Frequency	10	YEAR	-	-	
Discount Rate	1	%	-	-	
TOTALS					
				25-year NPV	\$ 591,118.52
				50-year NPV	\$ 1,456,029.37
				75-year NPV	\$ 2,018,458.36

Drainage Improvements Only					
Item	Quantity	Unit	Unit Price	Amount	
Mobilization	1	LS	\$ 95,916.67	\$ 95,916.67	
Revegetation and Erosion Control	1	LS	\$ 80,000.00	\$ 80,000.00	
Excavation for Finger Drains	6,667	CY	\$ 27.00	\$ 180,000.00	
Drain Aggregate for Finger Drains	6,667	CY	\$ 100.00	\$ 666,666.67	
Misc. Drainage Components	1	LS	\$ 32,500.00	\$ 32,500.00	
TOTAL				\$ 1,055,083.33	

Grading + Drainage Improvements Alternative					
Item	Quantity	Unit	Unit Price	Amount	
Mobilization	1	LS	\$ 347,967.82	\$ 347,967.82	
Cut Volume	57,324	CY	\$ 25.00	\$ 1,433,100.00	
Fill Volume	38,052	CY	\$ 25.00	\$ 951,300.00	
Geosynthetic	8,889	SY	\$ 3.50	\$ 31,111.50	
ROW Acquisition	3.5	ACRE	\$ 30,000.00	\$ 105,000.00	
Revegetation and Erosion Control	1	LS	\$ 80,000.00	\$ 80,000.00	
Excavation for Finger Drains	6,667	CY	\$ 27.00	\$ 180,000.00	
Drain Aggregate for Finger Drains	6,667	CY	\$ 100.00	\$ 666,666.67	
Misc. Drainage Components	1	LS	\$ 32,500.00	\$ 32,500.00	
TOTAL				\$ 3,827,645.98	

Table E-1 - Cost Estimate Data

Ground Anchors and Drainage Improvements Alternative				
Item	Quantity	Unit	Unit Price	Amount
Mobilization	1	LS	\$ 618,954.17	\$ 618,954.17
Post-tensioned Anchors	21,783	LF	\$ 125.00	\$ 2,722,875.00
Reinforced Concrete Anchor Blocks	248	EA	\$ 8,000.00	\$ 1,984,000.00
Verification Test	4	EA	\$ 75,000.00	\$ 300,000.00
Load Cell	4	EA	\$ 8,000.00	\$ 32,000.00
Strain Gauge	8	EA	\$ 8,000.00	\$ 64,000.00
Earthwork for Construction Benches	8,500	CY	\$ 15.00	\$ 127,500.00
Revegetation and Erosion Control	1	LS	\$ 80,000.00	\$ 80,000.00
Excavation for Finger Drains	6,667	CY	\$ 27.00	\$ 180,000.00
Drain Aggregate for Finger Drains	6,667	CY	\$ 100.00	\$ 666,666.67
Misc. Drainage Components	1	LS	\$ 32,500.00	\$ 32,500.00
TOTAL				\$ 6,808,495.83

Anchored Shafts and Drainage Improvements Alternative				
Item	Quantity	Unit	Unit Price	Amount
Mobilization	1	LS	\$ 1,143,858.33	\$ 1,143,858.33
Post-tensioned Anchors	5,753	LF	\$ 125.00	\$ 719,125.00
Verification Test	2	EA	\$ 75,000.00	\$ 150,000.00
Load Cell	2	EA	\$ 8,000.00	\$ 16,000.00
Strain Gauge	6	EA	\$ 8,000.00	\$ 48,000.00
Drilled Shaft - 4.0 ft Diameter	3,636	LF	\$ 750.00	\$ 2,727,000.00
Concrete Cap Beam	800	LF	\$ 1,000.00	\$ 800,000.00
CSL Integrity Testing	15	EA	\$ 6,000.00	\$ 90,000.00
Earthwork for Construction Benches	14,000	CY	\$ 15.00	\$ 210,000.00
Revegetation and Erosion Control	1	LS	\$ 80,000.00	\$ 80,000.00
Excavation for Finger Drains	6,667	CY	\$ 27.00	\$ 180,000.00
Drain Aggregate for Finger Drains	6,667	CY	\$ 100.00	\$ 666,666.67
Misc. Drainage Components	1	LS	\$ 32,500.00	\$ 32,500.00
TOTAL				\$ 6,863,150.00

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken

impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland.