Construction Risk	All Alternatives	Alternative 1	Alternative 2	Alternative 3
Groundwater Conditions	• Alternating layers of sandy and clayey strata from soil borings present a potential for artesian conditions. The potential for artesian conditions and site-specific groundwater conditions must be evaluated through site-specific geotechnical investigation(s), including groundwater monitoring, to inform groundwater control considerations and recommendations required by the TCEQ (Section 4.4) [3]. • Based on findings from the 2024 borings, the shallowest groundwater depth encountered was 5 feet bgs. Saturated foundation conditions are anticipated based on location of Project on watercourse. Dewatering may be required for the Project foundation if groundwater has the potential to pond, pipe, or disturb foundation soils.	Dewatering of the cut-off trench excavation using a dewatering system to remove groundwater may be required. A dewatering plan following applicable environmental regulations is anticipated to be required.	Dewatering of the cut-off trench excavation using a dewatering system to remove groundwater may be required. A dewatering plan following applicable environmental regulations is anticipated to be required.	No specific considerations
Care for Water				
Creek Flow	• Embankment alignment across Walnut and Birch creeks may require diversion of water or creek flow during construction. A plan for care and diversion of water (including sedimentation and pollution control—SWP3 provisions) will be required. • Evaluation of site-specific conditions are required to develop cost-effective and efficient water diversion plans. • It is anticipated that the Project will be constructed under dry conditions to minimize the potential for flood events during construction.	No specific considerations	No specific considerations	No specific considerations

Construction Risk	All Alternatives	Alternative 1	Alternative 2	Alternative 3
Design Considerations				
Dam and Appurtenances	• Embankment slopes, materials (e.g., shell, fill, core, blankets, filters, drains, slope protection), must be evaluated during advanced design (TCEQ Section 6.1 [3]) based on site-specific survey and geotechnical investigations. • Spillway alternatives and other dam appurtenances must be evaluated during advanced design (TCEQ Chapter 7 [3]) based on site-specific survey and geotechnical investigations.	No specific considerations	No specific considerations	No specific considerations
Stability	• Updated stability analyses of the foundation, upstream and downstream slopes will be required during advanced design (TCEQ guidelines Section 4.4 [3]) when site-specific information become available. • Adverse soil conditions (e.g., dispersive, expansive, compressible, soluble material), ground subsidence related to groundwater pumping, and other factors affecting dam stability will be incorporated in stability analyses advanced design when site-specific information become available.	No specific considerations	No specific considerations	No specific considerations
Seismic Stability	• An unnamed southwest-northeast oriented fault approximately 10 miles long crosses the Project area approximately 2 miles north of the northern end of the proposed lake extents (Han, 2013), and seismic stability analyses for natural seismicity may be required (TCEQ guidelines Section 4.4 [3]). • It is recommended that TCEQ is engaged during design advancement to determine if seismic analyses will be required.	No specific considerations	No specific considerations	No specific considerations
Seepage	• An updated seepage analysis will be required during advanced design (TCEQ guidelines Section 4.4 [3]) when site-specific information become available.	No specific considerations	No specific considerations	No specific considerations
Deformation	• Deformation or settlement analysis will be required during advanced design (TCEQ guidelines Section 4.4 [3]) when site-specific information become available.	No specific considerations	No specific considerations	No specific considerations

8.2 Service life

Historically, it has been demonstrated that embankments can have service lives of over 100 years. Proper design, construction, and operation and maintenance practices will extend the service life of an embankment.

8.3 Construction materials

Estimated net volume of the embankment for Walnut Creek Detention Basin and Birch Creek Detention Basin is approximately 238,000 cubic yards and 133,600 cubic yards respectively. Anticipated main dam fill should consist of 20 to 40 percent fines (passing the #200 sieve). This fill should be moisture conditioned and compacted to a specified maximum dry density. Whenever possible, excavated material may be re-used/ repurposed for the main dam fill. Selected materials for embankment fill of the three alternative embankment geometries assume that there exists enough in-situ borrow. Otherwise, main dam fill materials will be imported for selected zones of the embankment where in-situ borrow is lacking based on excavation depth and groundwater constraints. The maximum excavation depths for in-situ borrow sources may be dictated by the groundwater depths. It is assumed some or all vertical chimney drains, horizontal blanket drain, and rock riprap materials will be imported from external borrow sources.

Alternative 1 consists of a homogenous embankment fill and a cutoff trench. The homogenous fill will be constructed from the silty sand and clayey sand soil type (hereafter referred to as Zone B) or materials with similar index properties capable of maintaining slope and foundation stability, and with acceptable permeability. The cutoff trench will be constructed from silty clay and sandy clay type (hereafter referred to as Zone A) or materials with similar or better index properties. Zone A will consist of relatively low permeability materials capable of minimizing seepage to reduce exit gradients that may result from high under-seepage flow. It is assumed that all embankment homogenous fill materials for Alternative 1 will be sourced on-site and the cutoff trench backfill material will be sourced from outside borrow sources.

The cut-off trench backfill material and impervious clay core zone for Alternative 2 will be constructed from Zone A or materials with similar or better index properties. It is assumed that all of the low permeable zones embankment fill (Zone A) for Alternative 2 will be sourced from external borrow sources. The shell zones for Alternative 2 will be constructed from Zone B and it is assumed that these soils will be sourced on-site.

The homogenous earthen fill for Alternative 3 will be constructed from Zone B or materials with similar or better index properties, and the SBC wall will be constructed from imported materials using specialized construction methods. It is assumed that all Zone B materials for the homogenous zone of Alternative 3 will be sourced on-site. A summary of construction materials for the three embankment alternatives is presented in Table 8-2.

Table 8-2. Construction Material for Embankment Zonation

Zonation	Fill Type		
	Alternative 1	Alternative 2	Alternative 3
Shell/Homogenous[1]	Zone B	Zone B	Zone B
Core[2]	—	Zone A	—
Cutoff trench[2]	Zone A	Zone A	—

[1] Zone B— assumed to comprise onsite borrow sources

[2] Zone A— assumed to comprise offsite borrow sources

8.4 Site civil design

8.4.1 Access road design

Access roads will be required for construction, and operation and maintenance following construction. Access roads will be designed along the crest of the Project embankment.

Access roads will be designed in accordance with the Texas Department of Transportation Roadway Design Manual [22], consistent with the State of Texas requirements. The access roads are considered low volume roads and will not be accessible to the public.

All access roads will be geometrically designed to accommodate the following:

- Passenger car
- Single unit truck
- Single unit truck (three axle)
- Car trailer
- Road grader
- Loader
- Intermediate semi-trailer

8.4.2 Clearing and grubbing

Clearing and grubbing of the land at the Project sites will be required for construction of the new facilities, for access road construction and for construction staging. These areas will be further defined during design advancement.

8.4.3 Stormwater

Permanent stormwater provisions will be incorporated as required to prevent site erosion. Features may include curbs, gutters, concrete drainage ditches, or storm drain inlets. Energy dissipation devices will be provided as required to slow down flow velocities.

8.4.4 Temporary facilities

Temporary facilities described in this section include those required during the construction of the Project.

8.4.5 Construction trailers and support facilities

Temporary support facilities required during construction include equipment trailers, temporary storage for equipment maintenance operations, fuel storage, and other facilities required to construct the Project. The Contractor will be allowed to utilize those portions of the site that are designated to be disturbed as required to locate these facilities.

8.4.6 Site utilities

Utilities required for the Project sites during construction and post completion include electrical, communications, sewer, and potable water. Prior to construction, existing utilities at the site, if any, will be confirmed to evaluate if the Contractor will be required to facilitate the installation of any new utilities or the connection to existing utilities. Utilities to be left in place permanently will be evaluated during design advancement.

8.4.7 Commissioning

The initial filling of the Project will be completed by re-diverting creek flow from the construction phase diversion channel to the embankment outlet culverts after construction of the Project. Filling is not anticipated to be completed in stages due to the primary function of the Project as dry detention basins with conduits.

9 Operations and maintenance considerations

It is anticipated that an Operation, Maintenance, and Surveillance Manual and the Emergency Management Plan will be developed for the Project, as required by existing regulations (refer to Section 2).

10 Recommended embankment option

Three individual alternatives for flood mitigation under the Spring Creek Watershed Flood Control Dams conceptual design task have been discussed in this report. Table 10-1 presents a summary of each alternative and its relative pros and cons for constructability, permitting, operation and maintenance, and an anticipated cost of construction based on experience. The estimated construction cost for all three alternatives was performed by Halff Associates and was not available at the time of this DBM.

The anticipated quantities of required import fill for Alternative 2 and specialized construction for Alternative 3 may present increased construction cost and permitting issues, and construction complexities for the Project. Due to the primary function of the Project as dry detention basins, a zoned embankment with an impervious core (Alternative 2) may not be economical or critical to the safe operation of the dam given that long-term seepage conditions are not expected to be established in the dam due to the relatively short flood impoundment durations. Potential seepage losses are not of primary concern for the function of the Project and high exit gradients resulting from high under seepage are not anticipated when a foundation seepage barrier with sufficient imperviousness and depth is installed, hence the installation of a specialized impervious barrier in the case of Alternative 3 may not be economical or warranted for the safe operation of the Project.

Due to these limitations for Alternatives 2 and 3, Black & Veatch recommends Alternative 1 as presented in Section 3.3.2 and Section 7. Alternative 1 would allow for the potential use of on-site borrow sources for the construction of high-volume zones of the dam. Modification to the foundation seepage barrier presented in Section 7 for Alternative 1 (as illustrated in Figure 7-1) may be explored for an advanced design to reduce construction cost while maintaining a sufficient design and safe operation of the dam. The recommended alternative would not only improve construction cost and potentially represent the lowest construction cost amongst other alternatives considered in this DBM, but also reduce construction complexities and time.

Table 10-1 Summary of Alternatives

Alternative	Pros	Cons	Estimated Construction Cost
1— Homogenous embankment with cutoff trench	• Allows for potential use of onsite borrow sources for entire dam construction except for filtered drainage zone and riprap. • Reduced environmental impact from external embankment fill borrow sources. • Reduced construction complexities by use of conventional construction techniques and homogenous embankment. • Potential reduction in construction time.	• No dam through-seepage barrier.	Refer to cost estimate by Halff Associates.
2— Zoned embankment with impervious core and cutoff trench	• Allows for potential use of onsite borrow sources for dam shell construction. • Dam through-seepage barrier. • Less construction complexities compared to Alternative 3 by use of conventional construction techniques.	• Increased cost from potential clay fill import from external borrow sources. • Increased construction complexities compared to Alternative 1. • Potential increase in construction time compared to Alternative 1. • Increased environmental impact from external embankment fill borrow sources.	Refer to cost estimate by Halff Associates.
3— Homogenous embankment with soil-bentonite cutoff wall	• Allows for potential use of onsite borrow sources for entire dam construction except for SBC wall, filtered drainage zone and riprap. • Reduced environmental impact from external embankment fill borrow sources. • Potential reduction in foundation excavation footprint and cost compared to Alternatives 1 and 2.	• No dam through-seepage barrier. • Increased construction complexities by use of specialized construction techniques. • Potential increase in construction time compared to Alternative 1. • Potential increased construction cost by use of specialized construction techniques.	Refer to cost estimate by Halff Associates.

11 Future work

11.1 Next steps

The following items are anticipated to be completed in order to move the Spring Creek Watershed Flood Control Dams project from conceptual design to preliminary and detailed design, and construction level:

- Spillway Sizing and Location, and Freeboard Evaluation
- Geologic and Geotechnical Understanding (site-specific subsurface exploration)
- Borrow Evaluation and Embankment Zoning Plan
- Site Material Parameters
- Settlement Analysis
- Seismic Site Evaluation
- Seepage Analysis
- Stability Analysis
- Foundation Seepage Control
- Filter Compatibility and Internal Stability
- Embankment Slope Protection
- Flood Rim and Upper Reach Considerations
- Diversion Plan
- Conduit Plan
- Permanent Instrumentation
- First Fill and Long-Term Monitoring
- Operation and Maintenance Manual

12 References

1. Texas Water Code (TWC) Chapter 11— Water Rights, current as of October 18, 2024.
2. Texas Administrative Code (TAC) Title 30 Part 1 Chapter 299— Dams and Reservoirs, current as of October 18, 2024.
3. Texas Commission on Environmental Quality (2009). Design and Construction Guidelines for Dams in Texas, RG-473.
4. Texas Commission on Environmental Quality (2007). Hydrologic and Hydraulic Guidelines for Dams in Texas, GI-364.
5. U.S. Army Corps of Engineers EM 1110-2-1902— Slope Stability, 2003.
6. US Department of Energy (USDOE) Wind Energy Study Volume 7 – The South Central Region, March 1981.
7. U.S. Department of Interior Bureau of Reclamation, Design of Small Dams, 3rd Edition, 1987.
8. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 2: Embankment Design, December 2012.
9. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 5: Protective Filters, November 2011.
10. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 3: Foundation Surface Treatment, July 2012.
11. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 6: Freeboard, June 2021.
12. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 7: Riprap Slope Protection, May 2014.
13. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 8: Seepage, January 2014.
14. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 9: Static Deformation Analysis, November 2011.
15. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 13: Seismic Analysis and Design, May 2015.
16. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 12: Foundation and Earth Materials Investigation, July 2012.
17. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 11: Instrumentation and Monitoring, March 2014.

18. U.S. Department of Interior Bureau of Reclamation, Design Standards No. 13 Chapter 10: Embankment Construction, May 2012.

19. U.S. Army Corps of Engineers EM 1110-2-1902— Seepage Analysis and Control for Dams, 2003.

20. Aviles Engineering Corporation (2024). Geotechnical Investigation, Spring Creek Watershed Flood Control Engineering Feasibility Study Report. Report prepared for Halff Associates in November 2024.

21. U.S Department of Interior Bureau of Reclamation, Characteristics of Dispersive and Problem Clay Soils, October 1991.

22. Texas Department of Transportation, Roadway Design Manual, November 2024.

23. Strategic Mapping Program (StratMap). Upper Coast Lidar, 2018-03-22.

Appendix B-1 Design standards, guidelines, and criteria

This appendix contains description of guidelines and/or standards relevant to the various analyses or activities anticipated for the Project design. The sections contained in the appendix provide design criteria and general implementation guidelines for specific elements of the Project.

Embankment Design

Guidelines and standards for embankment design are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 2 Embankment Design [8]. Project considerations for the embankment design are summarized in Table A-1.

It is anticipated that the Project alternative embankments will be designed as either a homogenous or a zoned earthfill embankment, composed primarily of compacted, relatively impervious fill (refer to Section 3.3). Internal drainage and slope protection are described in subsequent sections. The geometry of the conceptual embankment design presented in this DBM based on the stability analyses is presented in Section 7, and it is anticipated that additional refinements to the geometry will be made during design advancement.

Table A-1 Summary of Embankment Design Guidelines and Project Considerations

Chapter/Section No.	Chapter/Section Title	Project Considerations
Design and Construction Guidelines for Dams in Texas TCEQ [3]		
Chapter 4.2	Foundation Examination and Treatment — • Permeable foundation • Saturated foundation • Weak foundation	• It is anticipated that the foundation of the Project embankments near the centerline of the watercourse will be saturated silty and clayey sands, and drier conditions of similar soil type is anticipated along the mild slopes and beyond based on assumption from the 2024 field investigations findings. Note that the borings performed were situated about 1 mile from the sites. • Low density silt and sandy foundations may be subject to strength loss during earthquake loading; however, the risk for seismic activities is low for the Project sites. • There is potential for deformation or even shear failure and erosion resulting from dispersive silty sand soils. Crumb tests based on the ASTM D6572 test standard have been performed to determine the dispersive grade of foundation soils. • At a minimum, a cutoff trench must be excavated along the long axis of the dam foundation in overburden material (e.g., soil, weak rock) if competent rock is not encountered. The cutoff trench must have adequate contact with a suitable impervious subsurface stratum, the suitability and depth of which must be evaluated through site-specific geotechnical investigation(s).

Chapter/Section No.	Chapter/Section Title	Project Considerations
		• A partial cutoff trench or wall to a depth necessary to satisfactorily limit seepage may be explored during advanced design (see Section 7). • Filtered drainage system will be required to provide a free flow of seepage and to prevent internal erosion. • A staged construction or increased embankment width with flatter slopes may be required as mitigation to sliding potential from saturated foundation.
Chapter 4.3	The Analysis of Available Materials	• Borrow investigation(s) will be required to identify suitable embankment and filter/drain materials during design advancement. • Findings from the 2024 field investigations and testing program have been assumed for the conceptual design in this DBM.
Chapter 4.4	Geotechnical Report Requirements — • Test borings • Laboratory testing and analyses • Seepage analysis • Stability analyses • Seismic stability analyses	• Field investigations comprising four Standard Penetration test borings and laboratory testing on sampled soils were performed about 1 mile off the Project sites. See Aviles (2024) [20] for field exploration and test findings. Site specific field exploration and testing program will be required for advanced design. • It is anticipated that the slope stability for the embankments presented in this DBM will be updated during design advancement to incorporate new information (e.g., additional field/laboratory data, earthquake ground motions) and additional cross sections based on site-specific investigations. • It is anticipated that seepage analysis will be updated during design advancement to incorporate new information to advance filter and drain design and to evaluate the need for additional seepage mitigation measures based on site-specific investigations. • Static deformation analysis (settlement, cracking) will be required during design advancement. • Seismic stability analysis may be required during design advancement.
Chapter 6.1	The Basic Components of Embankment Dams— • Homogeneous embankment dams • Zoned embankment dams • Soil filter and drainage system designs	• It is anticipated that filtered drainage system will be incorporated in both homogenous and zoned embankment alternatives. • It is anticipated that stability berms extending the lengths of the upstream and downstream slopes will be incorporated in the dam design. • It is anticipated that embankment slope surface will be protected using rock riprap on the upstream slope and vegetation (short grass cover, free from any trees, large bushes, etc.) on the downstream slope. • It is anticipated that surface drainage of the crest will be provided by sloping the crest at a 2-percent slope to drain

Chapter/Section No.	Chapter/Section Title	Project Considerations
	Surface protection on embankment slopes and crest	towards the upstream slope unless environmental considerations dictate other requirements. • It is anticipated that additional camber may be required (typically 1 to 2 percent of the embankment height), depending on the compressibility of foundation materials. • It is anticipated that aggregate will be placed on the embankment crest to provide surface protection of the crest and function as an inspection road. • Delineation of the crest along the inspection road, by posts or other markers, may be required for safety. • The top of the chimney filter will be designed to extend to or above the maximum reservoir water level corresponding to the PDF. • Sufficient cover will be designed over the chimney filter to protect it from freezing.

Protective Filters

Guidelines and standards for protective filters are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 5 Protective Filters [9]. The Design and Construction Guidelines for Dams in Texas TCEQ [3] notes that filter (sand, gravel, or crushed rock) criteria must be developed based on gradation tests and filter criteria standards. Filter criteria standards are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 5 Protective Filters [9]. Granular filter design criteria from the USBR are summarized in Table A-2.

Table A-2. Granular Filter Design Criteria from the USBR Design Standard No. 13 Embankment Dams: Chapter 5 Protective Filters [9]

Base Soil Category	Percent Finer than No. 200 sieve (0.075 mm) (after regrading where applicable)[1]	Filtering Criteria
(1) Fine silts and clays	> 85	The maximum $D_{15}F$ should be ≤ 9 x $D_{85}B$, but not less than 0.2 mm, unless the soils are dispersive. Dispersive soils require a maximum $D_{15}F$ that is ≤ 6.5 x $D_{85}B$ size, but not less than 0.2 mm.
(2) Silts, clays, silty sands, and clayey sands	40 – 85	The maximum $D_{15}F$ should be ≤ 0.7 mm unless soil is dispersive, in which case the maximum $D_{15}F$ should be < 0.5 mm.

Base Soil Category	Percent Finer than No. 200 sieve (0.075 mm) (after regrading where applicable)[1]	Filtering Criteria
(3) Silty and clayey sands and gravels	15 – 39	A. For nondispersive soils, the maximum $D_{15}F$ should be: $\leq \left[\frac{40-A}{25}\right]\left[(4\,D_{85}\,B)-0.7mm\right]+0.7mm$ where: A = Percent passing No. 200 sieve. When 4 x D85B is less than 0.7 mm, use 0.7 mm B. For dispersive soils, use 0.5 mm.
(4) Sands and gravels	< 15	The maximum $D_{15}F$ should be ≤ 4 x $D_{85}B$ of base soil after regrading.

[1]mm = millimeter

A chimney drain, filter blanket, and toe drain are included in the conceptual design for the embankment (refer to Section 3.3). The chimney drain and filter blanket may be one or two-stage, depending on filter compatibility requirements and embankment materials. Seepage will be conveyed through the chimney drain and filter blanket to a toe drain embedded pipe collection system and discharged into a surface ditch.

Borrow investigation(s) will be required to evaluate filter criteria and compatibility during design advancement (refer to Site Investigation section below).

Foundation Preparation

Guidelines and standards for foundation preparation are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 3 Foundation Surface Treatment [10].

Foundation preparation is anticipated for the Project embankments where saturated soft soil is present in the foundation. For the embankment foundation including abutments and in the creek bed, it is anticipated that the foundation soil will be prepared by excavation, proof rolling or treated based on determined geological conditions for embankment construction. The need for a cutoff trench or other seepage barrier will be evaluated based on the selected embankment alternative.

Slope Protection

Guidelines and standards for slope protection are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 7 Riprap Slope Protection [12]. The Design and Construction Guidelines for Dams in Texas TCEQ [3] note that the upstream slopes should have adequate protection against erosion and breaching from waves, and the downstream slopes should have adequate protection against erosion from runoff, seepage, traffic, and burrowing animals.

The conceptual design of the Project embankments includes riprap protection on the upstream slope and topsoil and vegetation protection on the downstream slope. It is anticipated that riprap on the upstream slope may require a two-stage sand or filter cloth and gravel filter to meet filter compatibility requirements. Excavated material from the Project embankments foundation may be suitable for reuse as topsoil on the downstream slope. Borrow investigation(s) will be required to establish suitable slope protection materials and filter compatibility requirements during design advancement (refer to Site Investigation section below).

Slope Stability Analysis

Guidelines and standards for slope stability analysis are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3], USBR Design Standard No. 13 Embankment Dams: Chapter 9 Static Deformation Analysis [14], and USACE EM 1110-2-1902 – Slope Stability [5].

Consistent with the Design and Construction Guidelines for Dams in Texas TCEQ [3], a standards-based approach with target factors of safety (FoS) as acceptance criteria is used for slope stability analysis. A limit equilibrium analysis is considered sufficient to evaluate slope stability under normal operating conditions, as described in the USACE EM 1110-2-1902 – Slope Stability [5]. Industry standard FoS for various loading conditions, the selected target FoS, and target FoS justifications are summarized in Table A-3.

End of construction, long term, flood, and rapid drawdown loading conditions are evaluated for the Project embankments within the scope of this DBM (Section 6 and 0).

Table A-3. Acceptance Criteria (Factors of Safety) for Slope Stability Analysis

Loading Condition	TCEQ Min. FoS	USBR Min. FoS	Design Basis Shear Strength Parameters	Design Basis FoS	Justification
End of Construction	1.25	1.3 – 1.4[1]	Undrained	1.3	Consistent with minimum FoS from published guidelines; undrained shear strength parameters are the design basis for the analysis
Long Term (Normal 100-year Flood)	1.5	1.5	Drained	1.5	Consistent with minimum FoS from published guidelines
Peak Design Flood	—	1.2 – 1.3[2]	Drained	1.2 – 1.3	Consistent with minimum FoS from published guidelines; the Project is a flood control detention dam with outlet works and is expected to drain flood storage quickly
Full or Partial Rapid Drawdown	1.2	1.2 – 1.3[3]	Drained Undrained	1.2 – 1.3	Consistent with published guidelines; frequent drawdown is considered for the Project[3]

[1] USBR notes that a minimum FoS of 1.3 is adequate for analysis using effective shear strength parameters with field monitoring during construction or for analysis using undrained shear strength parameters. A minimum FoS of 1.4 should be used for effective shear strength parameters and if pore pressures are not monitored during construction.

[2] USBR notes that a FoS of 1.2 is adequate for short flood pool durations and steady-state seepage conditions. A FoS approaching 1.2 is adequate considering the relatively short flood durations that is expected for the dry detention dams. Long-term seepage phreatic surface under normal reservoir level is not anticipated to be established considering that the dams are not intended to impound water for long durations.

[3] USBR notes that a FoS of 1.3 is adequate for drawdown below the normal operating pool; a FoS of 1.3 is adequate for drawdown from maximum flood pool to dry creek channel grade.

Seepage Analysis

Guidelines and standards for seepage analysis are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3], USBR Design Standard No. 13 Embankment Dams: Chapter 8 Seepage [13], and USACE EM 1110-2-1901– Seepage Analysis and Control for Dams [19]. The Design and Construction Guidelines for Dams in Texas TCEQ [3] note that for high- and significant-hazard dams that will permanently impound water, seepage exit gradients should be within acceptable limits for the embankment and foundation materials. The purpose of the Project as flood control may not impose significant limitations on the allowable seepage, however the potential occurrence of dispersive foundation soils may require adequate seepage control measures. The selected seepage control measure will be evaluated based on the selected embankment alternative. Seepage analysis is evaluated for the Project embankments within the scope of this DBM (Section 5 and 0).

Industry standard FoS for exit gradients, the selected target FoS, and target FoS justifications are summarized in Table A-4.

Table A-4. Acceptance Criteria (Factors of Safety) for Exit Gradient

Type of Facility	TCEQ Min. FoS	USBR Min. FoS	USACE Min. FoS	Design Basis FoS	Justification
New Dam	—	4.0[1]	1.5 - 15[2]	4.0	Consistent with minimum FoS from published guidelines.

[1] USBR notes that a minimum FoS of 4.0 is adequate for high exit gradients in a cohesionless soil when designing either a new dam or remedial repairs at an existing dam to rectify a high exit gradient situation.

[2] USACE notes that a FoS of 1.5 - 15 is adequate for escape gradient depending on knowledge of soil and possible seepage conditions. USACE also notes that generally, factors of safety in the range of 4-5 (Harr 1962, 1977) or 2.5-3 (Cedergren 1977) have been proposed.

Site Investigation

Guidelines and standards for site investigations, including borrow investigations, are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 12 Foundation and Earth Materials Investigation [16].

A field and laboratory testing program will be required to evaluate potential borrow sources and suitability of potential borrow materials for the Project. Imported materials may be required if suitable borrow quantities or materials are not identified.

One field exploration and laboratory testing program has been completed to evaluate foundation conditions 1 mile away from the Project vicinity, documented in the Aviles 2024 Spring Creek Watershed Flood Control Engineering Feasibility Study Report [20]. Project site-specific field exploration and laboratory testing program(s) will be required during design advancement.

Instrumentation and Monitoring

Guidelines and standards for instrumentation and monitoring are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 11 Instrumentation and Monitoring [17]. Instrumentation and monitoring are required for various activities during the project life cycle of a dam, including the following:

- Original design
- Original construction
- Modification
- First reservoir filling
- Long-term performance monitoring
- Response to adverse or anomalous performance
- Decommissioning

Project considerations for instrumentation and monitoring of the Project are summarized in Table A-5. These considerations are primarily intended to encompass long-term performance monitoring for the Project; however, water pressure and deformation monitoring will also be required for design, construction, and first embankment flooding. Strategic instrumentation and monitoring planning may allow for instruments used in design, construction, and first embankment flooding to be converted for long-term performance monitoring. Instrumentation and monitoring in response to adverse or anomalous performance would be developed on an as-needed basis if indicated.

Given the primary function of the Project as dry detention creek, the need, type, and quantity of instrumentation for the Project would be developed during design advancement taking into account what critical elements of the Project require monitoring.

Table A-5 Summary of Instrumentation and Monitoring Guidelines and Project Considerations

Monitoring Type[1]	Project Purpose	Project Considerations
Seepage	• Long-term performance monitoring	• It is anticipated that seepage through the dam will be minimal, if any, based on the assumption that long-term phreatic surface may not be established in the lifetime of the dam considering the relatively short duration of impoundment after flood. • Installation of weirs or flow measuring devices to measure through-dam seepage may not be required.
Water Pressure	• Design • Construction • First reservoir filling • Long-term performance monitoring	• It is anticipated the dams will remain dry than wet for longer durations. Hence, it is anticipated that manual nested vibrating wire piezometers will be installed at large intervals for the Project. • The nested piezometers will be installed at 1500 feet spacing along the embankment. At each location, one piezometer will be installed within the foundation to monitor groundwater pressures; installation of a second piezometer near the embankment toe to monitor through-seepage pore pressures may not be required.

Monitoring Type[1]	Project Purpose	Project Considerations
Deformation	Design Construction First reservoir filling Long-term performance monitoring	It is anticipated that in-place inclinometers and horizontal settlement arrays with ADAS will not be installed for the Project. Survey monuments may be installed on the crest at 1000 feet spacing, or in place of that field survey measurements may be conducted at regular time intervals to monitor vertical settlements.

[1] Refer to USBR Design Standard No. 13 Embankment Dams: Chapter 11 Instrumentation and Monitoring [17].

Construction

Guidelines and standards for embankment construction are described in the Design and Construction Guidelines for Dams in Texas TCEQ [3] and USBR Design Standard No. 13 Embankment Dams: Chapter 10 Embankment Construction [18] and guidelines for construction performance monitoring are described in USBR Design Standard No. 13 Embankment Dams: Chapter 11 Instrumentation and Monitoring [17]. Project considerations for construction of the Project are summarized in Table A-6. Additional construction considerations are described in Section 8.

Table A-6 Summary of Construction Guidelines and Project Considerations

Chapter No.	Chapter Title	Project Considerations
USBR Design Standard No. 13 Embankment Dams: Chapter 10 Embankment Construction [18]		
10.3	Foundation Treatment	It is anticipated that soft soils in the foundation will be excavated or treated using foundation improvement methods before placement of compacted fill.
10.4	Dewatering and Unwatering	The Project is located in a creek with wet surface conditions and possibly shallow groundwater; it is anticipated that creek flow diversion, dewatering and unwatering systems will be required during construction.
10.5	Borrow Areas and Quarries	Borrow materials are required for construction of the Project; plans and/or specifications must be developed for excavation, hauling, handling, and separating equipment, borrow area operation, stockpiling, and borrow area treatment (remediation).
10.6	Embankment Construction	Specifications for fill requirements, including materials, compaction, and equipment, must be developed; it is anticipated that pervious and impervious fill will be required for the Project. Scheduling and sequencing of construction must be developed. Stability of temporary and permanent slopes during construction must be anticipated and evaluated.

Appendix B-2 Aviles Engineering Corp. geotechnical report

AVILES
ENGINEERING CORP.

GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
SPRING CREEK WATERSHED
FLOOD CONTROL ENGINEERING FEASIBILITY STUDY
WALLER COUNTY, TEXAS

Reported to:
Halff Associates, Inc.
The Woodlands, Texas

by

Aviles Engineering Corporation
5790 Windfern
Houston, Texas 77041
713-895-7645

REPORT NO. G154-21 (Final)

November 2024

AVILES
ENGINEERING CORP.

November 18, 2024

Mr. Andrew Moore, P.E., CFM
Halff Associates, Inc.
9303 New Trails Drive, Suite 400
The Woodlands, Texas 77381

Reference: **Geotechnical Investigation**
San Jacinto River Authority
Spring Creek Watershed
Flood Control Engineering Feasibility Study
Walnut Creek and Birch Creek Earth Dams
Waller County, Texas
AEC Report No. G154-21 (Final)

Dear Mr. Moore,

Aviles Engineering Corporation (AEC) is pleased to present this geotechnical investigation for the above referenced project. The project terms and conditions were in accordance with the Standard Subcontract for Subsurface/Underground Services between Halff Associates, Inc. (HAI) and AEC. The project scope was authorized under HAI Contract #21-0016, Work Order No. 1 (dated September 23, 2021), based upon AEC Proposal No. G2021-07-07R3, dated August 6, 2021.

AEC appreciates the opportunity to be of service to you. Please call us if you have any questions or comments concerning this report or when we can be of further assistance.

Respectfully submitted,
Aviles Engineering Corporation
(TBPELS Firm Registration No. F-42)

WILBER L. WANG
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11/18/2024

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Reports Submitted: 1 Halff Associates, Inc. (electronic)
1 Black & Veatch
1 File (electronic)

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GEOTECHNICAL INVESTIGATION

SAN JACINTO RIVER AUTHORITY
SPRING CREEK WATERSHED
FLOOD CONTROL ENGINEERING FEASIBILITY STUDY
WALLER COUNTY, TEXAS

1.0 INTRODUCTION

1.1 Project Description

Aviles Engineering Corporation (AEC) performed a geotechnical investigation for the San Jacinto River Authority's (SJRA) proposed Spring Creek Watershed Flood Control Engineering Feasibility Study. The feasibility study covers two proposed earthfill dams that will be located on Walnut Creek and Birch Creek, at sites that are approximately 0.9 miles upstream of both creeks, respectively, from where they cross FM 1488, in Waller County, Texas. A vicinity map of the project location is presented on Plate A-1, in Appendix A.

1.2 Purpose and Scope

Because of site access issues AEC was only able to perform soil borings along FM 1488, which as noted above are approximately 0.9 miles away from the location of the proposed earthfill dams. AEC's soil boring locations in relation to the proposed earth dam sites are presented on Plate A-2, in Appendix A. Given the locations of the borings from the proposed earth dam sites, **this report is to be used for preliminary analysis and design purposes only and should not be used for final design purposes**. Additional geotechnical borings must be performed at the actual dam sites during the final design phase.

Based on AEC's discussion with Black & Veatch (B&V), the purpose of this investigation is to perform soil borings and geotechnical laboratory testing to obtain an idea of the soil types and strata as well as groundwater information that hopefully is representative of the soil and groundwater conditions at the proposed earth dam sites. This includes baseline soil parameters for each boring that can be used for preliminary analysis and design purposes, and to provide material requirements for the major components of the earth dams. AEC understands that three cross sections are under consideration for the preliminary earthfill dam designs. These preliminary cross sections from B&V are presented in Plates B-5 and B-6, in Appendix B, for reference.

The scope of this preliminary design geotechnical investigation is summarized as below:

1. Drilling and sampling a total of four soil borings ranging from 90 to 120 feet deep.
2. Soil laboratory testing on selected soil samples.
3. Provide baseline soil parameters for each boring.
4. Provide material requirements for the three preliminary earthfill dam cross sections.

2.0 SUBSURFACE EXPLORATION

Subsurface conditions downstream of the proposed dam sites were investigated by drilling a total of four borings ranging in depth from 90 to 120 feet along FM 1488. Borings B-1 and B-2 were drilled for the Walnut Creek Dam and Borings B-3 and B-4 were drilled for the Birch Creek Dam. Boring locations were marked by AEC personnel in the field using a hand-held GPS unit. A boring location plan is presented on Plate A-2, in Appendix A. AEC boring logs are presented on Plates A-3 to A-6, in Appendix A. AEC's boring locations were not surveyed after drilling was completed. Rough elevations for the borings were estimated from Google Earth and are included on the boring logs.

AEC's borings were drilled using a truck-mounted drilling rig. The borings were initially advanced using dry auger method and then completed using wet rotary method once groundwater was encountered, aside from Boring B-1 where wet rotary method was started before groundwater was encountered.

Most samples of cohesive and granular soils were sampled with a 2-inch split-barrel sampler in accordance with ASTM D 1586. The split-barrel samplers were driven using an automatic hammer. An automatic hammer energy calibration report prepared by Fugro USA Land, Inc. (dated March 2, 2023) for AEC's drilling subcontractor (Van & Sons Drilling Services, Inc.) is presented in Appendix C-5, for reference. AEC notes that the calibration report is based on the use of an automatic hammer to drive Texas Department of Transportation (TxDOT) Texas Cone Penetrometer (TCP) tests, but the same automatic hammer is also used when driving split-barrel samplers. Standard Penetration Test resistance (N) values were recorded for the granular soils as "Blows per Foot" and are shown on the boring logs; the value presented in the boring logs are the direct blow counts, $N_{measured}$, without applied correction factor.

Some relatively undisturbed samples of cohesive soils and disturbed samples of granular soils were also obtained from the borings by pushing 3-inch diameter thin-wall, seamless steel Shelby tube samplers in accordance with ASTM D 1587. Strength of the cohesive soils was estimated in the field using a hand penetrometer. The undisturbed samples of cohesive soils were extruded mechanically from the tube samplers in the field and wrapped in aluminum foil; all samples were sealed in plastic bags to reduce moisture loss and disturbance. The samples were then placed in core boxes and transported to the AEC laboratory for testing and further study. Groundwater levels in the borings were measured as soon as groundwater was encountered during drilling and again approximately 15 minutes after groundwater was encountered, aside for Boring B-1 where wet rotary was begun before groundwater was encountered. The borings were backfilled with bentonite chips upon completion of drilling; existing pavement was patched with cold-placed asphalt.

3.0 LABORATORY TESTING

Soil laboratory testing was performed by AEC personnel. Samples from the borings were examined and classified in the laboratory by a technician under supervision of a geotechnical engineer. Laboratory tests were performed on selected soil samples in order to evaluate the engineering properties of the foundation soils in accordance with applicable ASTM Standards. Soil classification and index property tests included Atterberg limits, moisture content, percent passing No. 200 sieve, grain size analysis (i.e., sieve and hydrometer), and dry unit weight tests. Torvane (TV), unconfined compression (UC) and unconsolidated-undrained (UU) triaxial tests were performed on selected samples to estimate the undrained shear strength of cohesive soils. Consolidated-undrained (CU) triaxial tests (with pore water pressure measurements) were performed to estimate the effective stress shear strength of cohesive soils. The laboratory test results are summarized on their respective boring logs, which are presented on Plates A-3 through A-6, in Appendix A. The key to symbols, classification of soils for engineering purposes, terms used on boring logs, and ASTM/TxDOT designation for soil laboratory testing are presented on Plates A-7 through A-10, respectively, in Appendix A.

Grain Size Analysis: To evaluate the grain size distribution of granular soils, sieve and hydrometer tests were performed on selected soil samples in accordance with ASTM D 6913 and D 7928, respectively. Sieve and hydrometer analysis results are presented on Plates A-11 through A-17, in Appendix A.

Double Hydrometer Tests: To evaluate the dispersive characteristics of clayey soils, double hydrometer tests were performed on selected soil samples in accordance with ASTM D 4221. The results of the double hydrometer tests are summarized in Table 1 and are presented on Plates A-18 through A-20, in Appendix A. When the percent dispersion is less than 30, it indicates that the soil is non-dispersive. When the percent dispersion equals 30 but is less than 50, it indicates that the soil is intermediately dispersive. When the percent dispersion is greater than 50, it indicates that the soil is dispersive. AEC notes that dispersive clayey soils are prone to erosion failure by piping.

Table 1. Summary of Double Hydrometer Test Results

Sample ID and Description	Dispersion (%)	Dispersive Classification
B-1, 8’-10’, Clayey Sand (SC)[(a)]	24.46	Non-dispersive
B-3, 4’-6’, Clayey Sand (SC) [(a)]	29.10	Non-dispersive
B-4, 2’-4’, Clayey Sand (SC) [(a)]	8.27	Non-dispersive

Notes: (a) Tests were performed on the clayey portion of the sample.

Crumb Tests: To evaluate the dispersive characteristics of clayey soils, crumb tests were performed on selected soil samples in accordance with ASTM D 6572, Method A. The results of the crumb tests are summarized in Table 2 and are presented on Plate A-21, in Appendix A. AEC notes that dispersive clayey soils are prone to erosion failure by piping.

Table 2. Summary of Crumb Test Results

Sample ID and Description	Dispersive Grade	Dispersive Classification
B-1, 6’-8’, Sandy Lean Clay (CL)	2	Intermediate
B-4, 4’-6’, Clayey Sand (SC)	1	Non-dispersive
B-4, 12’-14’, Silty Clayey Sand (SC-SM)	3	Dispersive

Unconfined Compressive (UC) Strength and Unconsolidated-Undrained (UU) Triaxial Tests: UC and UU tests were performed on selected soil samples in accordance with ASTM D 2166 and ASTM D 2850, respectively. The UC tests measure the unconfined compressive strength of relatively undisturbed cohesive soils (or in some cases, clayey sands) using strain-controlled deformation under axial load application (without confining pressure). The UU triaxial test is similar to the UC test, except that the soil specimen is first placed in a pressurized triaxial cell and subjected to a lateral confining pressure. The undrained shear strength of the soil sample is taken as one-half of its compressive strength. Stress-strain curves from the UC and UU tests are presented in Appendix C-4. A summary of undrained shear strength from the UC and UU tests is presented on Table 3.

Table 3. Summary of Undrained Shear Strength from UC and UU Tests

Sample ID and Description	Compressive Strength, Q_u (tsf)	Shear Strength, C_u (psf)	Peak Strain (%)
B-1, 6’-8’, Sandy Lean Clay (CL)	1.91	1,910	7.57
B-1, 12’-14’, Clayey Sand (SC)	1.03	1,030	4.05
B-1, 33’-35’, Lean Clay (CL)	2.01	2,010	10.08

Sample ID and Description	Compressive Strength, Q_u (tsf)	Shear Strength, C_u (psf)	Peak Strain (%)
B-1, 63’-65’, Sandy Fat Clay (CH)	0.69	690	3.27
B-2, 53’-55’, Sandy Lean Clay (CL)	4.26	4,260	4.17
B-2, 88’-90’, Sandy Lean Clay (CL)	3.13	3,130	3.36
B-3, 4’-6’, Clayey Sand (SC)	1.18	1,180	5.55
B-4, 4’-6’, Clayey Sand (SC)	1.09	1,090	5.55

Consolidated-Undrained Triaxial Tests: CU triaxial tests (with pore water pressure measurements) were performed to determine effective stress and total stress soil parameters. Using the CU data, AEC plotted the stress paths and determined the k_f (critical state) line from the stress paths in accordance with the US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4. Based on the k_f line, AEC determined the strength parameters (cohesion and friction angle) of the soil. Mohr’s circles were plotted in general accordance with ASTM D6747. The Mohr Coulomb diagrams (with Mohr’s Circles at failure) generated from the CU triaxial tests are included on Plates A-22 through A-24, in Appendix A. Stress paths and backup test data is presented on Appendices C-1 through C-3. The shear strength parameters obtained from the CU triaxial tests are summarized in Table 4.

Table 4. Summary of Shear Strength Parameters from CU Triaxial Tests

Sample ID and Description	Effective Stress		Total Stress	
	c' (psf)	φ' (deg)	c_{cu} (psf)	φ_{cu} (deg)
B-1, 8’-10’, Clayey Sand (SC)	200	31.1	210	23.6
B-2, 43’-45’, Sandy Fat Clay (CH)	780	24.4	830	18.1
B-3, 33’-35’, Fat Clay with Sand (CH)	240	18.0	240	14.6

Notes: (1) c' = effective cohesion, φ' =effective friction angle, obtained from CU tests with pore water pressure measurements;
(2) c_{cu} = cohesion in total stress, φ_{cu} = friction angle in total stress, obtained from CU tests.

One-Dimensional Consolidation Tests: One-dimensional consolidation tests was performed in accordance with ASTM D 2435 on a selected soil sample in order to evaluate the general compressibility characteristics of soils at the site. The results of the consolidation tests are presented in Table 5 and on Plates A-25 through A-28, in Appendix A.

Table 5. Summary of Consolidation Test Results

Sample ID and Description	e_0	C_c	C_r	p_c (tsf)	OCR
B-1, 58’-60’, Sandy Fat Clay (CH)	0.9065	0.3109	0.0769	10.1	2.9

Sample ID and Description	e_0	C_c	C_r	p_c (tsf)	OCR
B-2, 53’-55’, Sandy Lean Clay (CL)	0.5121	0.1498	0.0116	6.2	3.2
B-2, 83’-85’, Sandy Lean Clay (CL)	0.5459	0.1110	0.0133	3.8	1.4
B-4, 48’-50’, Fat Clay (CH)	0.7329	0.2362	0.0341	8.8	3.8

Note: (1) e_0 = initial void ratio.
(2) C_c = compression ratio, C_r = recompression ratio, which is derived from the recompression curve.
(3) p_c = preconsolidation pressure, and OCR = overconsolidation ratio.

Permeability Tests: AEC performed permeability tests on selected soil samples in accordance with ASTM D 5084 Method F to evaluate the permeability of in-situ soils. A summary of permeability test results is presented on Table 6 and on Plates A-29 through A-35, in Appendix A.

Table 6. Permeability Test Results

Sample ID and Description	Hydraulic Conductivity (cm/sec)	Average Hydraulic Conductivity (cm/sec)
B-2, 48’-50’, Sandy Lean Clay (CL)	2.83E^{-08}	1.53E^{-08}
	1.21E^{-08}	
	8.89E^{-09}	
	1.18 E^{-08}	
B-4, 53’-55’, Clayey Sand (SC)	1.28E^{-07}	1.20E^{-07}
	1.22E^{-07}	
	1.09E^{-07}	

Moisture-Density Relationships: Based on AEC’s discussion with B&V, moisture-density relationships were performed on selected soil samples to determine compaction characteristics on soils that could potentially be used for dam fill material. AEC prepared composite samples by combining all “CL”-type samples from Borings B-1 and B-2 (i.e., Composite 1 for Walnut Creek Dam) and Boring B-3 (i.e., Composite 2 for Birch Creek Dam). The “CL”-type soils from the borings were mixed and split in general accordance with ASTM C 702. After splitting, Atterberg limits (ASTM D 4318) and a Percent Passing a 200-sieve analysis (ASTM D 1140) were performed to determine the index properties and grain size distribution of the samples. The samples were molded and compacted in accordance with ASTM D 698 (Standard Proctor).

Standard Proctor compaction test results on the composite soils are presented on Plates A-36 and A-37, in Appendix A. A summary of composite sample index properties and Proctor test results are presented on Table 7.

Table 7. Composite Sample Properties and Proctor Results

Sample ID and Description	Liquid Limit (%)	Plasticity Index (%)	Percent Passing #200 Sieve (%)	ASTM D 698 Maximum Dry Density (pcf)	ASTM D 698 Optimum Moisture Content (%)
Composite 1, Sandy Lean Clay (CL)	42	16	63.4	107.2	16.7
Composite 2, Lean Clay with Sand (CL)	47	17	70.6	107.1	18.6

4.0 SITE CONDITIONS

As noted in Section 1.2 of this report, AEC was not able to access the location of the proposed dam sites and as a result, was not able to perform a preliminary site reconnaissance.

4.1 Subsurface Conditions

Details of the soils encountered in AEC's borings are presented in the boring logs on Plates A-3 through A-6, in Appendix A. Soil strata encountered in the borings are also summarized below.

Boring	Depth (ft)	Description of Stratum
B-1	0 - 1.25	Pavement and Base: 3" asphalt and 12" sand and gravel base
	1.25 - 4	Silty Sand (SM)
	4 - 8	Firm to very stiff, Sandy Lean Clay (CL)
	8 - 22	Medium dense, Clayey Sand (SC)
	22 - 32	Medium dense, Poorly Graded Sand with Silt (SP-SM)
	32 - 38	Stiff to very stiff, Lean Clay (CL), with sand pockets
	38 - 42	Medium dense, Clayey Sand (SC), with lean clay pockets
	42 - 47	Very stiff, Lean Clay (CL)
	47 - 52	Medium dense, Poorly Graded Sand with Silt (SP-SM), with lean clay pockets
	52 - 67	Firm to very stiff, Sandy Fat Clay (CH)
	67 - 82	Medium dense to dense, Silty Sand (SM)
	82 - 87	Dense, Poorly Graded Sand with Silt (SP-SM)
	87 - 90	Very stiff, Lean Clay (CL), with sand pockets
B-2	0 - 1.25	Pavement and Base: 3" asphalt and 12" sand and gravel base
	1.25 - 16	Very loose to medium dense, Silty Sand (SM)
	16 - 18	Stiff, Sandy Lean Clay (CL)
	18 - 27	Loose to medium dense, Clayey Sand (SC), with lean clay pockets
	27 - 42	Loose to medium dense, Poorly Graded Sand with Silt (SP-SM)
	42 - 47	Hard, Sandy Fat Clay (CH), with calcareous and ferrous nodules
	47 - 71	Stiff to hard, Sandy Lean Clay (CL), with calcareous nodules
	71 - 77	Medium dense, Silty Sand (SM), with lean clay pockets
	77 - 97	Stiff to hard, Sandy Lean Clay (CL)

Boring	Depth (ft)	Description of Stratum
B-2 (cont.)	97 - 112	Dense to very dense, Silty Sand (SM)
	112 - 120	Dense to very dense, Poorly Graded Sand with Silt (SP-SM)
B-3	0 - 1.25	Pavement and Base: 3" asphalt and 12" sand and gravel base
	1.25 - 4	Loose, Silty Sand (SM), with lean clay pockets
	4 - 27	Very loose to medium dense, Clayey Sand (SC)
	27 - 32	Very loose, Silty Sand (SM), with lean clay pockets
	32 - 42	Very stiff to hard, Fat Clay with Sand (CH), with ferrous nodules
	42 - 47	Medium dense, Silty Sand (SM), with lean clay layers
	47 - 62	Very soft to hard, Sandy Lean Clay (CL)
	62 - 82	Medium dense to dense, Poorly Graded Sand with Silt (SP-SM), with lean clay pockets
	82 - 97	Medium dense, Clayey Sand (SC)
	97 - 103	Very dense, Poorly Graded Sand with Silt (SP-SM)
	103 - 112	Medium dense to dense, Clayey Sand (SC)
	112 - 117	Very dense, Silty Sand (SM), with lean clay pockets
	117 - 120	Very stiff, Silty Clay with Sand (CL-ML)
B-4	0 - 0.92	Pavement and Base: 7" asphalt and 4" sand and iron ore base
	0.92 - 8	Medium dense, Clayey Sand (SC)
	8 - 22	Medium dense to dense, Silty Clayey Sand (SC-SM)
	22 - 47	Medium dense, Poorly Graded Sand with Silt (SP-SM)
	47 - 52	Hard, Fat Clay (CH), with slickensides
	52 - 62	Clayey Sand (SC)
	62 - 77	Dense, Poorly Graded Sand with Silt (SP-SM)
	77 - 90	Dense to very dense, Poorly Graded Sand (SP)

4.2 Ground Water

AEC monitored groundwater during drilling and upon completion of drilling in the borings. Groundwater levels and boring cave in depths encountered during drilling are presented in Table 8.

Table 8. Water Levels in Borings

Boring No.	Date Drilled	Approx. Boring Elevation (ft)	Boring Depth (ft)	Water Depth (ft)	Boring Cave in Depth (ft)
B-1	02/19/2024	+250	90	Wet rotary drilling started at 20', before groundwater was encountered. 10.2 (Complete)	13 (Drilling)
B-2	02/15/2024	+230	120	12 (Drilling) 5.5 (15 mins.)	-
B-3	02/13/2024	+230	120	8 (Drilling) 5.8 (15 mins)	-

Boring No.	Date Drilled	Approx. Boring Elevation (ft)	Boring Depth (ft)	Water Depth (ft)	Boring Cave in Depth (ft)
B-4	02/09/2024	+245	90	28 (Drilling) 26.5 (15 mins)	-

The information in this report summarizes conditions found on the dates the borings were drilled. It should be noted that our groundwater observations are short-term; groundwater depths and subsurface soil moisture contents will vary with environmental variations such as frequency and magnitude of rainfall, the time of year when construction is in progress, and the water level in nearby bodies of water such as Lake Raven itself.

4.3 Site Geology

The geologic formation exposed in the area of the proposed dams and most of the area to be covered by the resulting reservoir lakes is the Willis Formation. The formation is a mixture of clay, silt, sand, and quartz gravel up to pebble size. Some petrified wood is present and the sand is coarser grained than younger rocks. The formation is noncalcareous. The formation is mostly deeply weathered, but the amount of weathering decreases eastward. The formation can be hardened by clay and locally cemented by iron oxide. Iron oxide concretions are present. The thickness of the formation is approximately 100 feet.

A portion of the lake resulting from the proposed Walnut Creek Dam appears to also be located in the Lissie Formation. This formation is approximately 200 feet thick. It is composed of clay, silt, sand, and very minor to minor amounts of gravel. The upper part is locally calcareous with common calcium carbonate concretions and iron-manganese oxides and iron oxide. In the lower part, the gravel is slightly coarser. The lower part is noncalcareous and iron oxide concretions are more abundant.

An unnamed southwest-northeast oriented fault approximately 10 miles long crosses the area approximately 2 miles north of the northern end of the proposed lake extents (Han, 2013).

The Carlton Speed Oil and Gas field is located near the northern end of the lake that will result from the proposed Birch Creek Dam.

REFERENCES

1. Barnes, Virgil E., Project Director, Revised 1968, revised 1992, *Geologic Atlas of Texas, Beaumont Sheet, Harold Norman Fisk Memorial Edition:* The University of Texas at Austin, Bureau of Economic Geology, 1:250,000 scale.

2. Han, Xu, May 2013, Integrated Remote Sensing and Geophysical Study of the Hockley Fault in Harris and Montgomery Counties, Texas (Master's Thesis), University of Houston, Texas, Figures 1.3, 2.5.

4.4 Subsurface Variations

The information contained in this report summarizes the conditions encountered on the dates the borings were drilled. The ground water depths and subsurface soil moisture contents will vary with seasonal and environmental variations, frequency, and magnitude of rainfall and the time of year when construction is in progress.

Clay soils in the Texas Gulf Coast area typically have secondary features such as slickensides, ferrous/calcareous nodules, and contain sand/silt seams/lenses/layers/pockets. It should be noted that the information in the boring logs is based on 3-inch diameter soil samples. Samples from the borings were obtained continuously at intervals of 2 feet from the ground surface to a depth of 20 feet in the borings, then at 5 foot intervals thereafter to the boring termination depths. A detailed description of the soil secondary features may not have been obtained due to the small sample size and sampling interval between the samples. Therefore, while some of AEC's logs show the soil secondary features, it should not be assumed that the features are absent where not indicated on the logs.

5.0 ENGINEERING ANALYSIS AND RECOMMENDATIONS

As noted in Section 1.2 of this report, AEC's soil borings were performed approximately 0.9 miles downstream of the proposed earthfill dam sites. AEC reiterates that **this report is to be used for preliminary analysis and design purposes only and should not be used for final design purposes**. Additional geotechnical borings must be performed at the actual dam sites during the final design phase.

Based on AEC's discussion with B&V, the purpose of this investigation is to perform soil borings and geotechnical laboratory testing to obtain an idea of the soil types and strata as well as groundwater information that hopefully is representative of the soil and groundwater conditions at the proposed earth dam sites. This includes baseline soil parameters for each boring that can be used for preliminary analysis and design purposes, and to provide material requirements for the major components of the earth dams. AEC understands that three cross sections are under consideration for the preliminary earth dam designs. These preliminary cross sections from B&V are presented in Plates B-5 and B-6, in Appendix B, for reference.

According to the alternative cross sections provided by B&V (see Plates B-5 and B-6, in Appendix B), the earthfill dam cross sections are: (i) Alternative 1 - a homogenous dam with a vertical chimney drain and horizontal downstream drainage blanket; (ii) Alternative 2 - a exterior pervious shell and impervious core dam, with chimney drain and downstream horizontal drainage blanket, plus cutoff trench beneath the core; and (iii) Alternative 3 - a homogenous dam with a vertical soil-cement-bentonite (SCB) seepage cutoff wall, with a downstream heel horizontal drainage blanket. All three alternatives will have a H:V = 3.5:1 upstream slope (with mid-slope bench) and a H:V = 3:1 downstream slope (also with mid-slope bench). Based on the cross section elevation scale, the dam crest will be at an elevation of approximately +275 feet Mean Sea Level (MSL), and the base of the dam will be an elevation of approximately +226 feet MSL. The resulting dam height is approximately 49 feet. For Alternative 2, the bottom of the cutoff trench will be at an elevation of approximately +206.5 feet MSL. For Alternative 3, the bottom of the SCB seepage cutoff wall will be at an elevation of approximately +206.5 feet MSL.

5.1 Baseline Soil Parameters

As noted previously, Borings B-1 and B-2 were performed for the Walnut Creek Dam while Borings B-3 and B-4 were performed for the Birch Creek Dam. AEC's boring logs are presented on Plates A-3 through A-6, in Appendix A. As requested by B&V, AEC has provided baseline soil parameters for each individual boring that can be used for preliminary analysis and design of the proposed earthfill dams. The baselined soil parameters for each individual boring are presented on Plates B-1 through B-4, in Appendix B of this report.

The baseline parameters provided include soil type (by USCS classification), unit weight, and cohesion and friction angle parameters for undrained, effective stress, and total stress conditions. The baseline values are based on the results of field and laboratory test data on individual boring logs as well as AEC's experience with local soil conditions. The undrained parameters for cohesive soils (i.e., clays) are primarily based on pocket penetrometer (PP), torvane (TV), unconfined compression (UC), and unconsolidated-undrained (UU) triaxial tests. The effective stress (CD) and total stress (CU) cohesion and friction angle parameters for cohesive soils are based on consolidated-undrained (CU) triaxial tests with pore water pressure measurements. Granular (i.e., sands and gravels) soils and cohesionless (i.e., silts) soils friction angles are based on SPT blow counts taken during AEC's field exploration.

5.2 Foundation Soils

Based primarily on Borings B-2 and B-3, which were performed adjacent to Walnut Creek and Birch Creek downstream of the earth dam locations and considering a dam base elevation of approximately +226 feet MSL, it is AEC's opinion that the foundation soils beneath the earth dams should be considered stratified, consisting of alternating strata of very pervious and impervious soil layers. However, AEC notes that the soil conditions encountered in Boring B-4, which is further away from Birch Creek than Boring B-3 is, consists almost entirely of very pervious sandy soil layers. Based on the soil borings and laboratory tests, AEC considers the following soils (by USCS classification) to be pervious to very pervious: SM, SP-SM, and SC that is less pervious than SM and SP-SM, and considers the following soils to be most impervious to impervious: CH, CL. Permeability tests performed by AEC on selected samples of an impervious soil (CL) and pervious soil (SC) are presented on Table 6 in Section 3.0 of this report.

Recommendations for site preparation (such as clearing, excavation, proof-roll, subgrade preparation, etc.) at the actual dam locations should be provided based on soil borings performed within the proposed dam footprint areas during final design.

5.3 Dam Materials

Homogenous Embankment Fill: Homogenous embankment fill will be used for Alternatives 1 and 3. The embankment fill (whether imported from offsite or is already available from onsite borrow areas) should consist of ***uniform***, non-active, non-dispersive, inorganic lean clays with a LL less than 49 percent, PI between 15 and 30 percent, and more than 50 percent passing a No. 200 sieve. Material intended for use as homogeneous embankment fill shall not have clay clods with PI greater than 20, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 10. AEC recommends that the homogeneous embankment fill have a minimum permeability requirement of 1.0xE-7, or less. The homogenous embankment fill shall be placed in maximum 8 inch thick loose lifts and compacted to a minimum of 95 percent of its ASTM D 698 (i.e., Standard Proctor) maximum dry density at a moisture content between optimum and 3 percent above optimum. Backfill within 3 feet of any structures should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors.

At least one Atterberg Limits and one percent passing a No. 200 sieve test shall be performed for each 25,000 square feet (sf) of placed fill, per lift (with a minimum of one set of tests per lift), to determine whether it meets homogeneous embankment fill requirements.

Pervious Exterior Shell Fill: AEC recommends that clean sands (such as concrete sand) be used for pervious exterior shell fill purposes for Alternative 2. A recommended gradation for concrete sand is presented on Table 9. AEC notes that the gradation presented in Table 9 is for preliminary design purposes only. The gradation requirements for final design may be different, depending on available borrow material.

Table 9 - Recommended Pervious Exterior Shell Material Gradation

Sieve	Percent Passing
No. 4	100
No. 8	90 to 100
No. 16	55 to 100
No. 30	20 to 80
No. 50	10 to 45
No 100	0 to 15
No 200	0

The pervious exterior shell fill shall be placed in maximum 12 inch thick loose lifts and shall be compacted by a minimum of 2 to 6 passes of a vibratory roller.

Impervious Core and Cutoff Trench Fill: Impervious core and cutoff trench fill will be used for Alternative 2. The impervious fill (whether imported from offsite or is already available from onsite borrow areas) should consist *uniform*, non-active, non-dispersive, inorganic lean clays with a LL less than 45, PI between 15 and 30, and a percent passing a No. 200 sieve between 60 and 85. Material intended for use as impervious fill shall not have clay clods with PI greater than 30, clay clods greater than 2 inches in diameter, or contain sands/silts with PI less than 15. AEC recommends that the impervious fill have a minimum permeability requirement of 1.0xE-7, or less. The impervious fill shall be placed in maximum 8 inch thick loose lifts and compacted to a minimum of 98 percent of its ASTM D 698 (i.e., Standard Proctor) maximum dry density at a moisture content between optimum and 3 percent above optimum. Backfill within 3 feet of any structures should be placed in loose lifts no more than 4-inches thick and compacted using hand tampers, or small self-propelled compactors.

At least one Atterberg Limits and one percent passing a No. 200 sieve test shall be performed for each 25,000 square feet (sf) of placed fill, per lift (with a minimum of one set of tests per lift), to determine whether it meets impervious fill requirements.

Chimney Drain and Horizontal Drainage Blanket: The permeability of the proposed chimney drain and horizontal drainage blanket must be sufficient to provide easy seepage drainage and reduce seepage uplift forces.

The drains should be thick enough to handle total seepage flow expected both through the embankment and from the underlying foundation soils. AEC notes that the gradation presented in Table 10 is for preliminary design purposes only. The gradation requirements for final design may be different, depending on available borrow material, and the gradation shall be sufficient to prevent clogging from fine soil particles. A filter fabric may be required to help prevent clogging of the drains, if necessary.

Table 10 - Recommended Chimney Drain and Drainage Blanket Material Gradation

Sieve	Percent Passing
1”	100
3/4”	75 to 100
3/8”	50 to 100
No. 4	25 to 60
No. 8	0 to 30
No. 16	0

The drainage fill material shall be placed in maximum 12 inch thick loose lifts and shall be compacted by a minimum of 2 to 6 passes of a vibratory roller.

<u>Soil-Cement-Bentonite Slurry Seepage Cutoff Wall:</u> Based on Alternative 3, a SCB seepage cutoff wall will be installed through the dam after the embankment has been constructed. AEC recommends that the SCB slurry wall be a minimum of 4 feet thick. Based on AEC’s and considering the bottom elevation of the seepage wall will be at an elevation of approximately +206.5 feet MSL, AEC anticipates that the slurry trench excavation will generally encounter silty/clayey sands (SM/SC), along with some lean clay (CL) strata. Based on Table 8 in Section 4.2 of this report, depending on the time of year that construction takes place, groundwater is likely to be encountered within the foundation soils beneath the earthfill dams, with the likelihood of groundwater being encountered as the slurry trench excavation depth increases.

According to the US Bureau of Reclamation Design Standards No. 13, Chapter 16 “Cutoff Walls”, a SCB cutoff wall is designed with a controlled amount of soil, bentonite, and cement. The SCB cutoff wall achieves a higher strength compared to either a soil-bentonite cutoff wall or a cement-bentonite cutoff wall, but the SCB wall has slightly higher permeability compared to the other two mentioned wall types. Furthermore, the SCB wall potentially includes re-using the soil from the excavation volume as backfill, which also means less cement needs to be used for the cutoff wall construction.

For SCB slurry wall construction, the bentonite slurry trench is typically excavated first. In order to mitigate the possibility of trench sidewall collapse, **AEC recommends that the bentonite slurry head within the trench**

be maintained a minimum of 5 feet above the groundwater level within the trench; introducing the bentonite slurry from the start of the excavation is recommended. AEC recommends that only bentonite slurry be used for construction; polymer slurry should not be considered. Spoil excavated from the bentonite slurry trench should be monitored by a qualified soil technician; granular soil (such as sand and silt) and cohesive soil (such as silty/lean/fat clay) should be stockpiled separately. The soil component for the soil-cement-bentonite mixture should meet the gradation requirements presented in Plate B-7, in Appendix B, which are taken from the US Bureau of Reclamation Design Standards No. 13, Chapter 16 “Cutoff Walls”. Excavated spoils that meet the gradation requirements can be re-used as part of the soil-cement-bentonite mixture. Cohesive soils (whether excavated onsite or imported from offsite) should not be allowed for use in the soil-cement, since it will not provide a homogenous mixture. Stockpiled cohesive soils can instead be tested for re-use as embankment material for other backfill applications for this project.

7.0 RECOMMENDATIONS FOR FINAL DESIGN

Additional Soil Borings: As noted throughout this report, the recommendations in this geotechnical investigation report should be used for preliminary analysis and design purposes only. Additional soil borings must be performed within the footprint of the proposed dams to provide recommendations for the final design phase of the project.

Dispersive Soil Stabilization Tests: AEC recommends that onsite dispersive clay soils that are intended for reuse as structural fill be tested to confirm that lime-stabilization will be effective for reducing dispersive potential, and to determine the optimum lime application rate for stabilization.

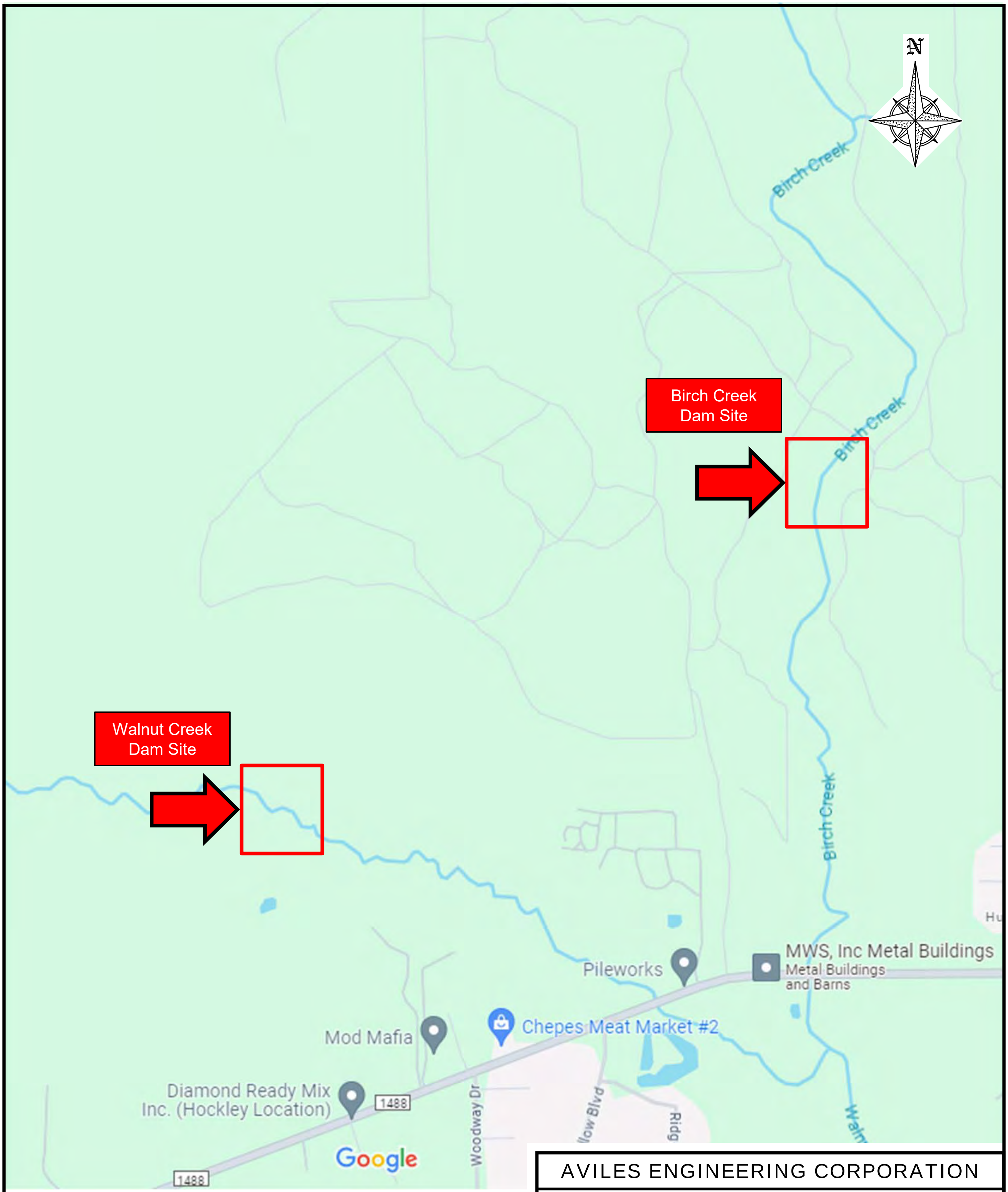
8.0 LIMITATIONS

The information contained in this report summarizes conditions found on the dates the borings were drilled. The attached boring logs are true representations of the soils encountered at the specific boring locations on the dates of drilling. Reasonable variations from the subsurface information presented in this report should be anticipated. AEC should be notified immediately if conditions encountered during construction are significantly different from those presented in this report.

This investigation was performed using the standard level of care and diligence normally practiced by recognized geotechnical engineering firms in this area, presently performing similar services under similar circumstances. This report is intended to be used in its entirety. The report has been prepared exclusively for the project and locations described in this report. If pertinent project details change or otherwise differ from those described herein, AEC should be notified immediately and retained to evaluate the effect of the changes on the recommendations presented in this report, and revise the recommendations if necessary. The recommendations presented in this report should not be used for other structures located along these alignments or similar structures located elsewhere, without additional evaluation and/or investigation.

APPENDIX A

Plate A-1	Vicinity Map
Plate A-2	Boring Location Plan
Plates A-3 to A-6	Boring Logs
Plate A-7	Key to Symbols
Plate A-8	Classification of Soils for Engineering Purposes
Plate A-9	Terms Used on Boring Logs
Plate A-10	ASTM & TXDOT Designation for Soil Laboratory Tests
Plates A-11 to A-17	Grain Size Analysis Results
Plates A-18 to A-20	Double Hydrometer Test Results
Plate A-21	Crumb Test Results
Plates A-22 to A-24	Mohr-Coulomb Diagrams (from CU Triaxial Tests)
Plates A-25 to A-28	Consolidation Test Results
Plates A-29 to A-35	Permeability Test Results
Plates A-36 and A-37	Moisture-Density Relationship - Standard Proctor Test Results

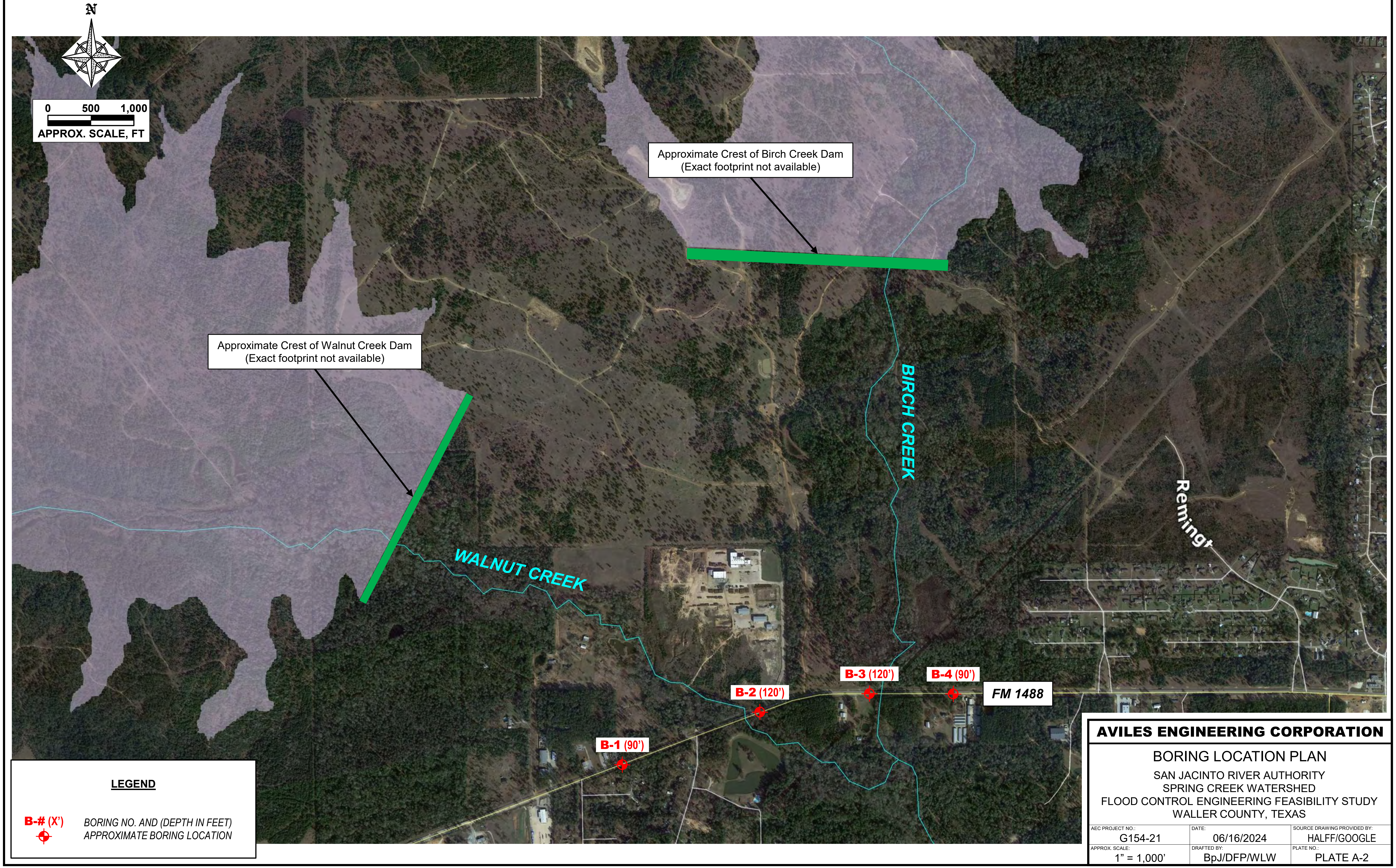


AVILES ENGINEERING CORPORATION

VICINITY MAP

SAN JACINTO RIVER AUTHORITY
SPRING CREEK WATERSHED
FLOOD CONTROL ENGINEERING FEASIBLITY STUDY
WALLER COUNTY, TEXAS

AEC PROJECT NO.: G154-21	DATE: 06/07/2024	SOURCE DRAWING PROVIDED BY: GOOGLE EARTH
APPROX. SCALE: N.T.S.	DRAFTED BY: DFP	PLATE NO.: PLATE A-1



AVILES ENGINEERING CORPORATION

BORING LOCATION PLAN

SAN JACINTO RIVER AUTHORITY
SPRING CREEK WATERSHED
FLOOD CONTROL ENGINEERING FEASIBILITY STUDY
WALLER COUNTY, TEXAS

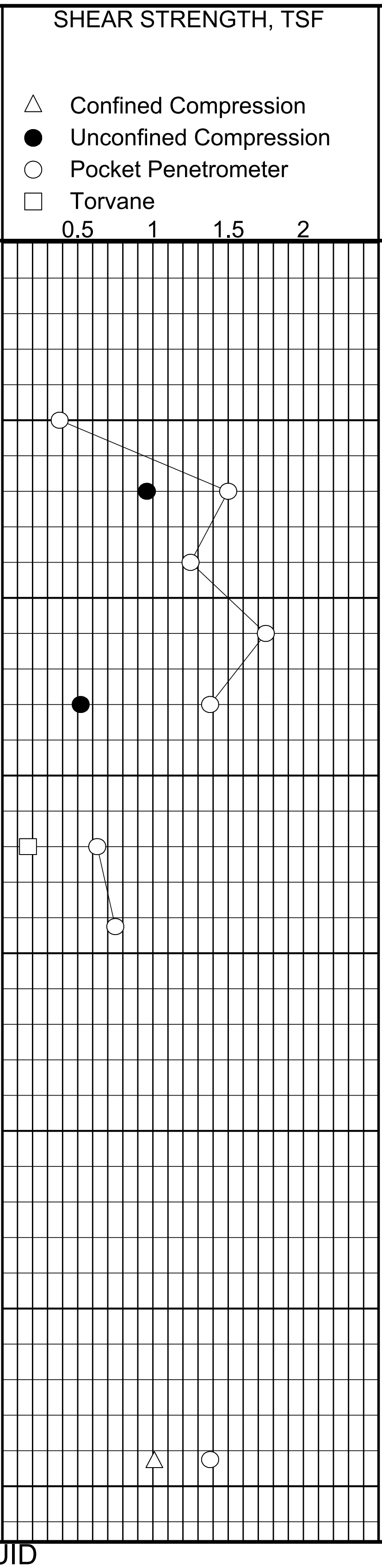
AEC PROJECT NO.: G154-21	DATE: 06/16/2024	SOURCE DRAWING PROVIDED BY: HALFF/GOOGLE
APPROX. SCALE: 1" = 1,000'	DRAFTED BY: BpJ/DFP/WLW	PLATE NO.: PLATE A-2

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-1

DATE 2/19/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	DESCRIPTION Approximate Surface Elevation (feet): 250	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 3" asphalt							
	Base: 12" sand and gravel							
	Brown Silty Sand (SM)		6					
	-tan 2'-4'		6		25.0	NP	NP	NP
5	Firm to very stiff, brown Sandy Lean Clay (CL)		9					
	-reddish brown and light gray, with slickensides and sand pockets 6'-8'		12	113.7				
	Medium dense, reddish brown and light gray Clayey Sand (SC), wet		13	115.4	28.3	26	13	13
10	▼							
			15					
	-light gray and tan 12'-16', with ferrous nodules 12'-14' -boring cave-in at 13' during drilling		12	114.8	31.4	30	12	18
15		27	13		17.6			
	-tan 16'-20'		15					
	-with lean clay layers 18'-20'	27	13		17.0	44	17	27
20								
	Medium dense, tan and light gray Poorly Graded Sand with Silt (SP-SM), wet -with lean clay pockets 23'-25'	19	16					
25								
	-tan 28'-30'	13	24		11.1			
30								
	Stiff to very stiff, tan Lean Clay (CL), with sand pockets	14	26	94.1				
35								



BORING DRILLED TO 20 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT ** FEET WHILE DRILLING ▽

WATER LEVEL AT 10.2 FEET AFTER Complete ▼

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

PROJECT NO. G154-21

PLATE A-3

PROJECT: SJRA Spring Creek Watershed Feasibility Study

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING B-1

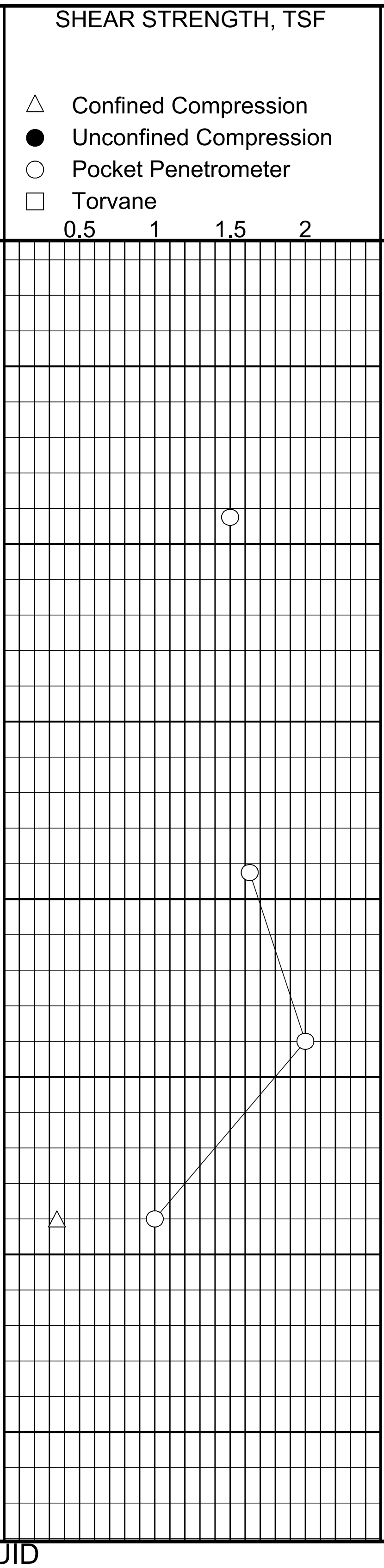
DATE 2/19/24

TYPE 4" Dry Auger / Wet Rotary

LOCATION See Boring Location Plan

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Lean Clay (CL) (Cont.)							
Medium dense, tan Clayey Sand (SC), wet, with lean clay pockets	12	22		30.2	26	13	13
Very stiff, tan and light gray Lean Clay (CL)	21	32					
Medium dense, Poorly Graded Sand with Silt (SP-SM), wet, with lean clay pockets	29	27		5.9			
Firm to very stiff, tan and light gray Sandy Fat Clay (CH)	15	34					
-with sand pockets and silt seams 58'-60'		24	90.2	66.3	60	22	38
-light gray 63'-65'		18	109.4				
Medium dense to dense, light gray and tan Silty Sand (SM), wet -with lean clay pockets 68'-70'	34	21		16.3			

DEPTH IN FEET: 40, 45, 50, 55, 60, 65, 70



BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT ** FEET WHILE DRILLING

WATER LEVEL AT **10.2** FEET AFTER **Complete**

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

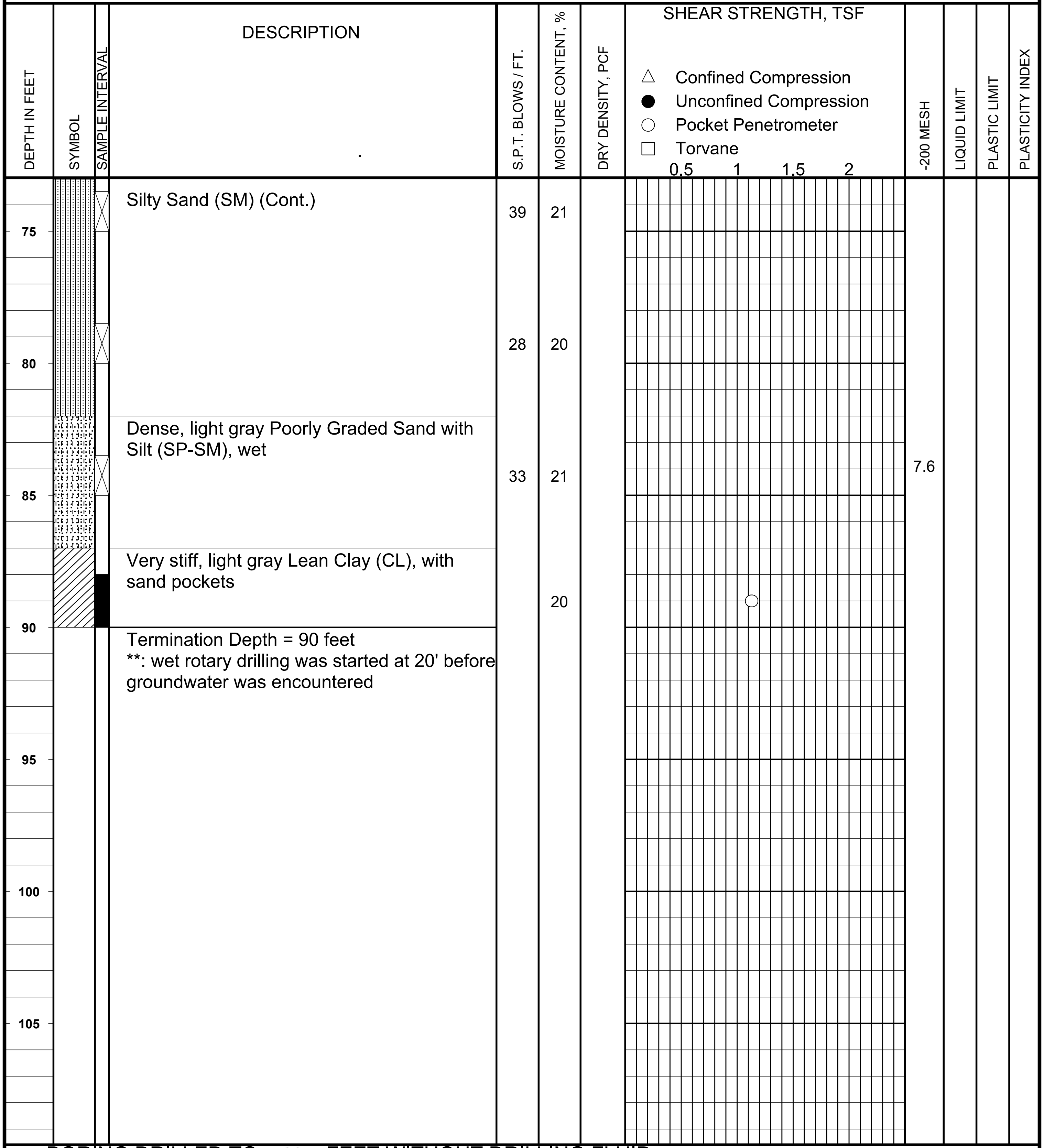
PROJECT NO. G154-21

PROJECT: SJRA Spring Creek Watershed Feasibility Study

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING B-1

DATE 2/19/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan



DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	Silty Sand (SM) (Cont.)	39	21					
		28	20					
	Dense, light gray Poorly Graded Sand with Silt (SP-SM), wet	33	21		7.6			
	Very stiff, light gray Lean Clay (CL), with sand pockets		20					
90	Termination Depth = 90 feet **: wet rotary drilling was started at 20' before groundwater was encountered							

BORING DRILLED TO **20** FEET WITHOUT DRILLING FLUID

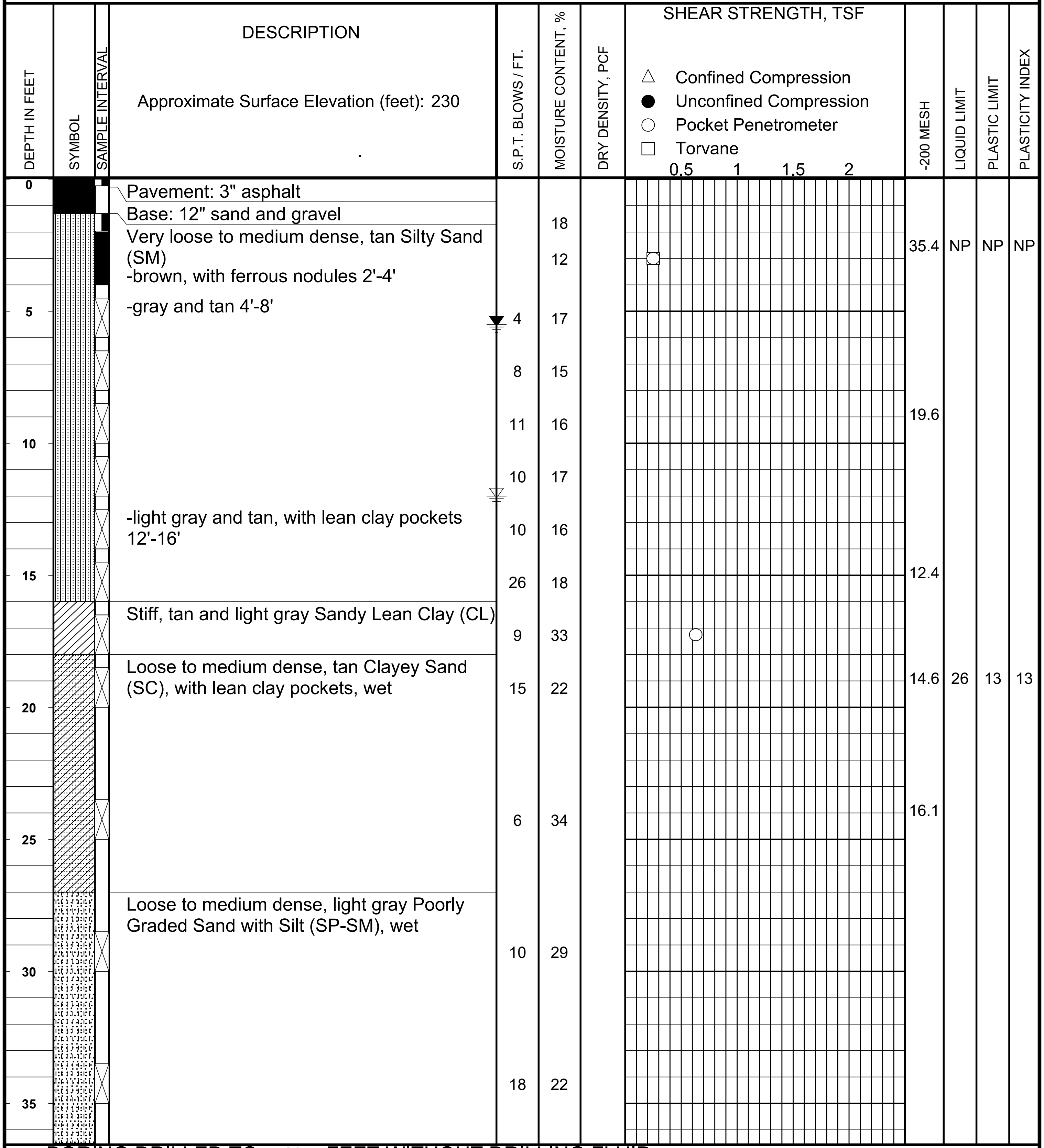
WATER ENCOUNTERED AT ** FEET WHILE DRILLING

WATER LEVEL AT **10.2** FEET AFTER **Complete**

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study
BORING B-2
DATE 2/15/24
TYPE 4" Dry Auger / Wet Rotary
LOCATION See Boring Location Plan



DESCRIPTION

Approximate Surface Elevation (feet): 230

DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 3" asphalt							
Base: 12" sand and gravel		18					
Very loose to medium dense, tan Silty Sand (SM)				35.4	NP	NP	NP
-brown, with ferrous nodules 2'-4'		12					
-gray and tan 4'-8'	4	17					
	8	15					
				19.6			
	11	16					
	10	17					
-light gray and tan, with lean clay pockets 12'-16'	10	16					
				12.4			
	26	18					
Stiff, tan and light gray Sandy Lean Clay (CL)	9	33					
Loose to medium dense, tan Clayey Sand (SC), with lean clay pockets, wet	15	22		14.6	26	13	13
				16.1			
	6	34					
Loose to medium dense, light gray Poorly Graded Sand with Silt (SP-SM), wet							
	10	29					
	18	22					

BORING DRILLED TO 12 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 12 FEET WHILE DRILLING
WATER LEVEL AT 5.5 FEET AFTER 15 mins
DRILLED BY Van & Sons DRAFTED BY EN LOGGED BY DN

PROJECT NO. G154-21

PROJECT: SJRA Spring Creek Watershed Feasibility Study

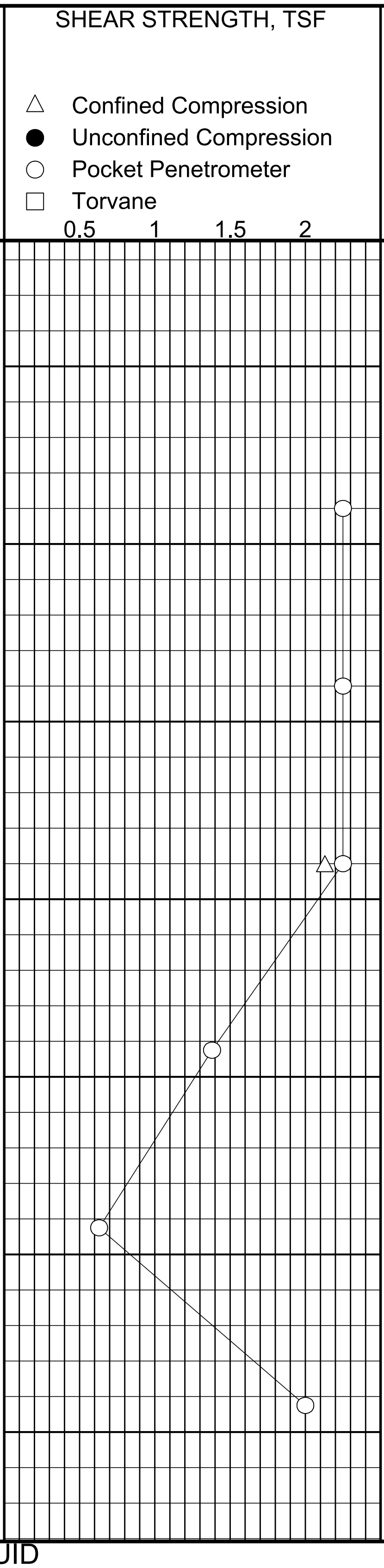
BORING B-2

DATE 2/15/24

TYPE 4" Dry Auger / Wet Rotary

LOCATION See Boring Location Plan

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
			Poorly Graded Sand with Silt (SP-SM) (Cont.) -tan 38'-40'	19	25		11.0			
40										
			Hard, light gray Sandy Fat Clay (CH), with calcareous and ferrous nodules		23	101.7	66.8	58	23	35
45										
			Stiff to hard, light gray and tan Sandy Lean Clay (CL), with calcareous nodules		21					
50										
			-gray 53'-55'		17	112.2				
55										
				26	21		68.9	41	18	23
60										
				19	24					
65										
				26	18					
70										
			Medium dense, light gray Silty Sand (SM), wet, with lean clay pockets							



BORING DRILLED TO 12 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 12 FEET WHILE DRILLING

WATER LEVEL AT 5.5 FEET AFTER 15 mins

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

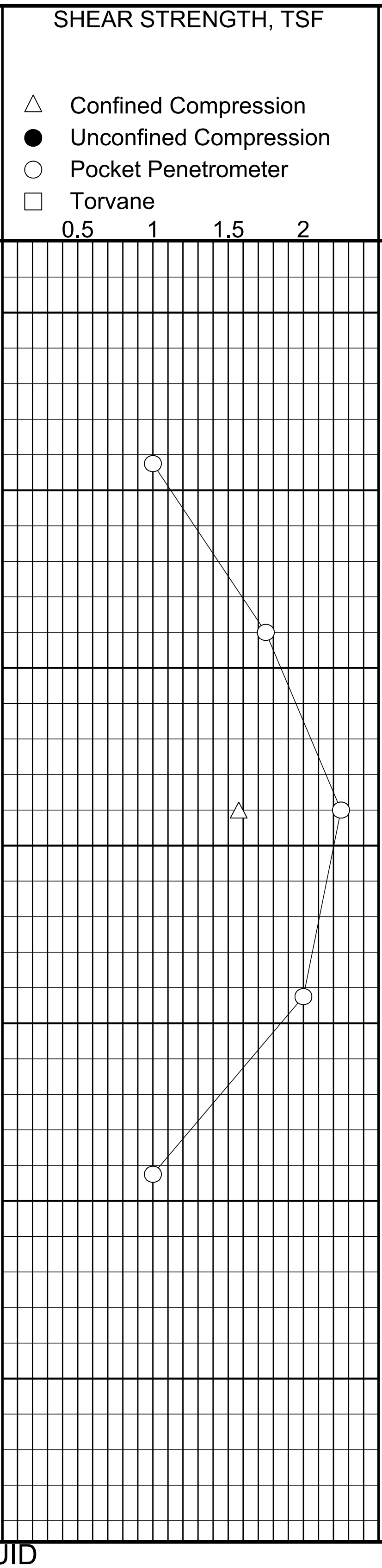
PROJECT NO. G154-21

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-2

DATE 2/15/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	Silty Sand (SM) (Cont.)	11	26		37.2			
75								
	Stiff to hard, light gray Sandy Lean Clay (CL) -with calcareous nodules 78'-80'	31	21					
80								
	-with sand pockets 83'-85'		19	108.5	63.4	32	16	16
85								
			18	111.0				
90								
	-light gray and tan 93'-95'	27	20					
95								
	Dense to very dense, light gray Silty Sand (SM), wet -with lean clay layers 98'-100'	42	16		18.9	18	16	2
100								
		59	22					
105								
	-light gray and tan 108'-110'	53	22					



BORING DRILLED TO 12 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 12 FEET WHILE DRILLING ▽
WATER LEVEL AT 5.5 FEET AFTER 15 mins ▼
DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-2

DATE 2/15/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF △ Confined Compression ● Unconfined Compression ○ Pocket Penetrometer ☐ Torvane 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
110			Silty Sand (SM) (Cont.)								
			Dense to very dense, light gray Poorly Graded Sand with Silt (SP-SM), wet	45	22			6.7			
115											
				51	22						
120			Termination Depth = 120 feet								
125											
130											
135											
140											
145											

BORING DRILLED TO **12** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **12** FEET WHILE DRILLING ▽

WATER LEVEL AT **5.5** FEET AFTER **15 mins** ▼

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

PROJECT NO. G154-21

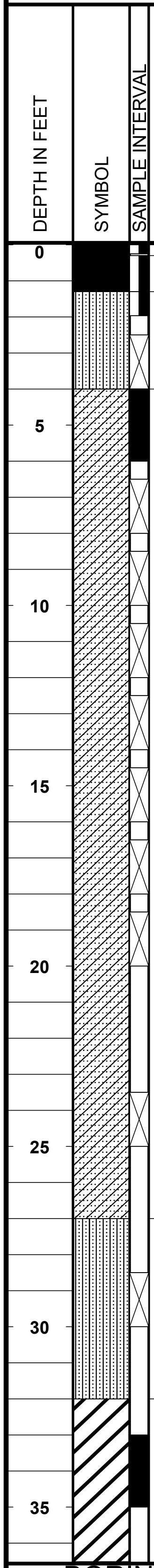
PLATE A-4

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

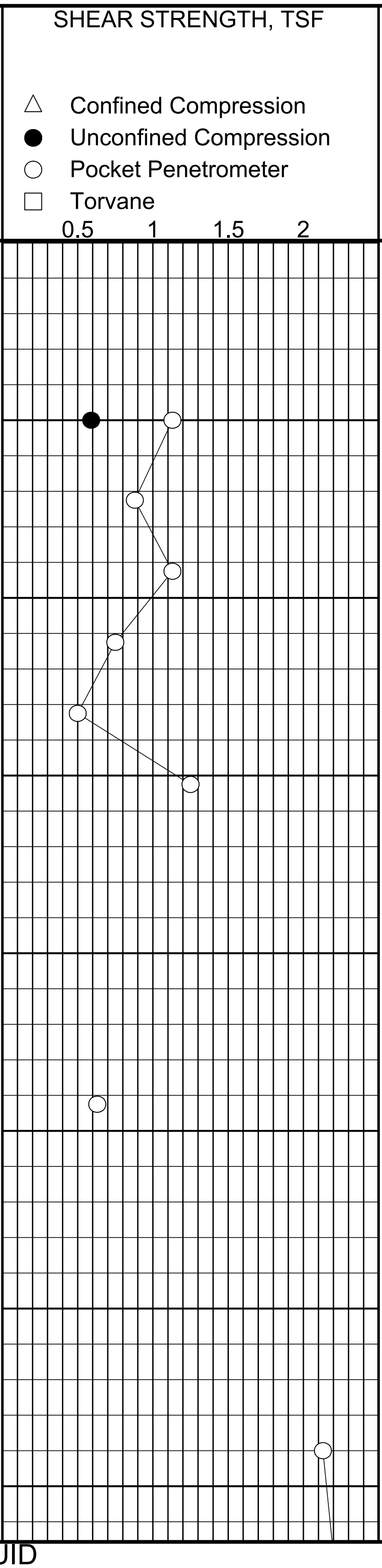
PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-3

DATE 2/13/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

Approximate Surface Elevation (feet): 230



DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
Pavement: 3" asphalt							
Base: 12" sand and gravel							
Loose, brown Silty Sand (SM), with lean clay pockets -with calcareous nodules 0'-2'		14					
				29.0			
	9	9					
Very loose to medium dense, tan and light gray Clayey Sand (SC)				32.9	23	15	8
		13	117.8				
	10	15					
				25.0			
	10	14					
				38.2	23	14	9
	11	18					
-light gray 12'-14'	11	19					
	4	24					
				35.1			
	3	31					
	3	30					
				25.3	52	16	36
	7	32					
Very loose, light gray Silty Sand (SM), with lean clay pockets	3	23					
Very stiff to hard, light gray Fat Clay with Sand (CH), with ferrous nodules				83.4	64	24	40
		27	95.5				



BORING DRILLED TO 8 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 8 FEET WHILE DRILLING ▽

WATER LEVEL AT 5.8 FEET AFTER 15 mins ▼

DRILLED BY Van & Sons DRAFTED BY EN LOGGED BY DN

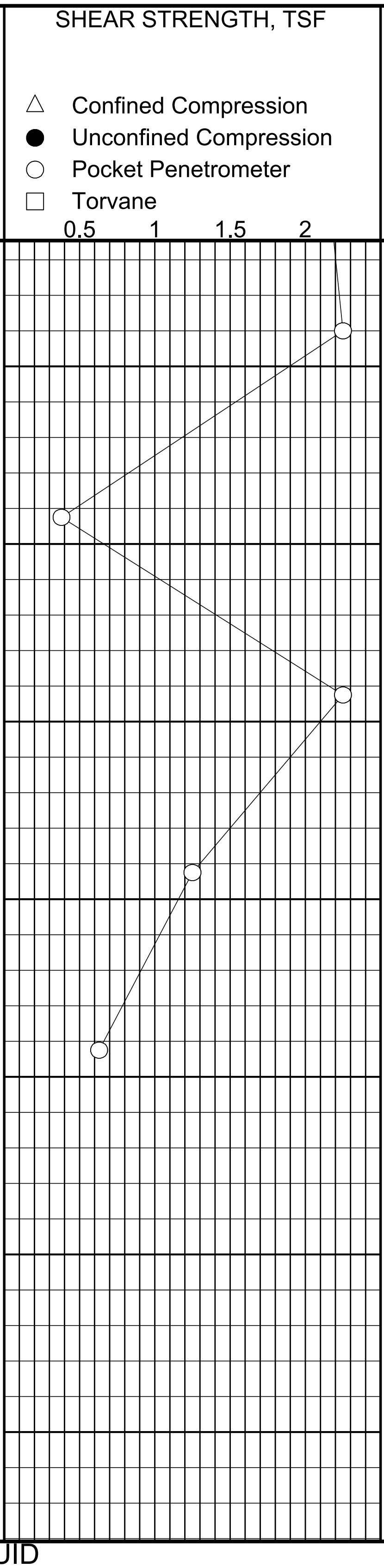
PROJECT: SJRA Spring Creek Watershed Feasibility Study

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

BORING B-3

DATE 2/13/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
40	Fat Clay (CH) (Cont.) -light gray and tan, with calcareous nodules 38'-40'		21					
45	Medium dense, light gray Silty Sand (SM), wet, with lean clay layers	28	26					
50	Very soft to hard, light gray Sandy Lean Clay (CL) -with calcareous nodules 48'-55'	12	19		67.0	47	15	32
55		23	17					
60	-light gray and tan 58'-60'	1	27		60.8	37	14	23
65	Medium dense to dense, light gray and tan Poorly Graded Sand with Silt (SP-SM), wet, with lean clay pockets	17	20					
70	-light gray 68'-80'	35	23		6.5			



BORING DRILLED TO 8 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 8 FEET WHILE DRILLING

WATER LEVEL AT 5.8 FEET AFTER 15 mins

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

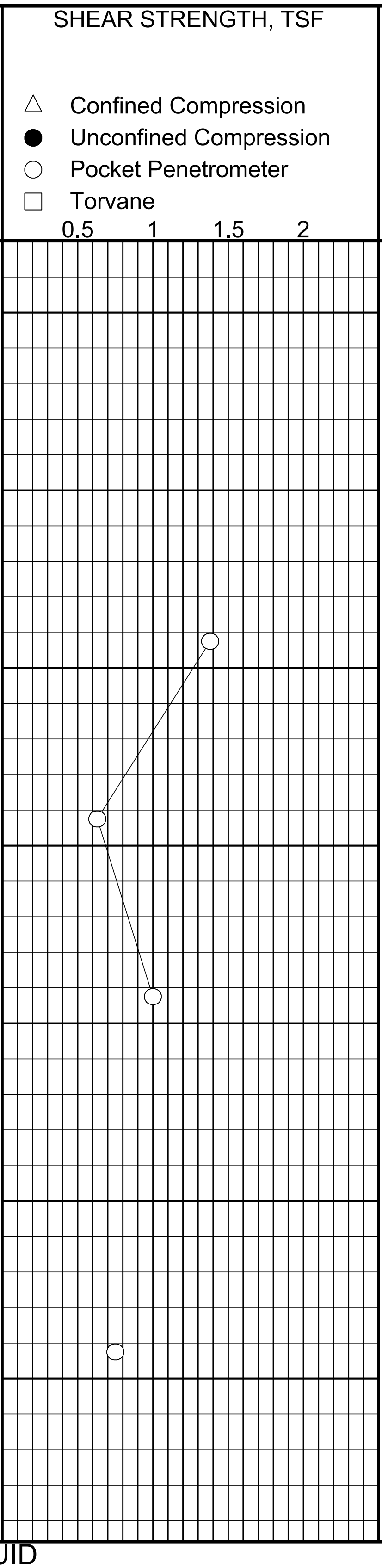
PROJECT NO. G154-21

ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-3

DATE 2/13/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
	Poorly Graded Sand with Silt (SP-SM) (Cont.)	39	22					
75								
		37	22		6.2			
80								
	Medium dense, light gray Clayey Sand (SC), wet							
		28	15					
85								
		28	21		24.5	37	14	23
90								
		27	16					
95								
	Very dense, tan Poorly Graded Sand with Silt (SP-SM), wet							
		55	20		8.3			
100								
	Medium dense to dense, light gray Clayey Sand (SC), wet							
		28	19					
105								
		39	25		29.4	23	11	12



BORING DRILLED TO 8 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 8 FEET WHILE DRILLING

WATER LEVEL AT 5.8 FEET AFTER 15 mins

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **DN**

PROJECT NO. G154-21

PLATE A-5

AVILES
ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-3

DATE 2/13/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF △ Confined Compression ● Unconfined Compression ○ Pocket Penetrometer □ Torvane 0.5 1 1.5 2	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
110			Clayey Sand (SC) (Cont.)								
			Very dense, light gray Silty Sand (SM), wet, with lean clay pockets	66	20						
115											
			Very stiff, light gray Silty Clay with Sand (CL-ML)	21	22						
120			Termination Depth = 120 feet								
125											
130											
135											
140											
145											

BORING DRILLED TO 8 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 8 FEET WHILE DRILLING ▽

WATER LEVEL AT 5.8 FEET AFTER 15 mins ▼

DRILLED BY Van & Sons DRAFTED BY EN LOGGED BY DN

PROJECT NO. G154-21

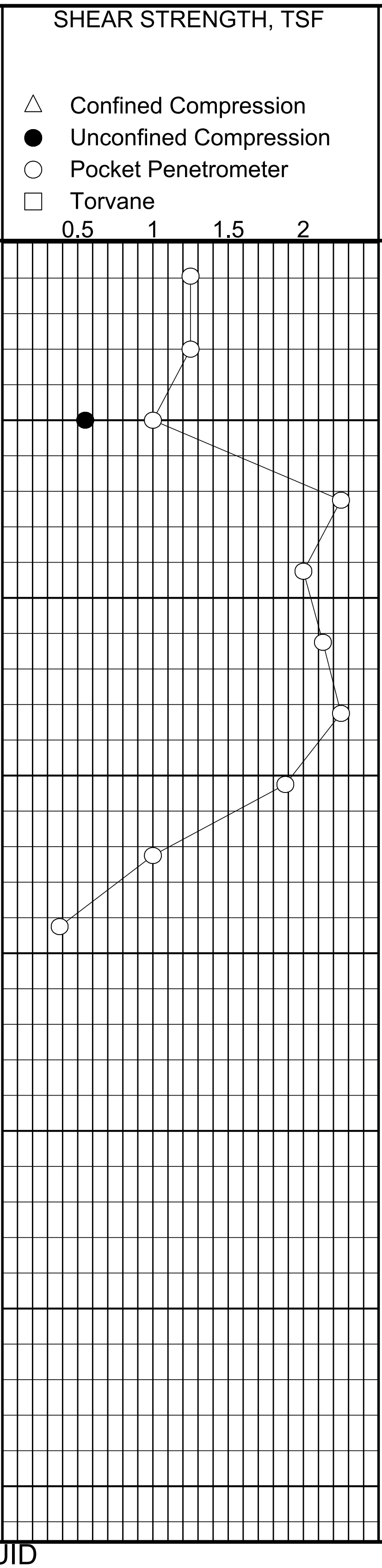
PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-4

DATE 2/9/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

Approximate Surface Elevation (feet): 245

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0	Pavement: 7" asphalt							
	Base: 4" sand and iron ore		15					
	Medium dense, brown Clayey Sand (SC) -reddish brown and light gray 2'-6', with ferrous nodules 2'-4'				45.2	38	14	24
			15					
5			13	119.2				
	-light gray and tan 6'-8'	28	11					
	Medium dense to dense, light gray and tan Silty Clayey Sand (SC-SM)				23.2	19	14	5
		35	11					
10								
		32	10					
		20	15					
15		19	11		24.0	23	19	4
		19	14					
					17.1			
		19	10					
20								
	Medium dense, tan Poorly Graded Sand with Silt (SP-SM) -with lean clay pockets 23'-25'	16	9					
25								
	-wet, tan and light gray 28'-40'	13	22					
30								
					8.6			
		27	20					
35								

SHEAR STRENGTH, TSF



BORING DRILLED TO 30 FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT 28 FEET WHILE DRILLING ▽

WATER LEVEL AT 26.5 FEET AFTER 15 mins ▼

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **JH**

AVILES ENGINEERING CORP. GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study

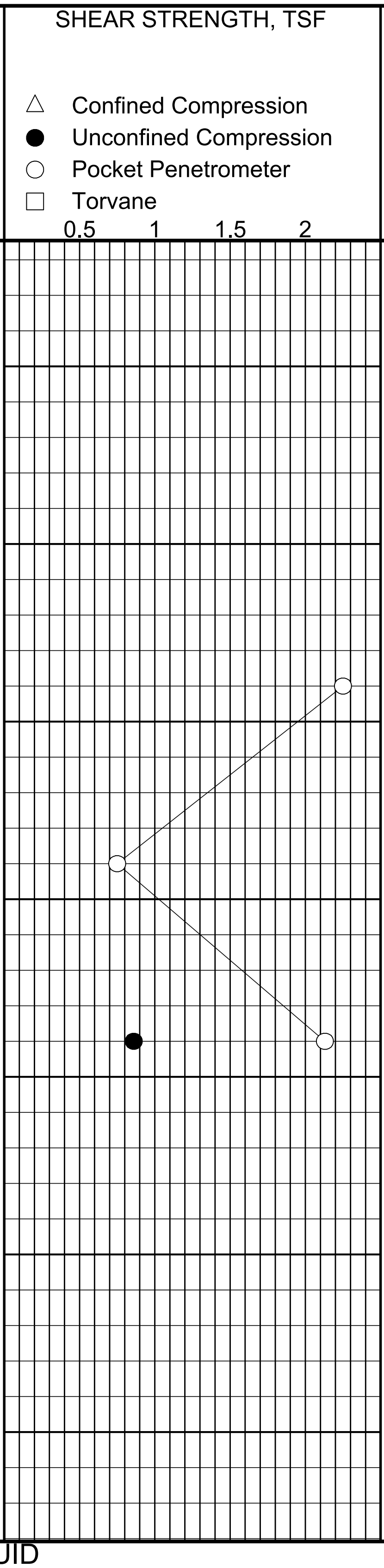
BORING B-4

DATE 2/9/24

TYPE 4" Dry Auger / Wet Rotary

LOCATION See Boring Location Plan

DEPTH IN FEET	SYMBOL	SAMPLE INTERVAL	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
40			Poorly Graded Sand with Silt (SP-SM) (Cont.) -with lean clay pockets 38'-45'	14	28						
45				29	34						
50			Hard, light gray Fat Clay (CH), with slickensides		26	97.6			66	22	44
55			Light Gray Clayey Sand (SC)		18			25.9	28	13	15
60			-light gray and tan 58'-60'		15	117.6					
65			Dense, tan Poorly Graded Sand with Silt (SP-SM)	37	21			9.7			
70			-tan and light gray 68'-70'	41	20						



BORING DRILLED TO **30** FEET WITHOUT DRILLING FLUID

WATER ENCOUNTERED AT **28** FEET WHILE DRILLING ▽

WATER LEVEL AT **26.5** FEET AFTER **15 mins** ▼

DRILLED BY **Van & Sons** DRAFTED BY **EN** LOGGED BY **JH**

PROJECT NO. G154-21

AVILES ENGINEERING CORP.
GEOTECHNICAL ENGINEERS

PROJECT: SJRA Spring Creek Watershed Feasibility Study BORING B-4

DATE 2/9/24 TYPE 4" Dry Auger / Wet Rotary LOCATION See Boring Location Plan

SHEAR STRENGTH, TSF
△ Confined Compression
● Unconfined Compression
○ Pocket Penetrometer
☐ Torvane
0.5 1 1.5 2

DEPTH IN FEET	DESCRIPTION	S.P.T. BLOWS / FT.	MOISTURE CONTENT, %	DRY DENSITY, PCF	SHEAR STRENGTH, TSF	-200 MESH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
75	Poorly Graded Sand with Silt (SP-SM) (Cont.)	40	21						
80	Dense to very dense, tan Poorly Graded Sand (SP)	31	21			4.6			
85	-with fat clay pockets 83'-85'	52	29						
90	-light gray and tan, with lean clay pockets 88'-90'	49	20						
	Termination Depth = 90 feet								
95									
100									
105									

BORING DRILLED TO 30 FEET WITHOUT DRILLING FLUID
WATER ENCOUNTERED AT 28 FEET WHILE DRILLING ▽
WATER LEVEL AT 26.5 FEET AFTER 15 mins ▼
DRILLED BY Van & Sons DRAFTED BY EN LOGGED BY JH

PROJECT NO. **G154-21**

KEY TO SYMBOLS

Strata symbols

Symbol	Description
■	Paving
▥	Silty sand
▨	Low plasticity clay
▨	Clayey sand
░	Poorly graded sand with silt
▨	High plasticity clay
▨	Silty low plasticity clay
▩	Silty clayey sand
░	Poorly graded sand

Misc. Symbols

Symbol	Description
▼	Subsequent water table depth
□	Torvane
○	Pocket Penetrometer
●	Unconfined Compression
△	Confined Compression
▽	Water table depth during drilling

Soil Samplers

Symbol	Description
◨	Auger
■	Undisturbed thin wall Shelby tube
☒	Standard penetration test

AVILES
ENGINEERING CORP.

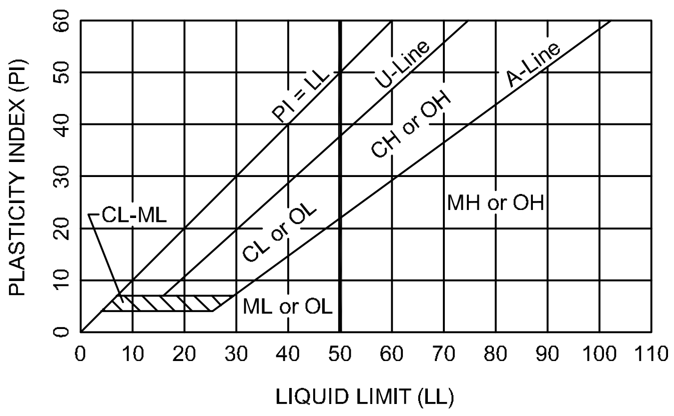
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation D-2487

MAJOR DIVISIONS				GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (Less than 50% of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)		GW	Well-graded gravel, well-graded gravel with sand
				GP	Poorly-graded gravel, poorly-graded gravel with sand
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	GM	Silty gravel, silty gravel with sand
			Limits plot above "A" line & hatched zone on plasticity chart	GC	Clayey gravel, clayey gravel with sand
	SANDS (50% or more of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 sieve)		SW	Well-graded sand, well-graded sand with gravel
				SP	Poorly-graded sand, poorly-graded sand with gravel
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart	SM	Silty sand, silty sand with gravel
			Limits plot above "A" line & hatched zone on plasticity chart	SC	Clayey sand, clayey sand with gravel
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)		SILTS AND CLAYS (Liquid Limit Less Than 50%)		ML	Silt, silt with sand, silt with gravel, sandy silt, gravelly silt
				CL	Lean clay, lean clay with sand, lean clay with gravel, sandy lean clay, gravelly lean clay
				OL	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt
		SILTS AND CLAYS (Liquid Limit 50% or More)		MH	Elastic silt, elastic silt with sand, sandy elastic silt, gravelly elastic silt
				CH	Fat clay, fat clay with sand, fat clay with gravel, sandy fat clay, gravelly fat clay
				OH	Organic clay, organic clay with sand, sandy organic clay, organic silt, sandy organic silt

NOTE: Coarse soils between 5% and 12% passing the No. 200 sieve and fine-grained soils with limits plotting in the hatched zone of the plasticity chart are to have dual symbols.

PLASTICITY CHART



Equation of A-Line: Horizontal at PI=4 to LL=25.5, then PI=0.73(LL-20)

Equation of U-Line: Vertical at LL=16 to PI=7, then PI=0.9(LL-8)

DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None	0 - 4
Slight	5 - 10
Medium	11 - 20
High	21 - 40
Very High	>40

SOIL SYMBOLS

⊠	Fill
▨	Clay (CH)
▨	Clay (CL)
⁙	Sand
▥	Silt

TERMS USED ON BORING LOGS

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

6" 3" 3/4" #4 #10 #40 #200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002

SOIL GRAIN SIZE IN MILLIMETERS

STRENGTH OF COHESIVE SOILS

Consistency	Undrained Shear Strength, Kips per Sq. ft.	SPT Blowcount
Very Soft	less than 0.25	< 2 bpf
Soft	0.25 to 0.50	2-4 bpf
Firm	0.50 to 1.00	4-8 bpf
Stiff	1.00 to 2.00	8-16 bpf
Very Stiff	2.00 to 4.00	16-32 bpf
Hard	greater than 4.00	>32 bpf

RELATIVE DENSITY OF COHESIONLESS SOILS FROM STANDARD PENETRATION TEST

Very Loose	<4 bpf
Loose	5-10 bpf
Medium Dense	11-30 bpf
Dense	31-50 bpf
Very Dense	>50 bpf

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows per Foot	Description
25	25 blows driving sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows driving sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows driving sampler 3 inches, during initial 6-inches seating interval.

NOTE: To avoid change to sampling tools, driving is limited to 50 blows during or after seating interval.

DRY STRENGTH ASTM D2488

None	Dry specimen crumbles into powder with mere pressure of handling
Low	Dry specimen crumbles into powder with some finger pressure
Medium	Dry specimen breaks into pieces or crumbles with considerable pressure
High	Dry specimen cannot be broken with finger pressure, it can be broken between thumb and hard surface
Very High	Dry specimen cannot be broken between thumb and hard surface

MOISTURE CONDITION ASTM D2488

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

SOIL STRUCTURE

Slickensided	Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the easiness of breaking along these planes.
Fissured	Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Friable	Crumbly, can be easily crushed with light pressure.
Blocky	Clays that have a block-like or polyhedral structure.
Pocket	Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting	Inclusion less than 1/8 inch thick extending through the sample.
Seam	Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer	Inclusion greater than 3 inches thick extending through the sample.
Laminated	Soil sample composed of alternating partings or seams of different soil types.
Interlayered	Soil sample composed of alternating layers of different soil types.
Intermixed	Soil sample composed of pockets of different soil types and layered or laminated structure is not evident.
Calcareous	Having appreciable quantities of calcium material.

ASTM & TXDOT DESIGNATION FOR SOIL LABORATORY TESTS

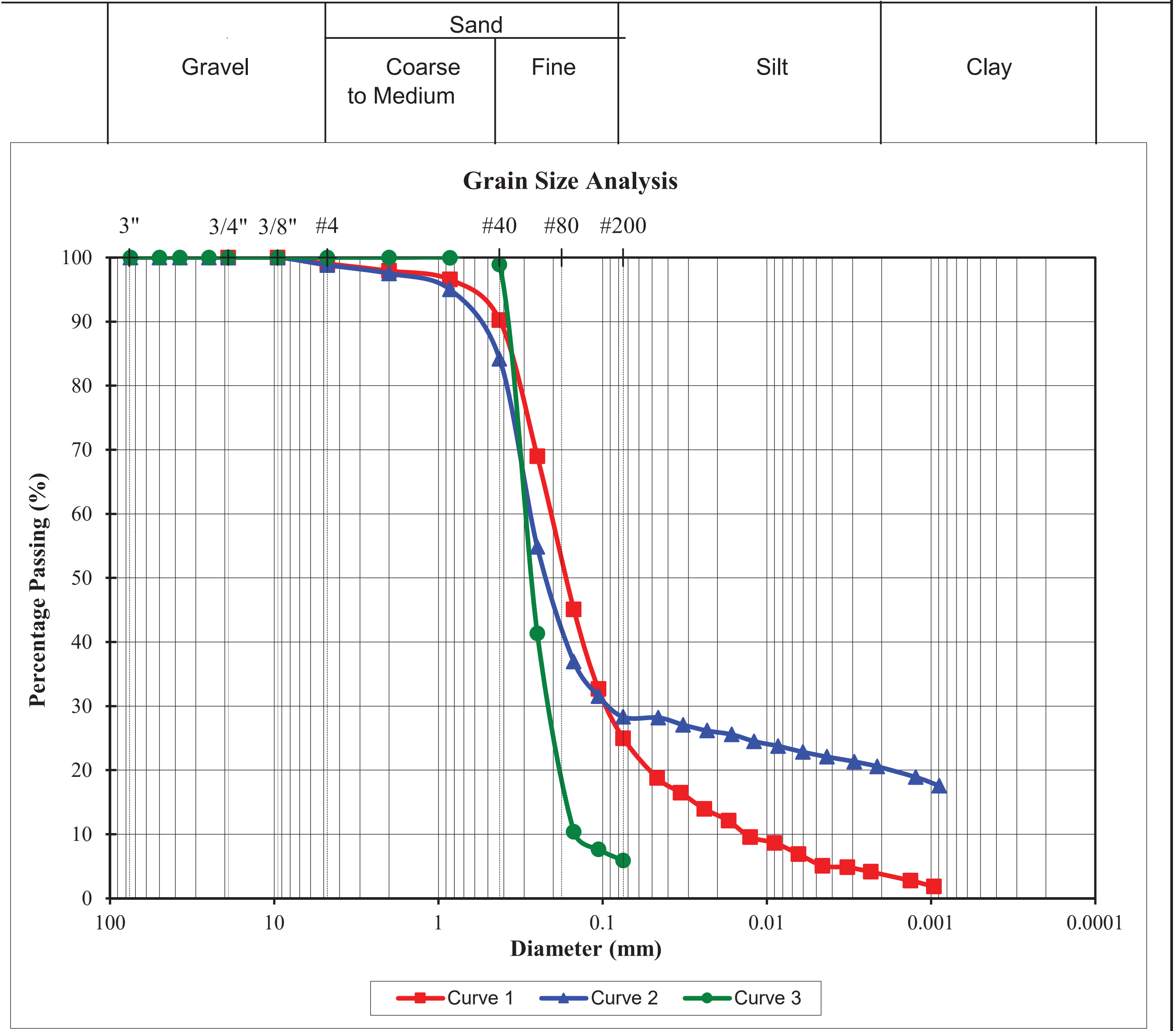
SOIL TEST	ASTM TEST DESIGNATION	TXDOT TEST DESIGNATION
Unified Soil Classification System	D 2487	Tex-142-E
Moisture Content	D 2216	Tex-103-E
Specific Gravity	D 854	Tex-108-E
Sieve Analysis	D 6913	Tex-110-E (Part 1)
Hydrometer Analysis	D 7928	Tex-110-E (Part 2)
Minus No. 200 Sieve	D 1140	Tex-111-E
Liquid Limit	D 4318	Tex-104-E
Plastic Limit	D 4318	Tex-105-E
Standard Proctor Compaction	D 698	Tex-114-E
Modified Proctor Compaction	D 1557	Tex-113-E
California Bearing Ratio	D 1883	-
Swell	D 4546	-
Consolidation	D 2435	-
Unconfined Compression	D 2166	-
Unconsolidated-Undrained Triaxial	D 2850	Tex-118-E
Consolidated-Undrained Triaxial	D 4767	Tex-131-E
Permeability (constant head)	D 5084	-
Pinhole	D 4647	-
Crumb	D 6572	-
Double Hydrometer	D 4221	-
pH of Soil	D 4972	Tex-128-E
Soil Suction	D 5298	-
Soil Sulfate	C 1580	Tex-145-E
Organics	D 2974	Tex-148-E

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913) & HYDROMETER (ASTM D 7928)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-1	2-4	Silty Sand (SM)	0.17	15.86	3.19
2	B-1	8-10	Clayey Sand (SC)	0.22	N/A	N/A
3	B-1	48-50	Poorly Graded Sand with Silt (SP-SM)	0.28	2.13	1.03

AVILES ENGINEERING CORPORATION

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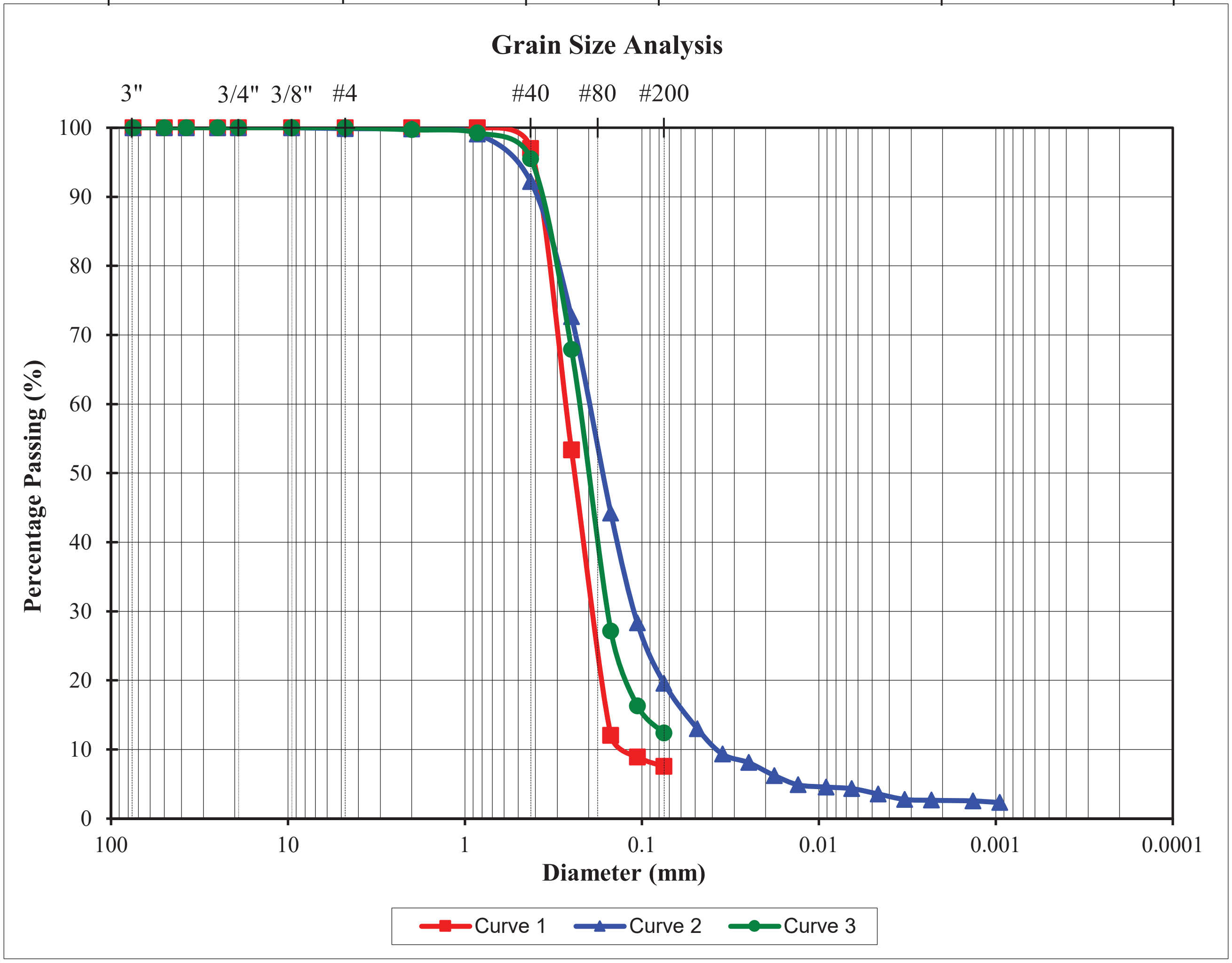
GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913) & HYDROMETER (ASTM D 7928)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas

	Gravel	Sand		Silt	Clay	
		Coarse to Medium	Fine			



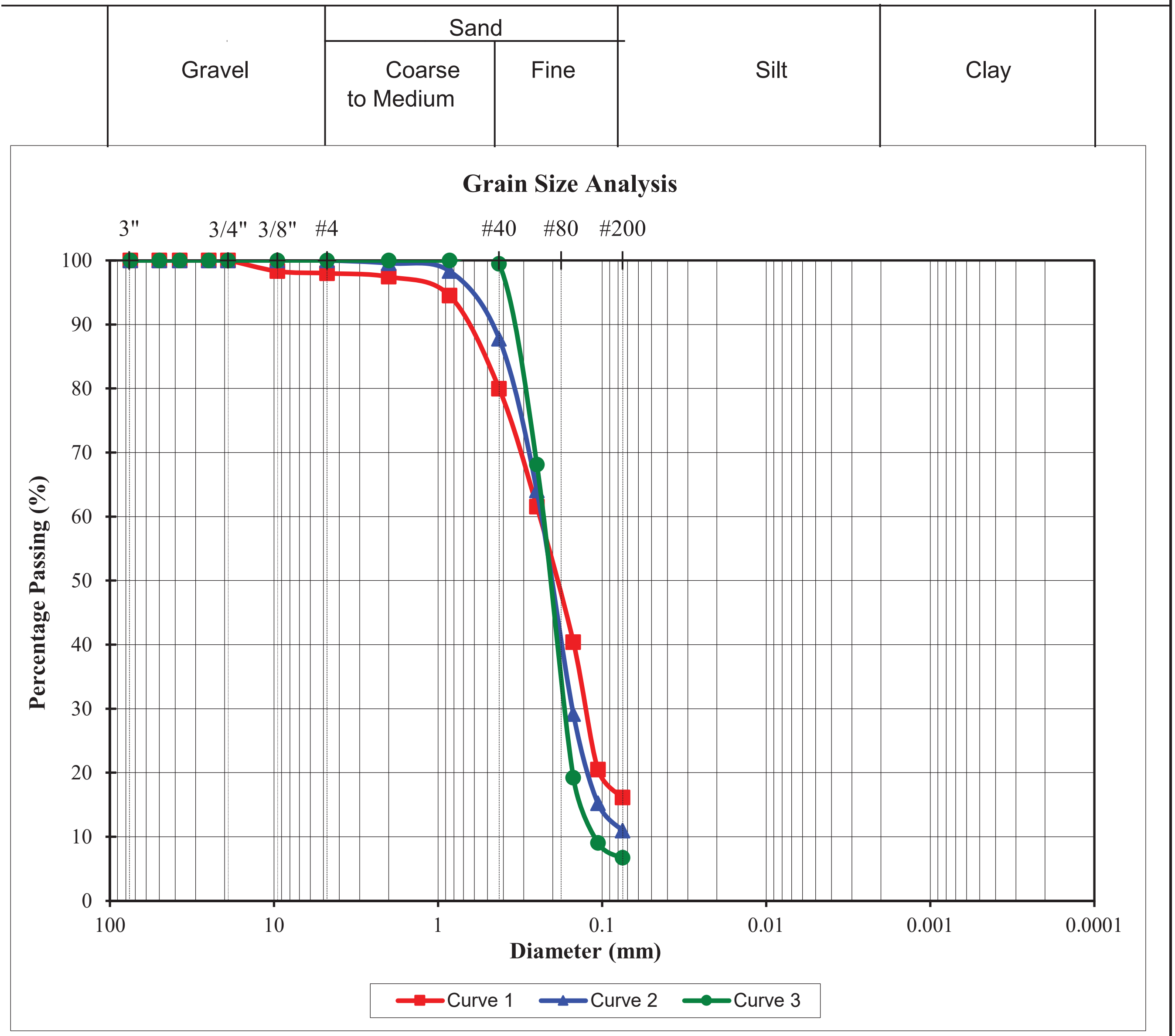
Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-1	83-85	Poorly Graded Sand with Silt (SP-SM)	0.24	2.28	1.12
2	B-2	8-10	Silty Sand (SM)	0.17	5.48	1.59
3	B-2	14-16	Silty Sand (SM)	0.21	N/A	N/A

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-2	23-25	Clayey Sand (SC)	0.20	N/A	N/A
2	B-2	38-40	Poorly Graded Sand with Silt (SP-SM)	0.21	N/A	N/A
3	B-2	113-115	Poorly Graded Sand with Silt (SP-SM)	0.21	2.12	1.15

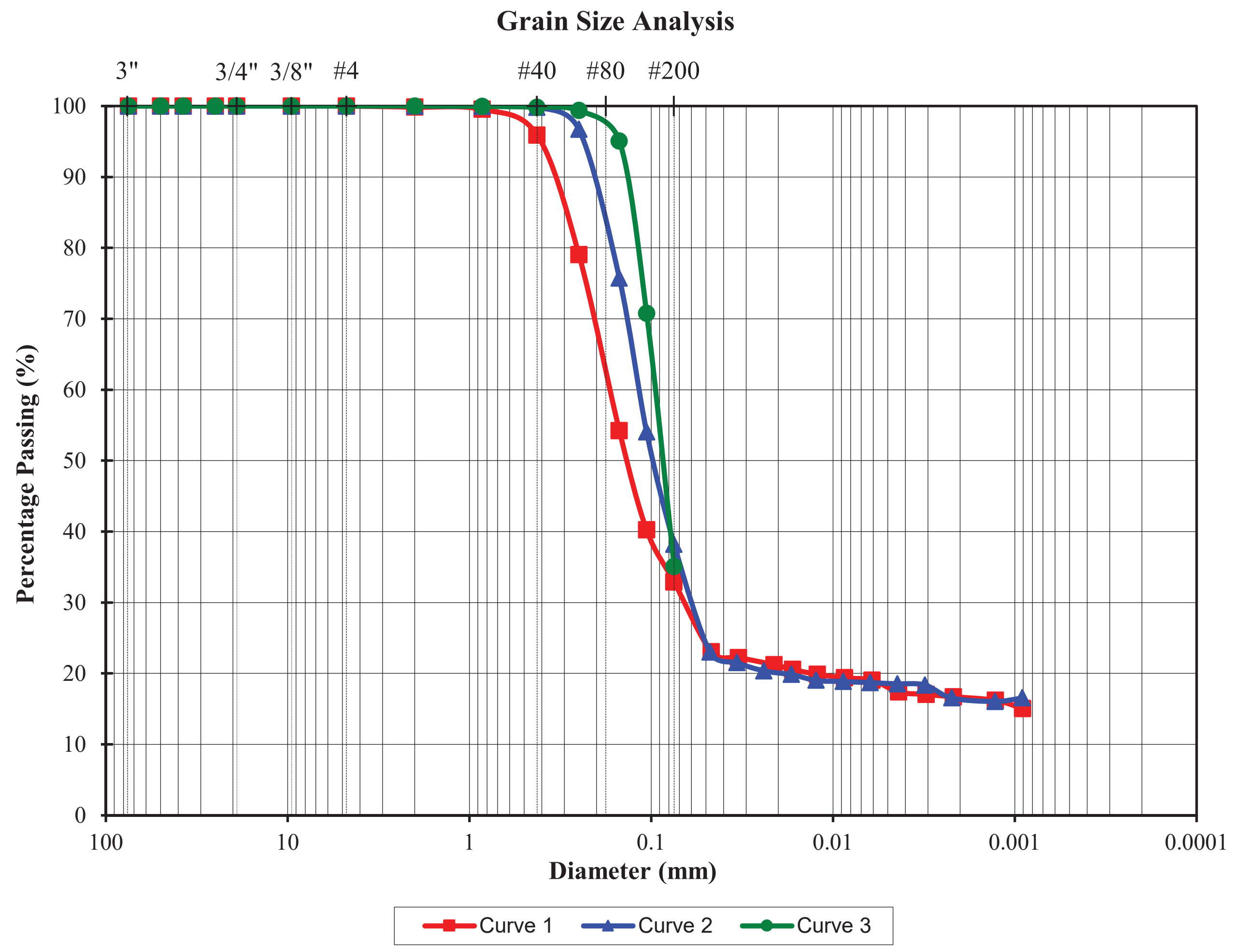
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913) & HYDROMETER (ASTM D 7928)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas

	Sand			
Gravel	Coarse to Medium	Fine	Silt	Clay



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-3	4-6	Clayey Sand (SC)	0.14	N/A	N/A
2	B-3	10-12	Clayey Sand (SC)	0.10	N/A	N/A
3	B-3	16-18	Clayey Sand (SC)	0.09	N/A	N/A

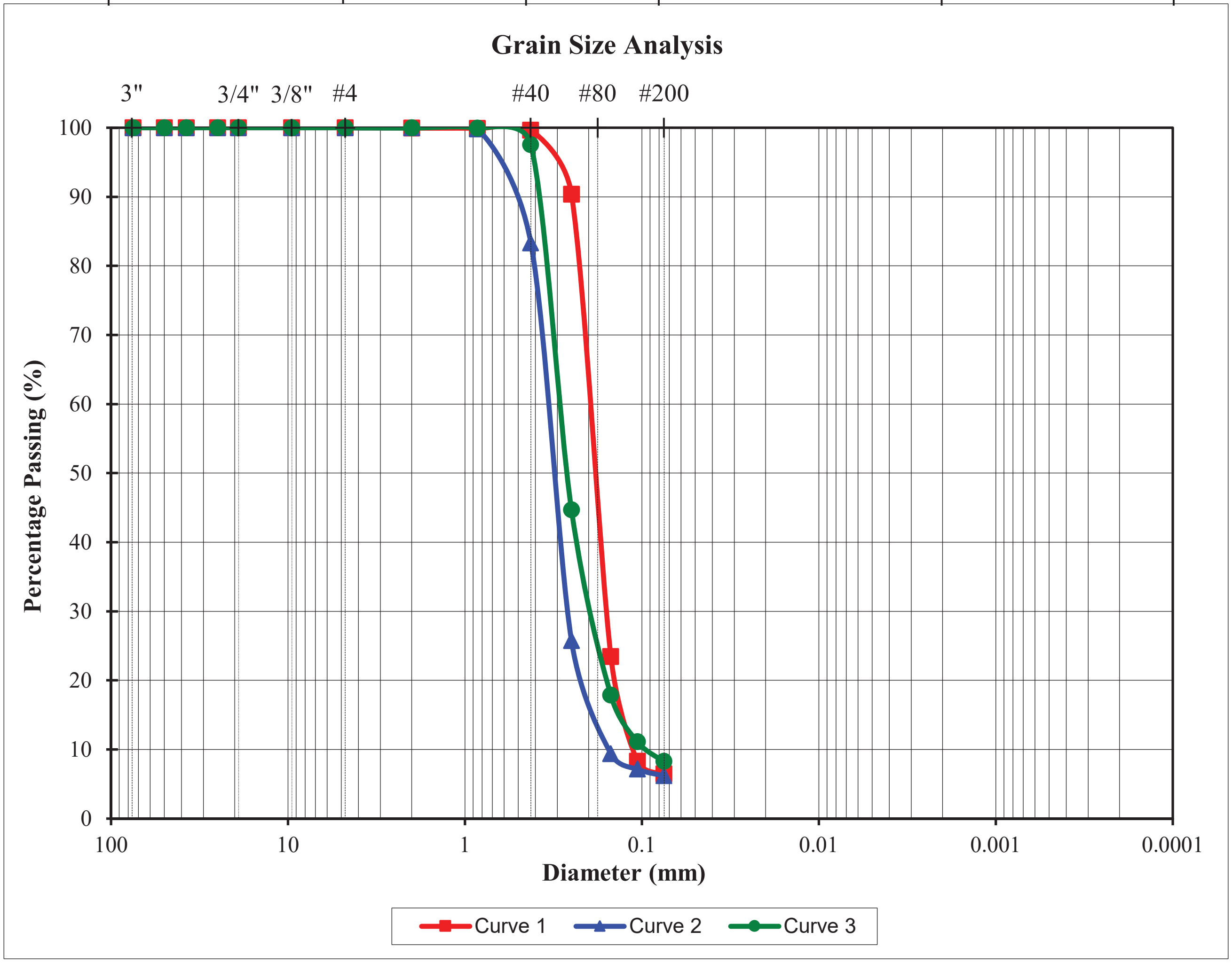
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas

		Sand				
	Gravel	Coarse to Medium	Fine	Silt	Clay	



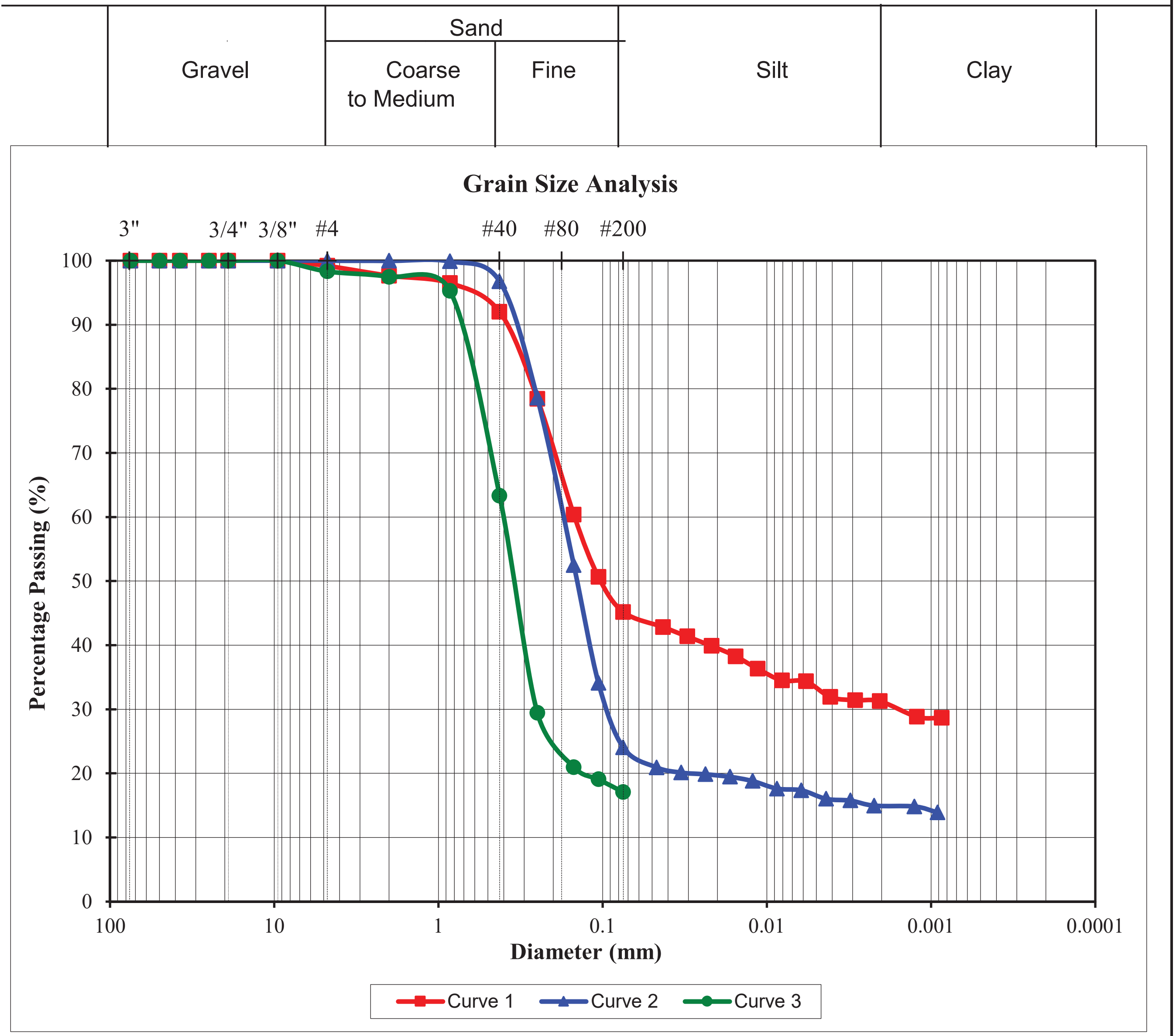
Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-3	68-70	Poorly Graded Sand with Silt (SP-SM)	0.19	1.84	1.12
2	B-3	78-80	Poorly Graded Sand with Silt (SP-SM)	0.32	2.30	1.27
3	B-3	98-100	Poorly Graded Sand with Silt (SP-SM)	0.27	3.20	1.35

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913) & HYDROMETER (ASTM D 7928)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-4	2-4	Clayey Sand (SC)	0.10	N/A	N/A
2	B-4	14-16	Silty Clayey Sand (SC-SM)	0.14	N/A	N/A
3	B-4	18-20	Silty Clayey Sand (SC-SM)	0.36	N/A	N/A

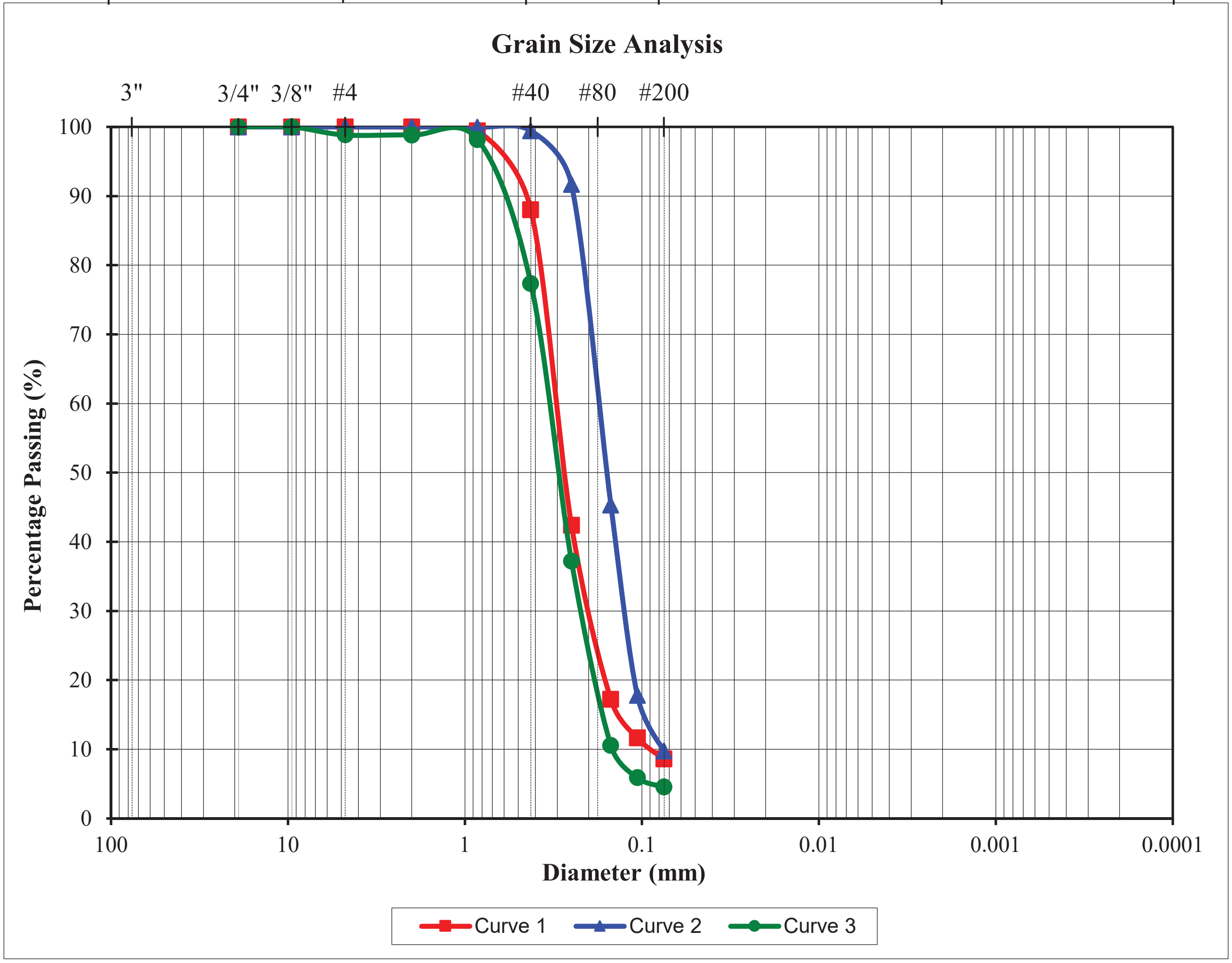
AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

GRAIN SIZE ANALYSIS - SIEVE (ASTM D 6913)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21
Location of Project: Waller County, Texas

Gravel	Sand		Silt	Clay
	Coarse to Medium	Fine		



Curve	Boring	Depth (ft)	Soil Description	D50 (mm)	Cu	Cc
1	B-4	33-35	Poorly Graded Sand with Silt (SP-SM)	0.28	3.55	1.42
2	B-4	63-65	Poorly Graded Sand with Silt (SP-SM)	0.16	2.39	1.14
3	B-4	78-80	Poorly Graded Sand (SP)	0.31	2.41	0.98

AVILES ENGINEERING CORPORATION

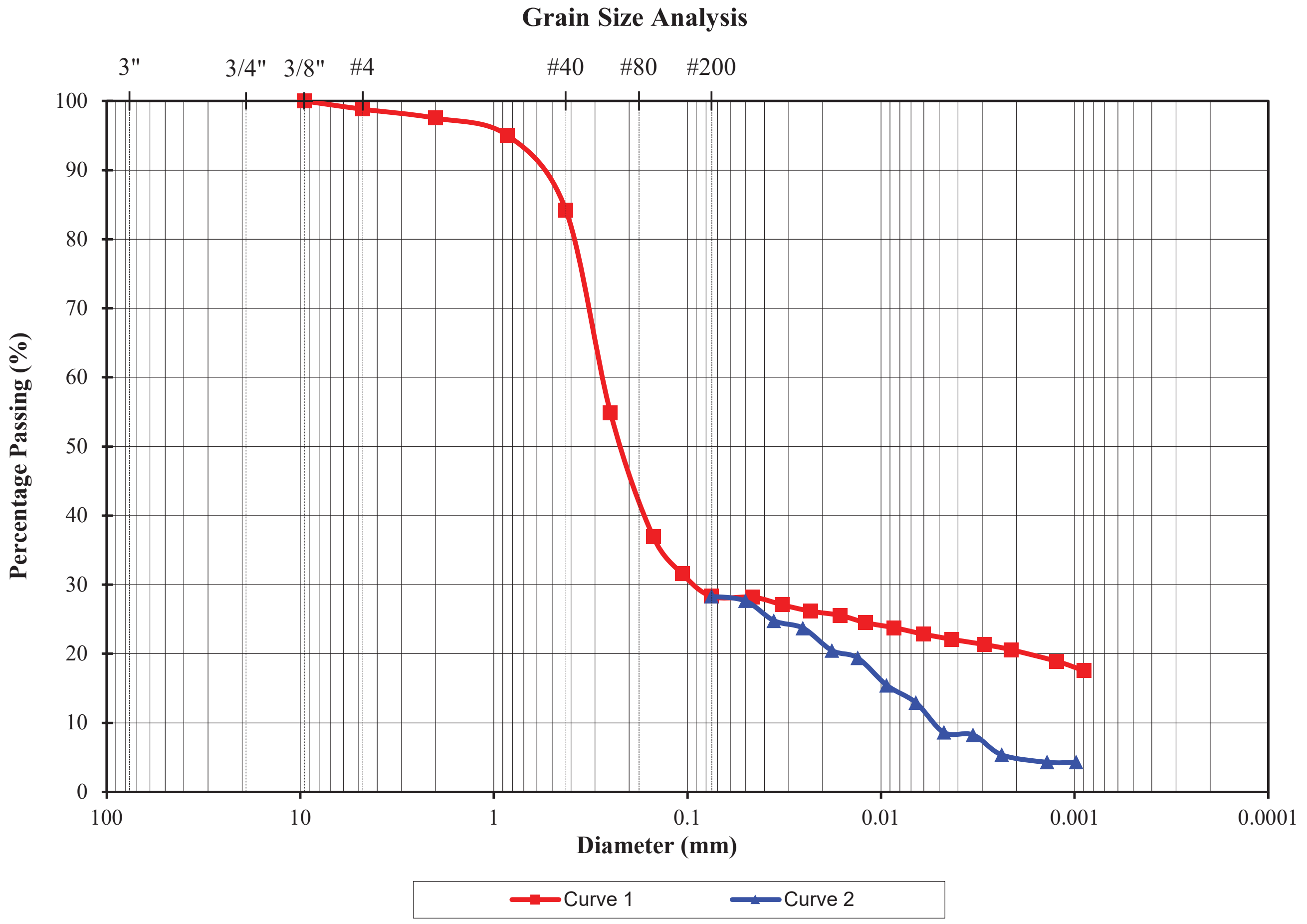
Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

DOUBLE HYDROMETER TESTS (ASTM D 4221)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21

Location of Project: Waller County, Texas

	Gravel	Sand		Silt	Clay	
		Coarse to Medium	Fine			



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	%-2μm	D_{50} (mm)
1	B-1	8-10 (a)	Clayey Sand (SC)	N/A	N/A	20.34	0.223
2	B-1	8-10 (b)	Clayey Sand (SC)			4.98	
				% Dispersion =		24.46%	

Notes:(a) Hydrometer test with added dispersant

(b) Hydrometer test without added dispersant

AVILES ENGINEERING CORPORATION

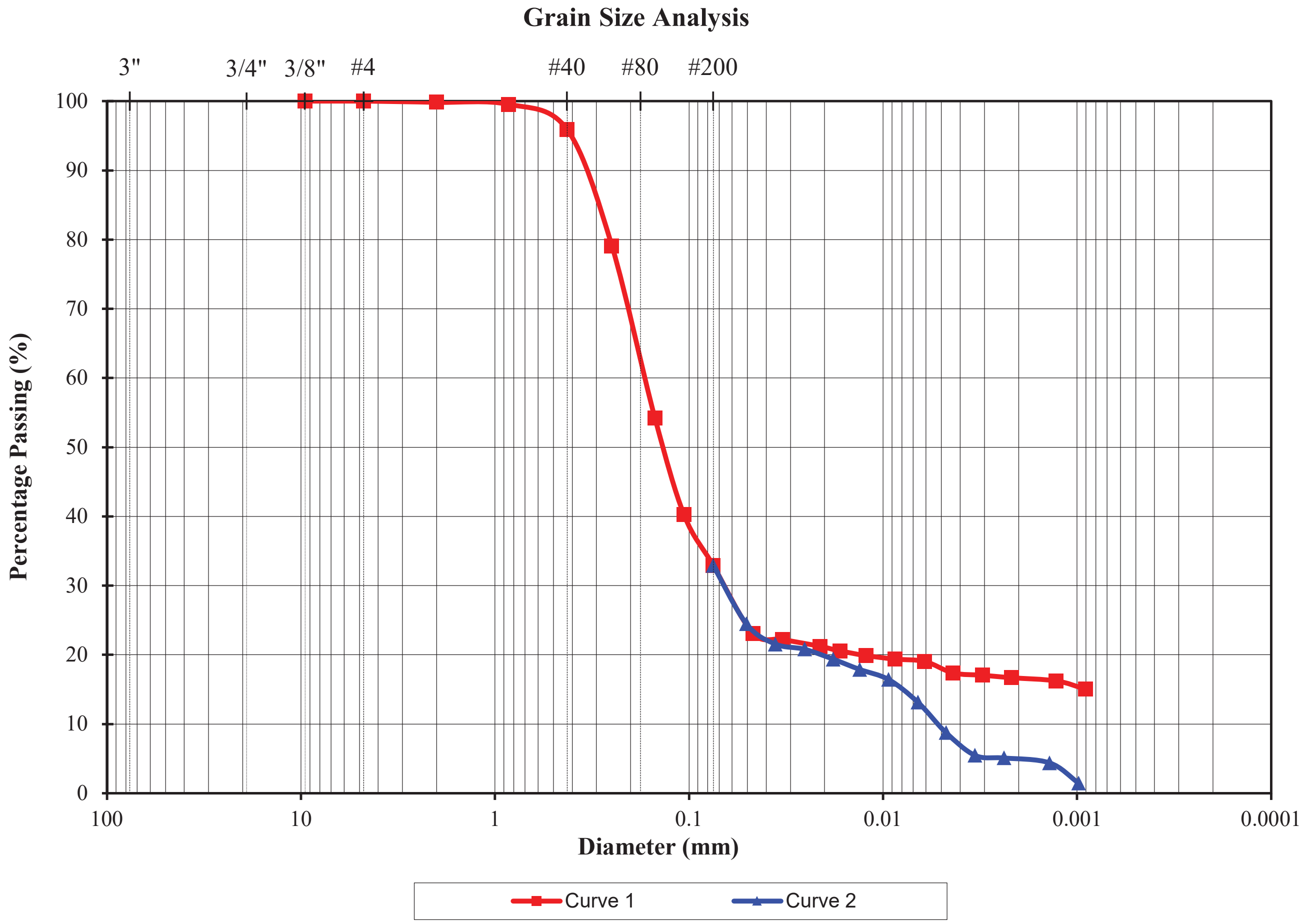
Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

DOUBLE HYDROMETER TESTS (ASTM D 4221)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21

Location of Project: Waller County, Texas

		Sand				
	Gravel	Coarse to Medium	Fine	Silt	Clay	



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	%-2μm	D_{50} (mm)
1	B-3	4-6 (a)	Clayey Sand (SC)	N/A	N/A	16.62	0.137
2	B-3	4-6 (b)	Clayey Sand (SC)			4.84	

% Dispersion = 29.10%

Notes:(a) Hydrometer test with added dispersant

(b) Hydrometer test without added dispersant

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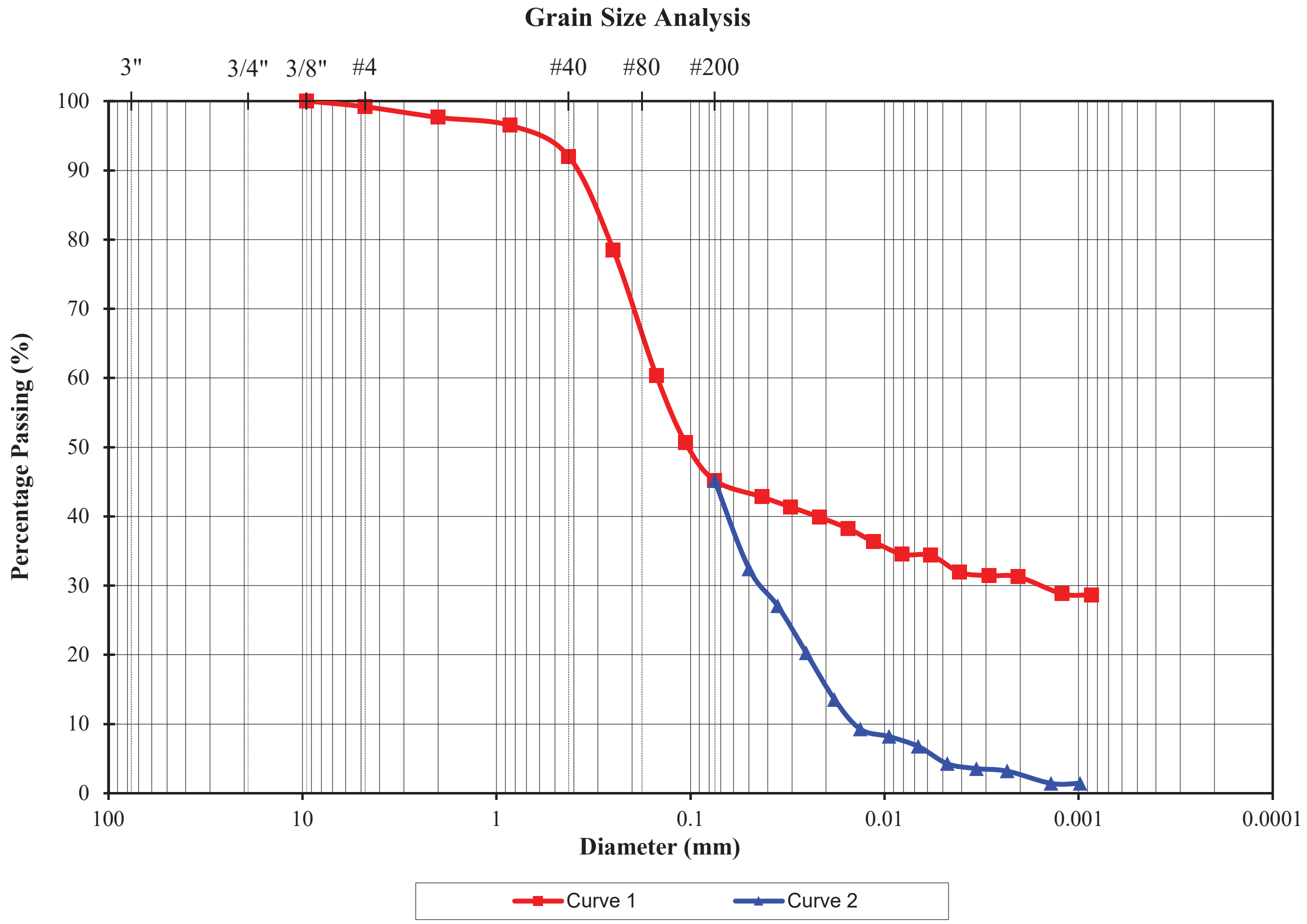
Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

DOUBLE HYDROMETER TESTS (ASTM D 4221)

Project: Spring Creek Watershed Feasibility Study **Job No.:** G154-21

Location of Project: Waller County, Texas

	Gravel	Sand		Silt	Clay	
		Coarse to Medium	Fine			



Curve	Boring	Depth (ft)	Soil Description	Cu	Cc	%-2μm	D_{50} (mm)
1	B-4	2-4 (a)	Clayey Sand (SC)	N/A	N/A	31.12	0.102
2	B-4	2-4 (b)	Clayey Sand (SC)			2.57	

% Dispersion = 8.27%

Notes:(a) Hydrometer test with added dispersant
(b) Hydrometer test without added dispersant

AVILES ENGINEERING CORPORATION

Consulting Engineers - Geotechnical, Construction Materials Testing, Environmental

RESULTS OF CRUMB TESTS (ASTM D 6572)

Project Name: Spring Creek Watershed Feasibility Study
Project No.: G154-21

Boring Number	Depth, feet	2 Minutes		1 Hour		6 Hours	
		Grade	C (deg)	Grade	C (deg)	Grade	C (deg)
B-1	6-8	2	21.8	2	21.8	2	21.3
B-4	4-6	1	21.8	1	21.8	1	21.3
B-4	12-14	2	21.8	3	21.8	3	21.3

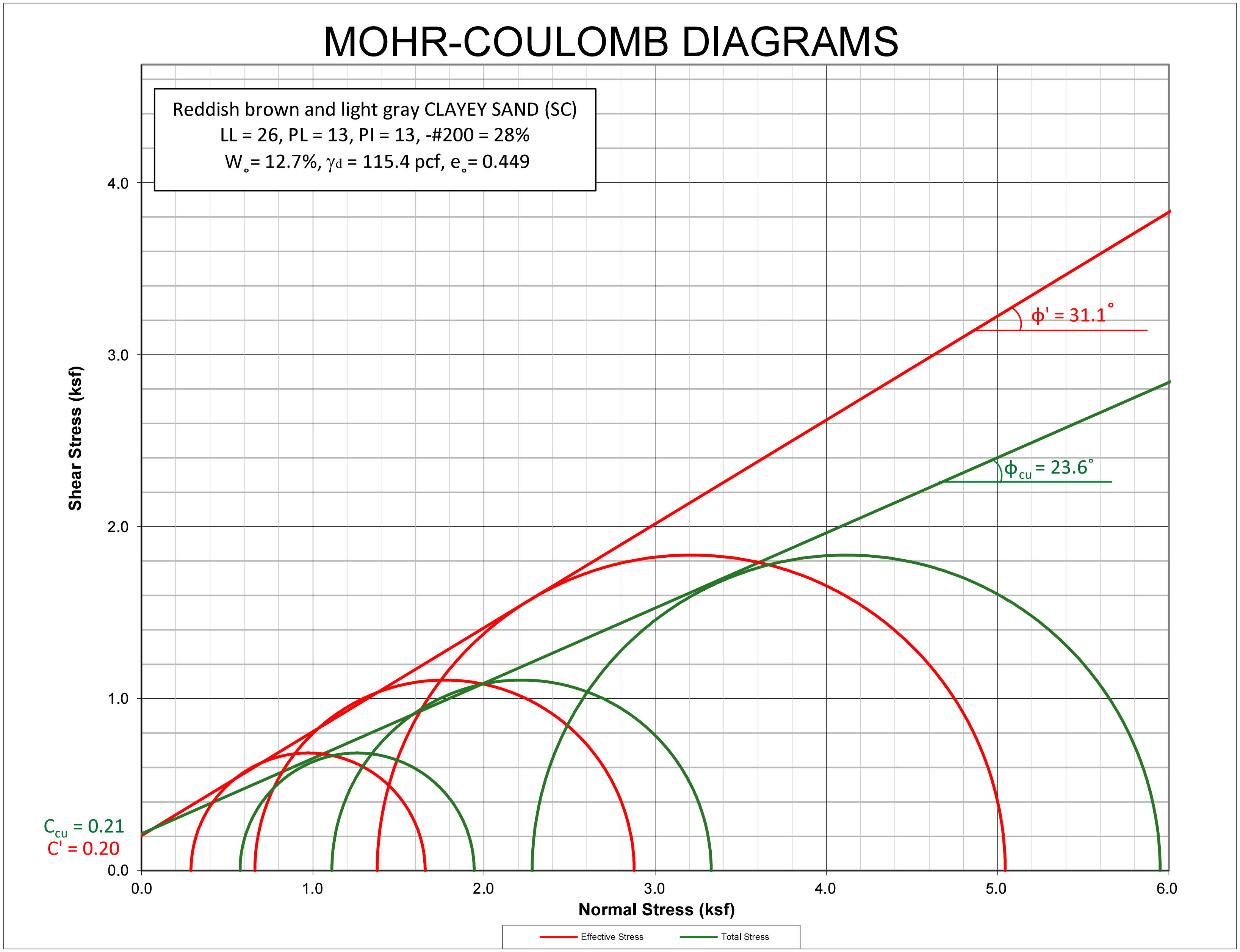
Grade Classification:

Grade 1	Non-dispersive; No reaction
Grade 2	Intermediate; Slight reaction
Grade 3	Dispersive; Moderate reaction
Grade 4	Highly Dispersive; Strong reaction

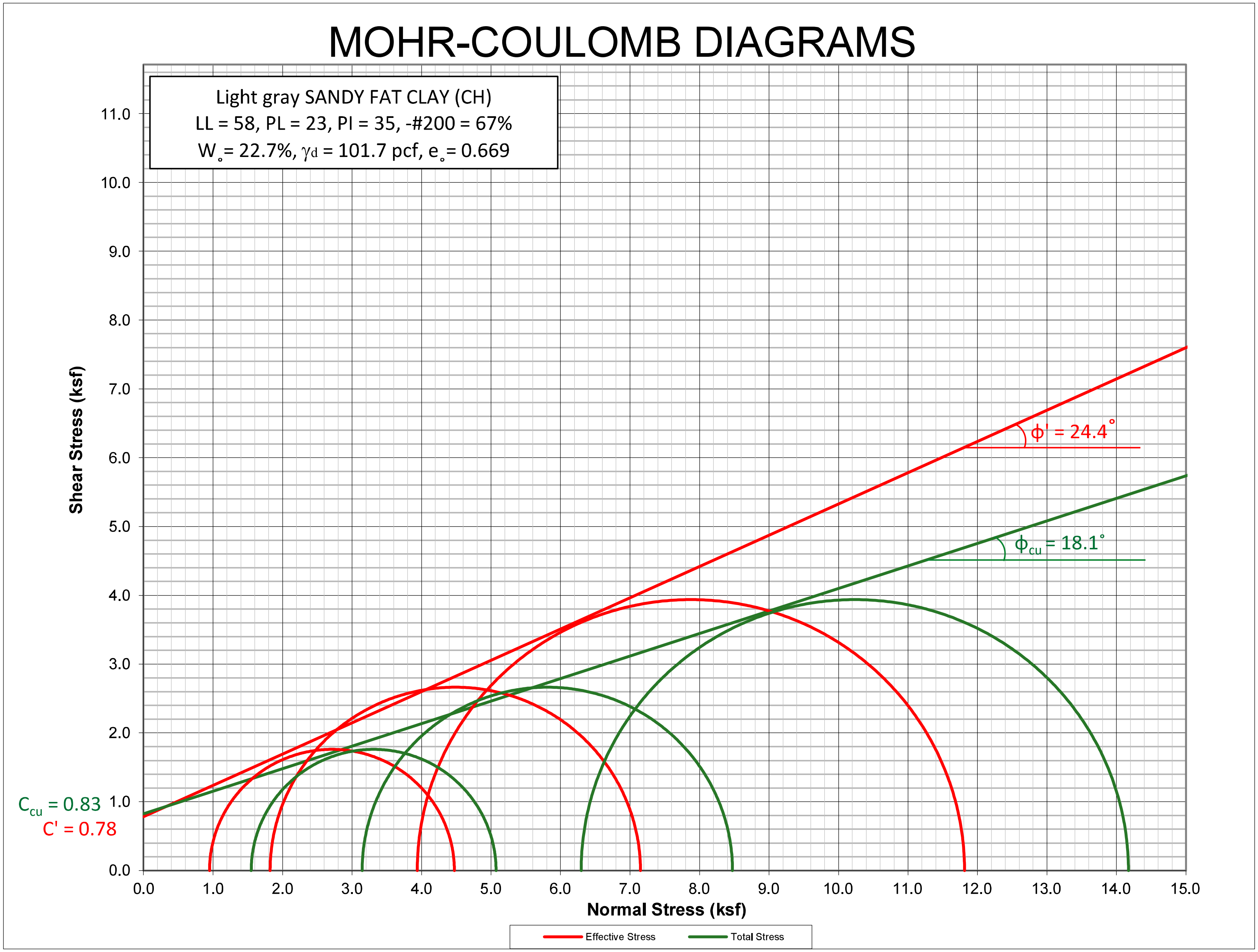
Interpretation:

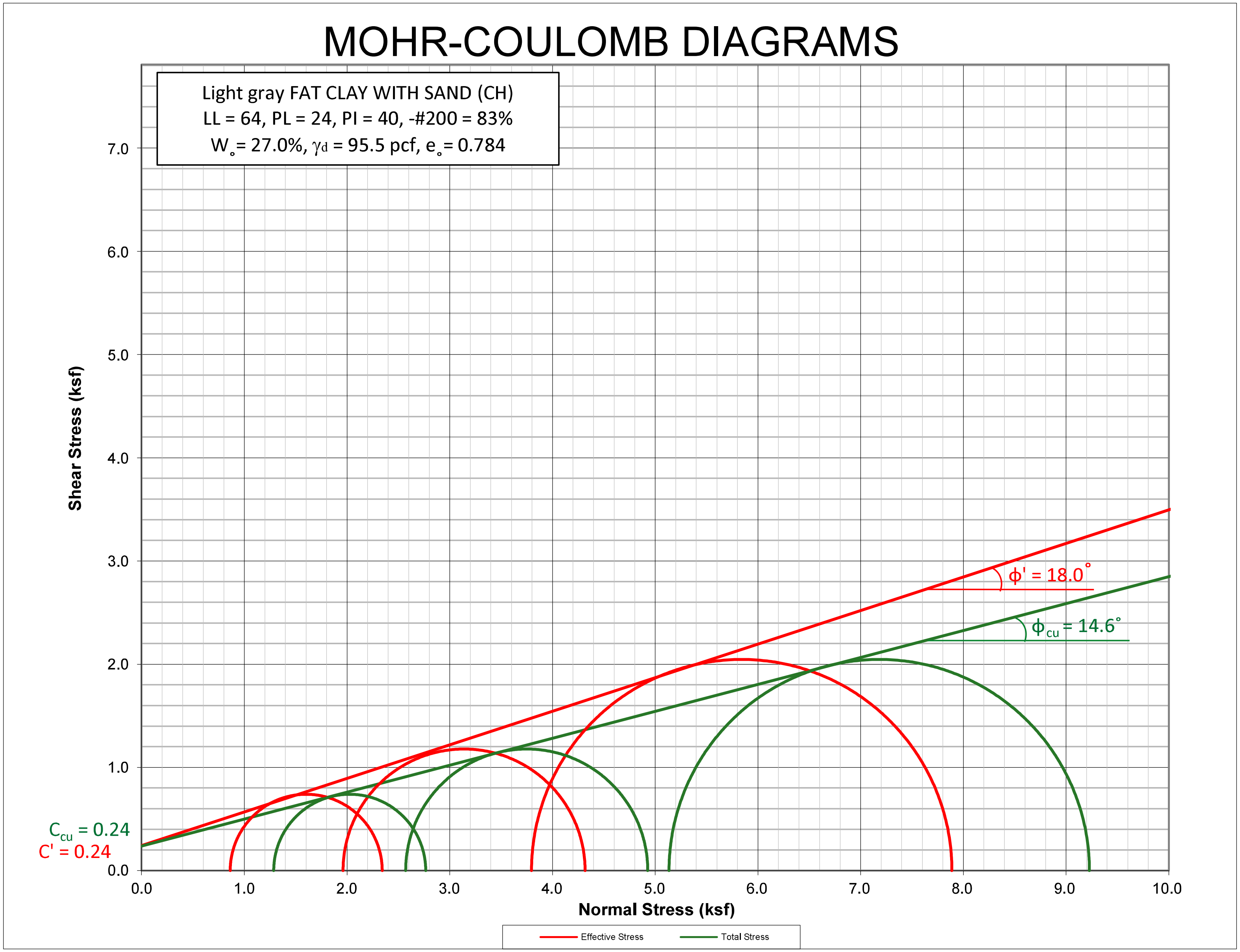
Under normal conditions, use the 1 hour reading to determine dispersive grade.
However, if the dispersive grade changes from 2 to 3 or from 3 to 4 between the 1 and 6 hour readings, use the 6 hour reading instead.

G154-21 Boring B-1, 8'-10'

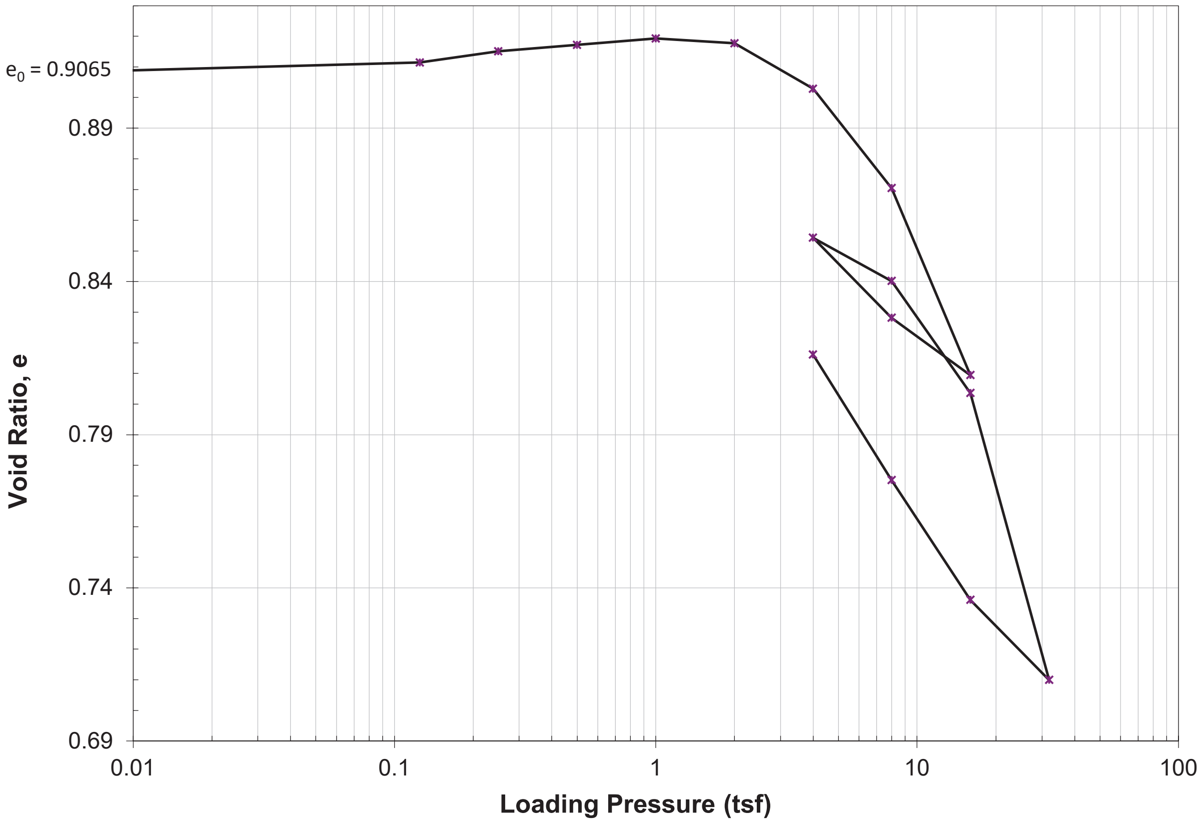


G154-21 Boring B-2, 43'-45'




MOHR-COULOMB DIAGRAMS
Light gray FAT CLAY WITH SAND (CH)
LL = 64, PL = 24, PI = 40, -#200 = 83%
W∘= 27.0%, γd = 95.5 pcf, e∘= 0.784
ϕ' = 18.0˚
ϕcu = 14.6˚
Ccu = 0.24
C' = 0.24
Shear Stress (ksf)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
Normal Stress (ksf)
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
Effective Stress
Total Stress

CONSOLIDATION TEST RESULTS



Project No.: G154-21

Project: SJRA Spring Creek Watershed Feasibility Study

Sample ID: B-1, 58 to 60 feet

Sample Description: Tan and light gray Sandy Fat Clay (CH)

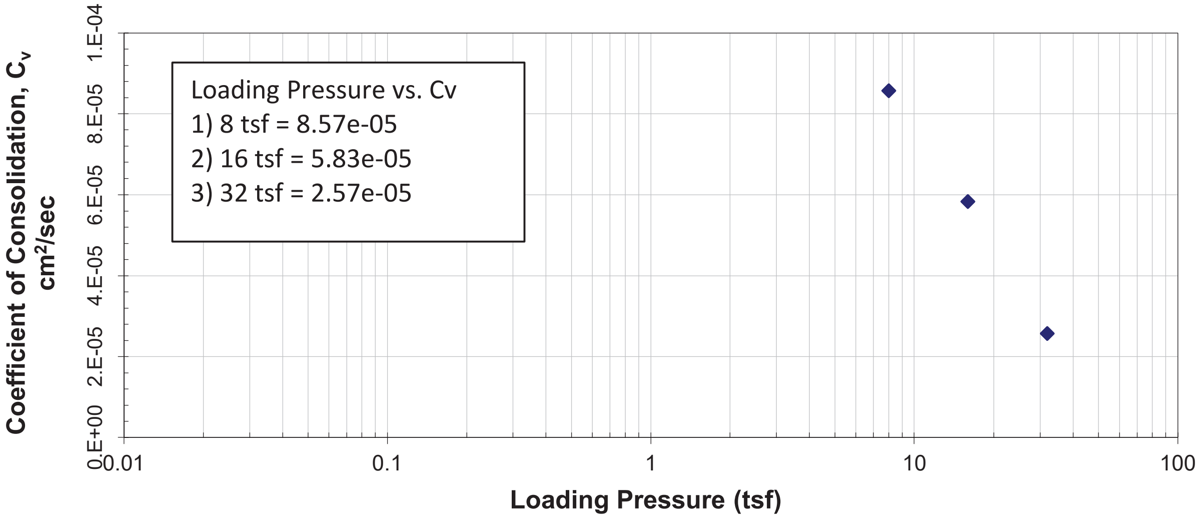
Dry Unit Weight (γ_d): 90.2 pcf

Estimated Consolidation Index (Cc): 0.3109

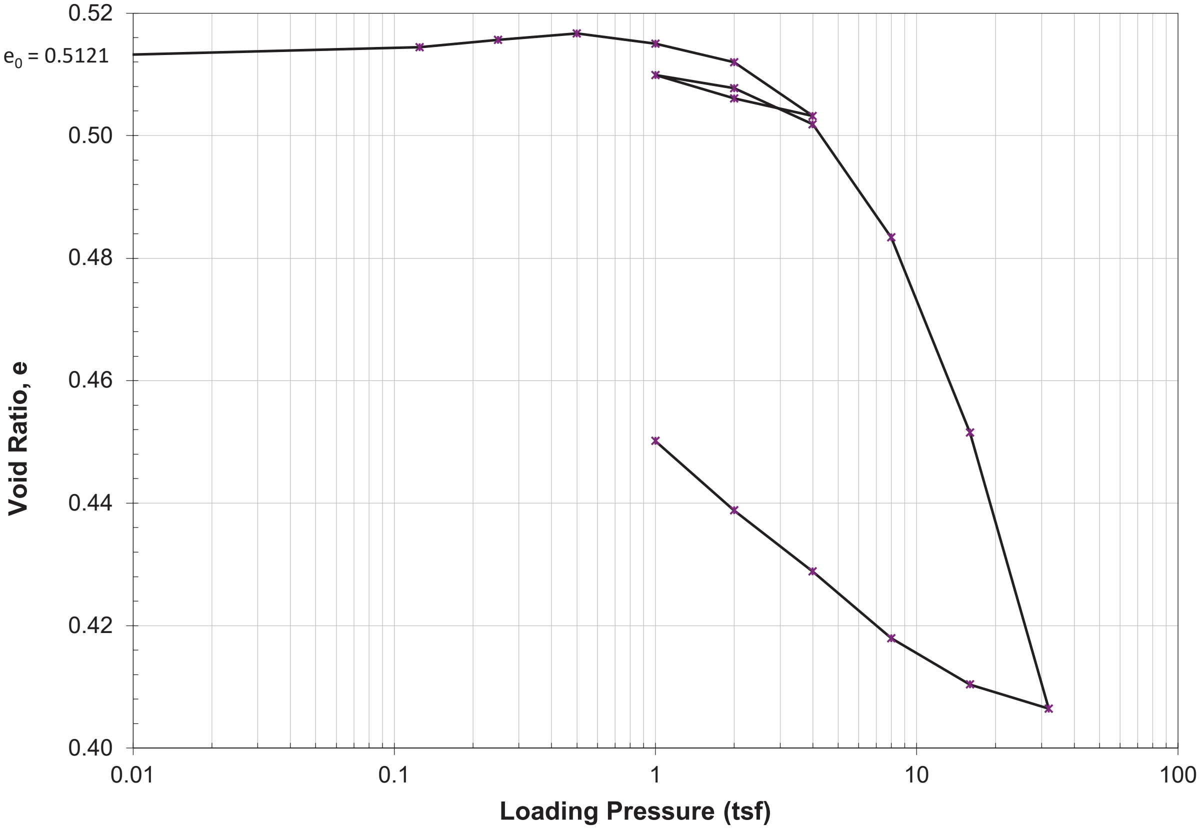
Estimated Recompression Index (Cr): 0.0769

Estimated OCR: 2.9

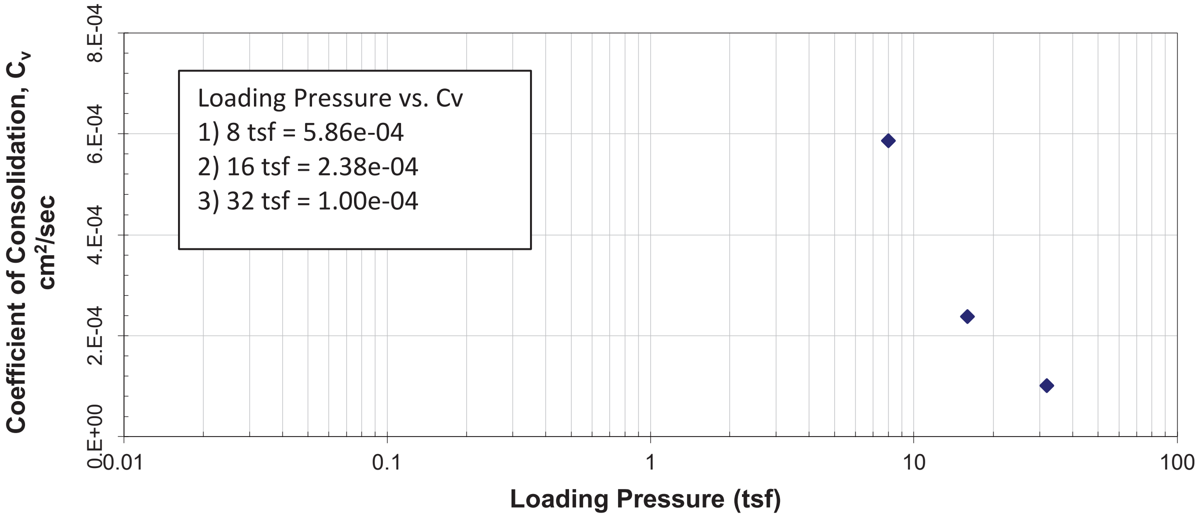
Estimated Preconsolidation Pressure (Pc): 10.1 tsf



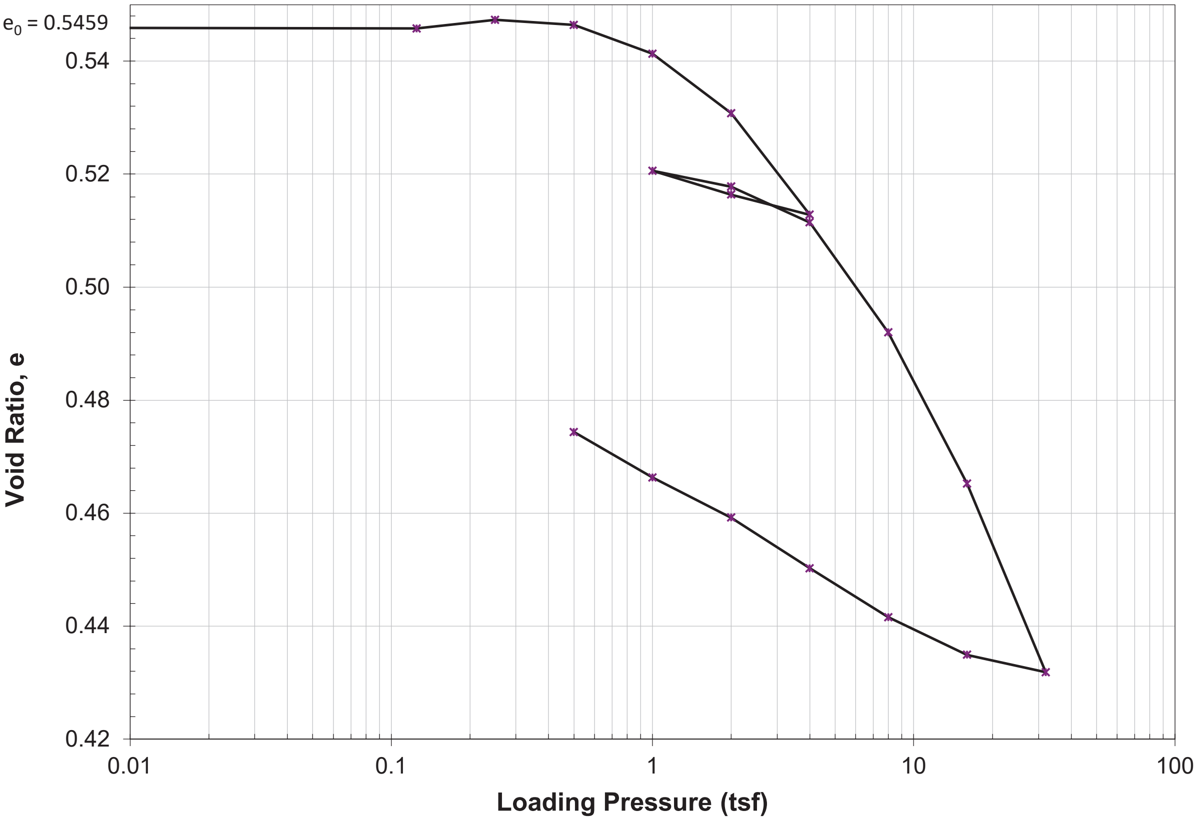
CONSOLIDATION TEST RESULTS



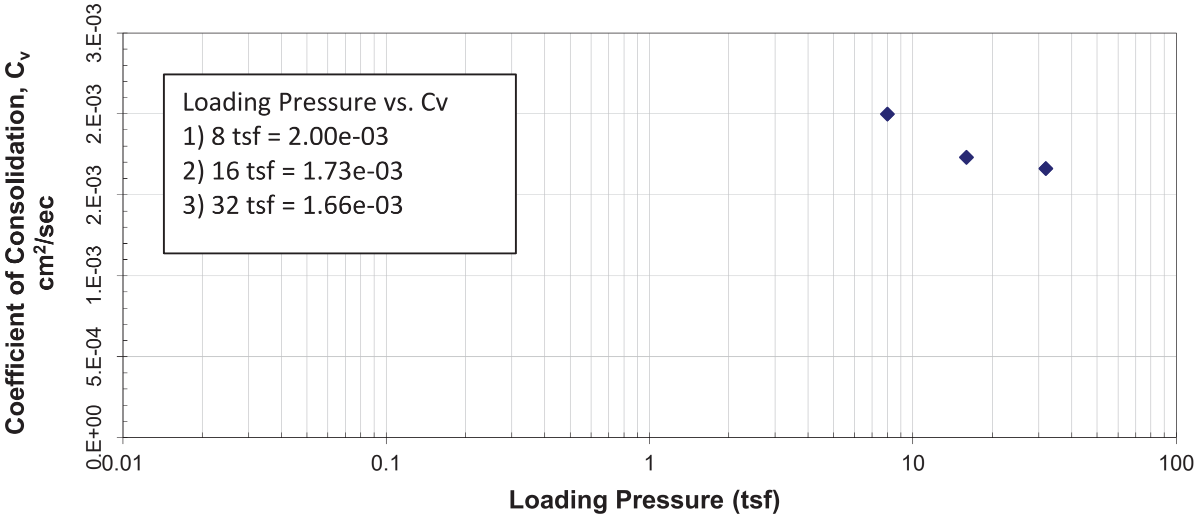
Project No.: G154-21
Project: SJRA Spring Creek Watershed Feasibility Study
Sample ID: B-2, 53 to 55 feet
Sample Description: Gray Sandy Lean Clay (CL)
Dry Unit Weight (γ_d): 112.2 pcf
Estimated Consolidation Index (Cc): 0.1498
Estimated Recompression Index (Cr): 0.0116
Estimated OCR: 3.2
Estimated Preconsolidation Pressure (Pc): 6.2 tsf



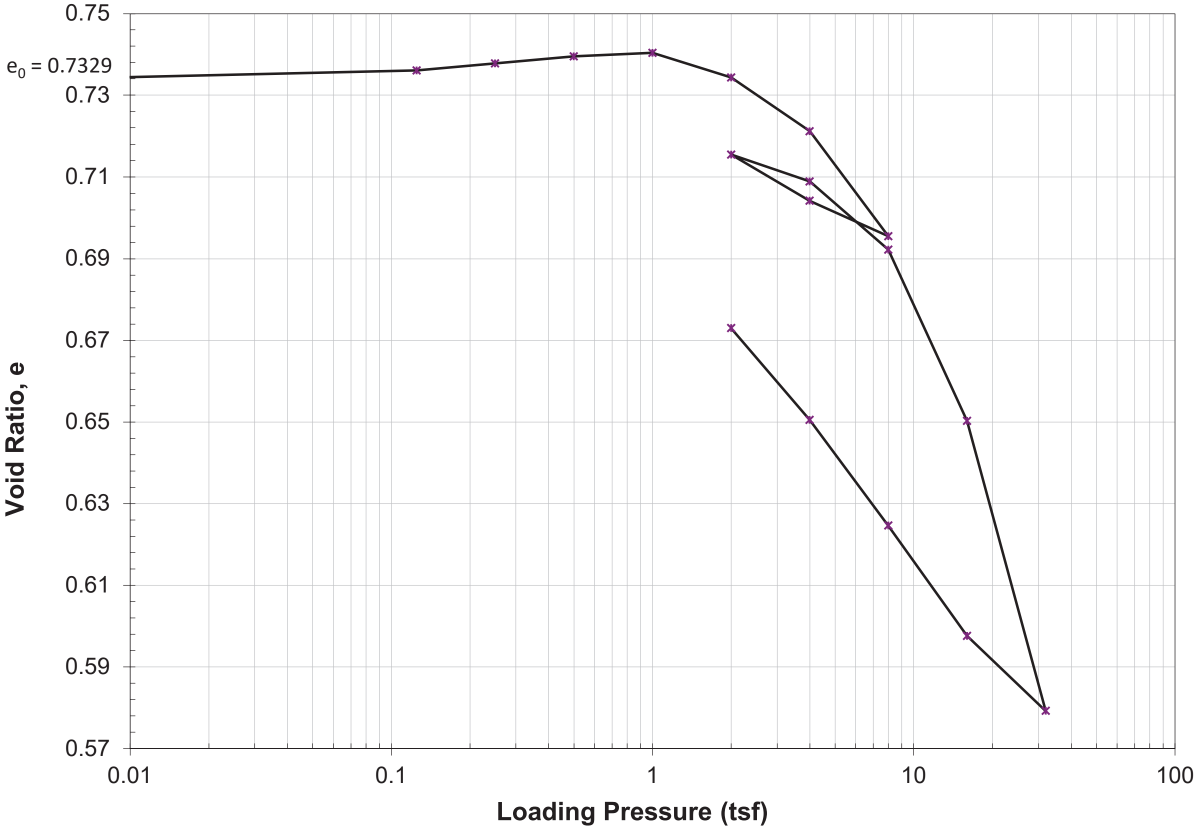
CONSOLIDATION TEST RESULTS



Project No.: G154-21
Project: SJRA Spring Creek Watershed Feasibility Study
Sample ID: B-2, 83 to 85 feet
Sample Description: Light gray Sandy Lean Clay (CL)
Dry Unit Weight (γ_d): 108.5 pcf
Estimated Consolidation Index (Cc): 0.1110
Estimated Recompression Index (Cr): 0.0133
Estimated OCR: 1.4
Estimated Preconsolidation Pressure (Pc): 3.8 tsf

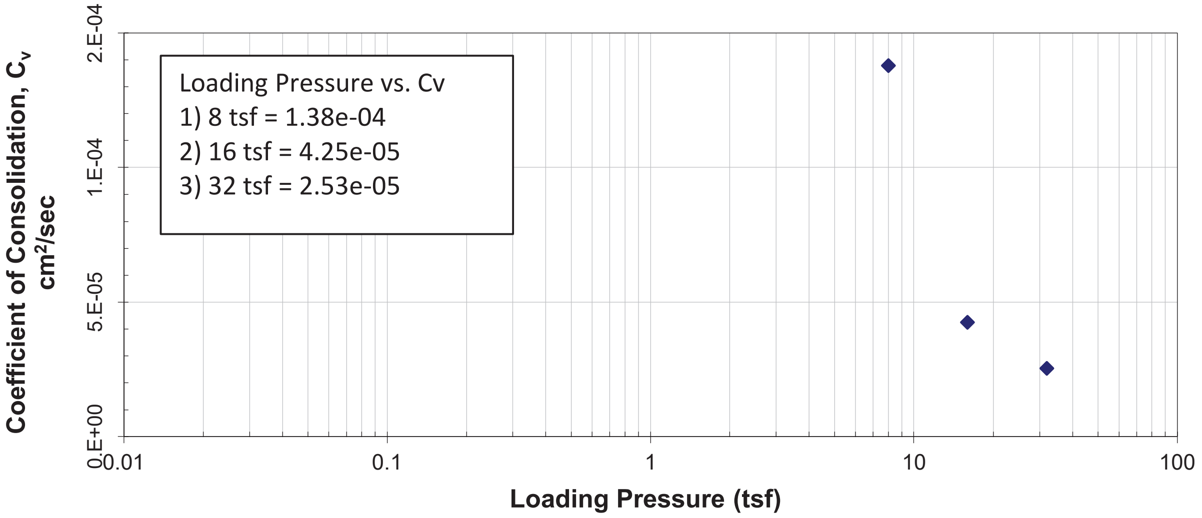


CONSOLIDATION TEST RESULTS



Project No.: G154-21
Sample ID: B-4, 48 to 50 feet
Sample Description: Light gray Fat Clay (CH)
Estimated Consolidation Index (Cc): 0.2362
Estimated OCR: 3.8

Project: SJRA Spring Creek Watershed Feasibility Study
Dry Unit Weight (γ_d): 97.6 pcf
Estimated Recompression Index (Cr): 0.0341
Estimated Preconsolidation Pressure (Pc): 8.8 tsf



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Waller County, Texas
Sample ID: B-2, 48'-50' **Trial #1**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.567	in =	14.140	cm		
Initial specimen diameter, d_i =	2.778	in =	7.056	cm		
Initial area of specimen, A_i =	6.061	sq in =	39.104	sq cm		
Initial volume of specimen, V_i =	33.742	cu in =	552.938	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.35	gram =	2.560	lbs		
Initial Moist Unit Weight, r_i =	131.12	pcf			Initial moisture content, w_i =	20.6%
Initial Dry Unit Weight, r_{di} =	108.73	pcf			Initial degree of saturation, S_i =	101.2%

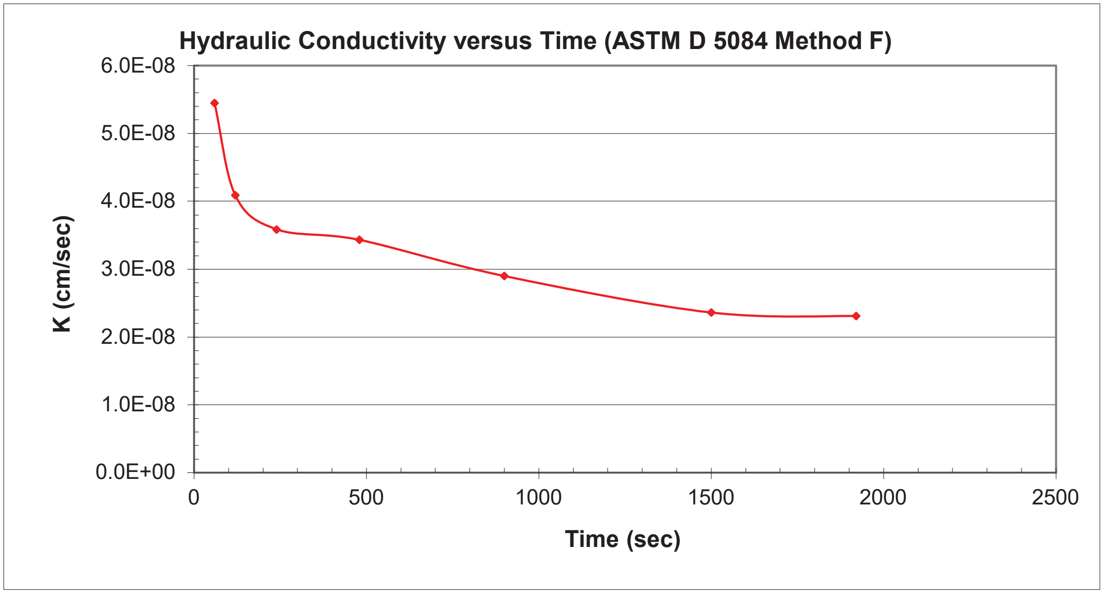
After Permeability Test:

Final specimen height, h_f =	5.570	in =	14.148	cm		
Final specimen diameter, d_f =	2.778	in =	7.056	cm		
Final area of specimen, A_f =	6.061	sq in =	39.104	sq cm		
Final volume of specimen, V_f =	33.761	cu in =	553.236	cu cm		
Final weight of specimen, W_f =	1167.68	gram =	2.574	lbs		
Final Moist Unit Weight, r_f =	131.76	pcf			Final moisture content, w_f =	18.6%
Final Dry Unit Weight, r_{df} =	111.12	pcf			Final degree of saturation, S_f =	97.2%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 72.0 psi
Cell Pressure: 75.0 psi Effective Overburden Pressure: 3.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	2.90E-08	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**2.83E-08**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Waller County, Texas
Sample ID: B-2, 48'-50' **Trial #2**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.567	in =	14.140	cm		
Initial specimen diameter, d_i =	2.778	in =	7.056	cm		
Initial area of specimen, A_i =	6.061	sq in =	39.104	sq cm		
Initial volume of specimen, V_i =	33.742	cu in =	552.938	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.35	gram =	2.560	lbs		
Initial Moist Unit Weight, r_i =	131.12	pcf			Initial moisture content, w_i =	20.6%
Initial Dry Unit Weight, r_{di} =	108.73	pcf			Initial degree of saturation, S_i =	101.2%

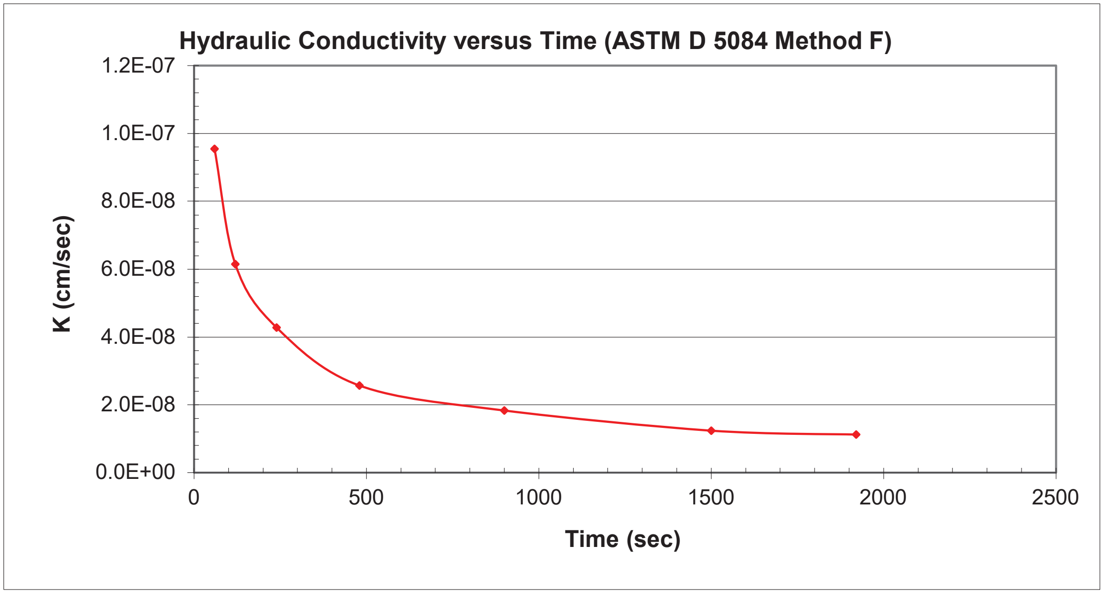
After Permeability Test:

Final specimen height, h_f =	5.570	in =	14.148	cm		
Final specimen diameter, d_f =	2.778	in =	7.056	cm		
Final area of specimen, A_f =	6.061	sq in =	39.104	sq cm		
Final volume of specimen, V_f =	33.761	cu in =	553.236	cu cm		
Final weight of specimen, W_f =	1167.68	gram =	2.574	lbs		
Final Moist Unit Weight, r_f =	131.76	pcf			Final moisture content, w_f =	18.6%
Final Dry Unit Weight, r_{df} =	111.12	pcf			Final degree of saturation, S_f =	97.2%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 72.0 psi
Cell Pressure: 75.0 psi Effective Overburden Pressure: 3.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	1.24E-08	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**1.21E-08**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Waller County, Texas
Sample ID: B-2, 48'-50' **Trial #3**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.567	in =	14.140	cm		
Initial specimen diameter, d_i =	2.778	in =	7.056	cm		
Initial area of specimen, A_i =	6.061	sq in =	39.104	sq cm		
Initial volume of specimen, V_i =	33.742	cu in =	552.938	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.35	gram =	2.560	lbs		
Initial Moist Unit Weight, r_i =	131.12	pcf			Initial moisture content, w_i =	20.6%
Initial Dry Unit Weight, r_{di} =	108.73	pcf			Initial degree of saturation, S_i =	101.2%

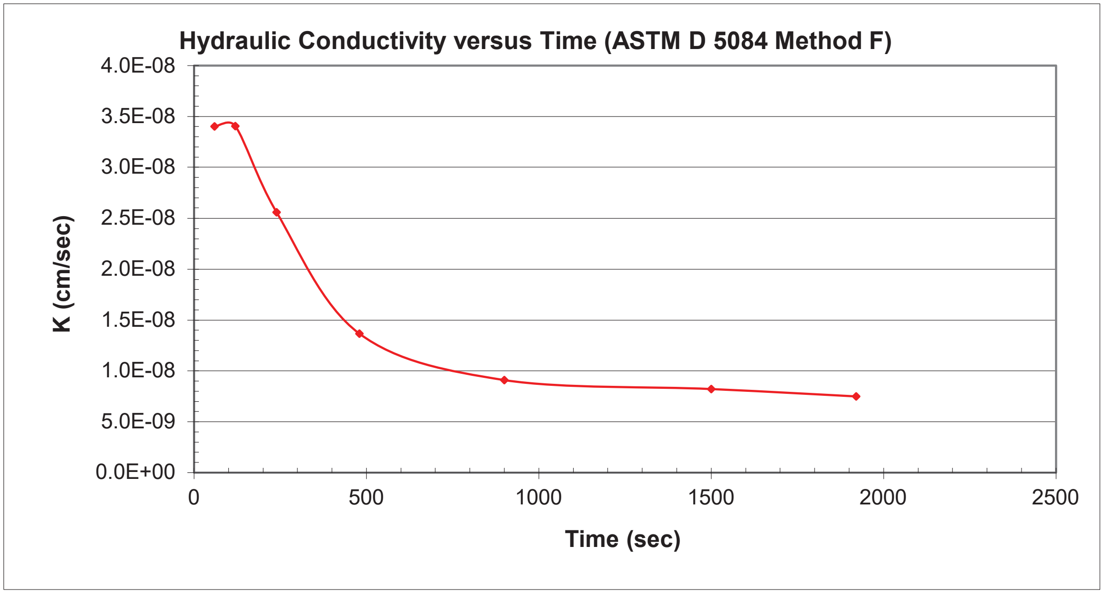
After Permeability Test:

Final specimen height, h_f =	5.570	in =	14.148	cm		
Final specimen diameter, d_f =	2.778	in =	7.056	cm		
Final area of specimen, A_f =	6.061	sq in =	39.104	sq cm		
Final volume of specimen, V_f =	33.761	cu in =	553.236	cu cm		
Final weight of specimen, W_f =	1167.68	gram =	2.574	lbs		
Final Moist Unit Weight, r_f =	131.76	pcf			Final moisture content, w_f =	18.6%
Final Dry Unit Weight, r_{df} =	111.12	pcf			Final degree of saturation, S_f =	97.2%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 72.0 psi
Cell Pressure: 75.0 psi Effective Overburden Pressure: 3.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	9.10E-09	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**8.89E-09**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Waller County, Texas
Sample ID: B-2, 48'-50' **Trial #4**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.567	in =	14.140	cm		
Initial specimen diameter, d_i =	2.778	in =	7.056	cm		
Initial area of specimen, A_i =	6.061	sq in =	39.104	sq cm		
Initial volume of specimen, V_i =	33.742	cu in =	552.938	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.35	gram =	2.560	lbs		
Initial Moist Unit Weight, r_i =	131.12	pcf			Initial moisture content, w_i =	20.6%
Initial Dry Unit Weight, r_{di} =	108.73	pcf			Initial degree of saturation, S_i =	101.2%

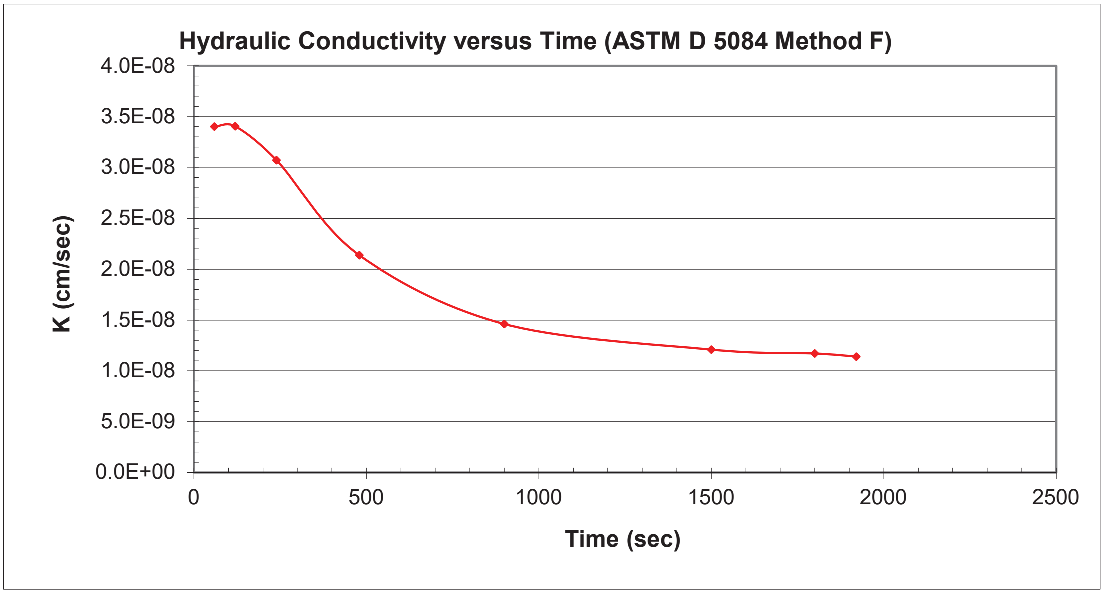
After Permeability Test:

Final specimen height, h_f =	5.570	in =	14.148	cm		
Final specimen diameter, d_f =	2.778	in =	7.056	cm		
Final area of specimen, A_f =	6.061	sq in =	39.104	sq cm		
Final volume of specimen, V_f =	33.761	cu in =	553.236	cu cm		
Final weight of specimen, W_f =	1167.68	gram =	2.574	lbs		
Final Moist Unit Weight, r_f =	131.76	pcf			Final moisture content, w_f =	18.6%
Final Dry Unit Weight, r_{df} =	111.12	pcf			Final degree of saturation, S_f =	97.2%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 72.0 psi
Cell Pressure: 75.0 psi Effective Overburden Pressure: 3.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	1.21E-08	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**1.18E-08**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Walle County, Texas
Sample ID: B-4, 53'-55' **Trial #1**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.523	in =	14.028	cm		
Initial specimen diameter, d_i =	2.753	in =	6.993	cm		
Initial area of specimen, A_i =	5.953	sq in =	38.403	sq cm		
Initial volume of specimen, V_i =	32.876	cu in =	538.739	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.82	gram =	2.561	lbs		
Initial Moist Unit Weight, r_i =	134.63	pcf			Initial moisture content, w_i =	18.1%
Initial Dry Unit Weight, r_{di} =	114.02	pcf			Initial degree of saturation, S_i =	102.2%

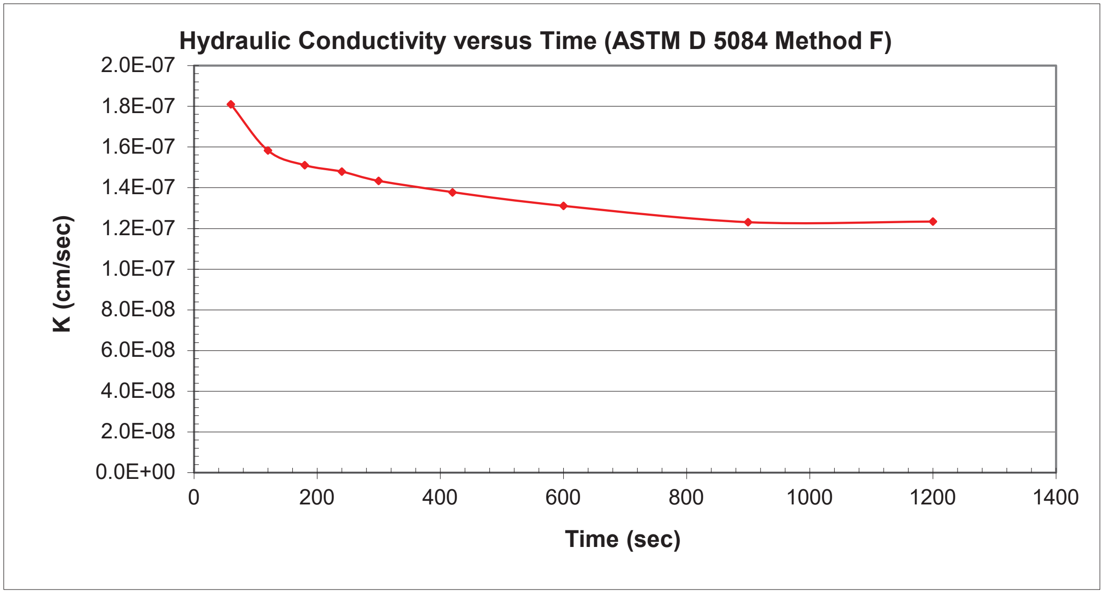
After Permeability Test:

Final specimen height, h_f =	5.498	in =	13.965	cm		
Final specimen diameter, d_f =	2.788	in =	7.082	cm		
Final area of specimen, A_f =	6.105	sq in =	39.386	sq cm		
Final volume of specimen, V_f =	33.564	cu in =	550.024	cu cm		
Final weight of specimen, W_f =	1152.44	gram =	2.541	lbs		
Final Moist Unit Weight, r_f =	130.80	pcf			Final moisture content, w_f =	18.5%
Final Dry Unit Weight, r_{df} =	110.40	pcf			Final degree of saturation, S_f =	94.8%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 35.0 psi
Cell Pressure: 69.0 psi Effective Overburden Pressure: 34.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	1.31E-07	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**1.28E-07**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study
Job No.: G154-21 Project Location: Walle County, Texas
Sample ID: B-4, 53'-55' **Trial #2**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.523	in =	14.028	cm		
Initial specimen diameter, d_i =	2.753	in =	6.993	cm		
Initial area of specimen, A_i =	5.953	sq in =	38.403	sq cm		
Initial volume of specimen, V_i =	32.876	cu in =	538.739	cu cm		
Specific gravity, assumed G_s =	2.70					
Initial weight of specimen, W_i =	1161.82	gram =	2.561	lbs		
Initial Moist Unit Weight, r_i =	134.63	pcf			Initial moisture content, w_i =	18.1%
Initial Dry Unit Weight, r_{di} =	114.02	pcf			Initial degree of saturation, S_i =	102.2%

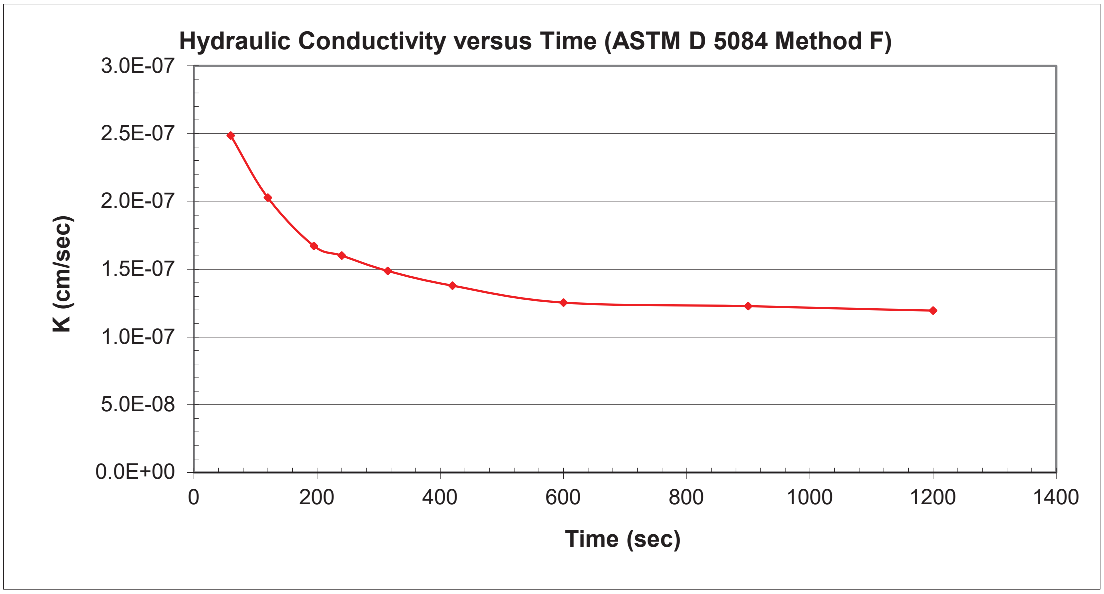
After Permeability Test:

Final specimen height, h_f =	5.498	in =	13.965	cm		
Final specimen diameter, d_f =	2.788	in =	7.082	cm		
Final area of specimen, A_f =	6.105	sq in =	39.386	sq cm		
Final volume of specimen, V_f =	33.564	cu in =	550.024	cu cm		
Final weight of specimen, W_f =	1152.44	gram =	2.541	lbs		
Final Moist Unit Weight, r_f =	130.80	pcf			Final moisture content, w_f =	18.5%
Final Dry Unit Weight, r_{df} =	110.40	pcf			Final degree of saturation, S_f =	94.8%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 35.0 psi
Cell Pressure: 69.0 psi Effective Overburden Pressure: 34.0 psi

5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	1.25E-07	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**1.22E-07**	**cm/sec**



Permeability Test using a Flexible Wall Permeameter (Constant Volume - Falling Head Method, ASTM D 5084)

Test Data Summary/Report:

Project: Spring Creek Watershed Feasibility Study

Job No.: G154-21 Project Location: Walle County, Texas

Sample ID: B-4, 53'-55' **Trial #3**

1. Testing Method: ASTM D 5084 Method F: Constant Volume - Falling Head, Flexible Wall Permeameter

2. Specimen Preparation:

3. **Before Permeability Test:**

Initial specimen height, h_i =	5.523	in =	14.028	cm
Initial specimen diameter, d_i =	2.753	in =	6.993	cm
Initial area of specimen, A_i =	5.953	sq in =	38.403	sq cm
Initial volume of specimen, V_i =	32.876	cu in =	538.739	cu cm
Specific gravity, assumed G_s =	2.70			
Initial weight of specimen, W_i =	1161.82	gram =	2.561	lbs
Initial Moist Unit Weight, r_i =	134.63	pcf	Initial moisture content, w_i =	18.1%
Initial Dry Unit Weight, r_{di} =	114.02	pcf	Initial degree of saturation, S_i =	102.2%

After Permeability Test:

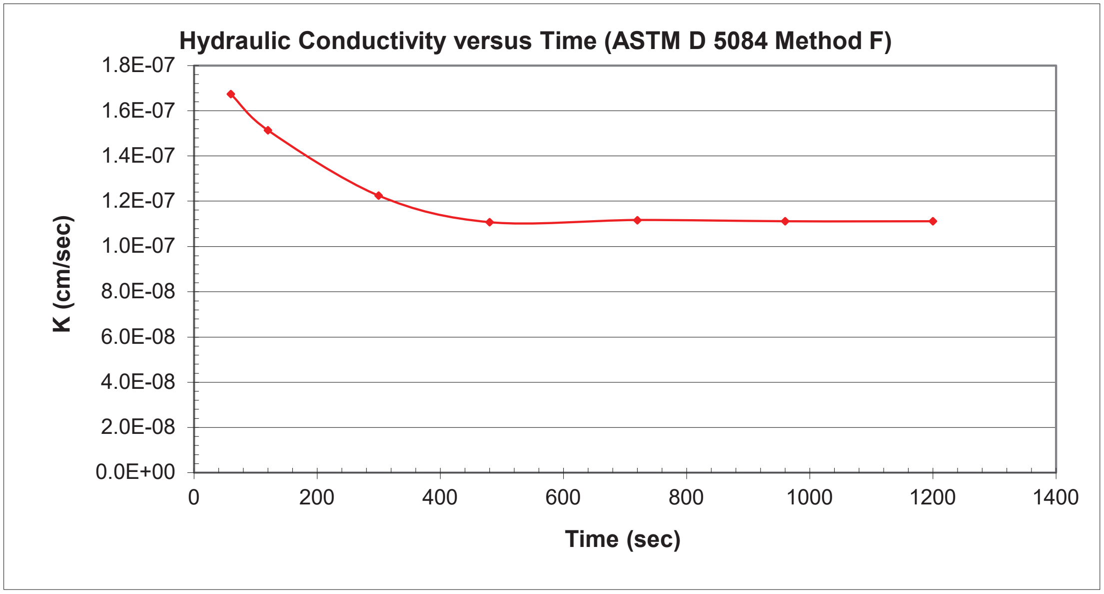
Final specimen height, h_f =	5.498	in =	13.965	cm
Final specimen diameter, d_f =	2.788	in =	7.082	cm
Final area of specimen, A_f =	6.105	sq in =	39.386	sq cm
Final volume of specimen, V_f =	33.564	cu in =	550.024	cu cm
Final weight of specimen, W_f =	1152.44	gram =	2.541	lbs
Final Moist Unit Weight, r_f =	130.80	pcf	Final moisture content, w_f =	18.5%
Final Dry Unit Weight, r_{df} =	110.40	pcf	Final degree of saturation, S_f =	94.8%

4. Type of Permeant Liquid: De-aired tap water Total Back Pressure: 35.0 psi

Cell Pressure: 69.0 psi Effective Overburden Pressure: 34.0 psi

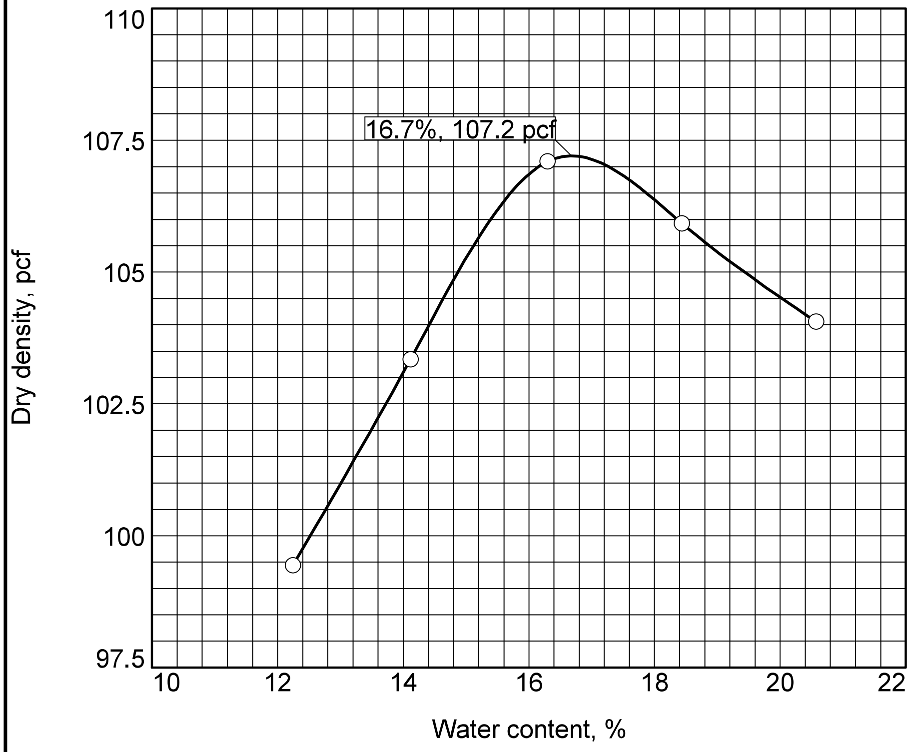
5. Corrected Hydraulic Conductivity, k_{20}

Temperature, T =	21.0	cel. deg.
Hydraulic Conductivity, k_T =	1.12E-07	cm/sec
Temperature Ratio, Rt =	0.976	
Hydraulic Conductivity, k_{20} =	**1.09E-07**	**cm/sec**



MOISTURE DENSITY RELATIONSHIP

Curve No.



Test Specification:
ASTM D 698 Method A Standard

Preparation Method
Hammer Wt. 5.5 lb.
Hammer Drop 12 in.
Number of Layers three
Blows per Layer 25
Mold Size 0.03333 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM **LL** 42 **PI** 16
Sp.G. (ASTM D 854)
%>#4 **%<No.200** 63.4
USCS CL **AASHTO**
Date Sampled
Date Tested
Tested By

TESTING DATA

	1	2	3	4	5	6
WM + WS	3680.9	3776.5	3876.5	3890.1	3890.4	
WM	1993.5	1993.5	1993.5	1993.5	1993.5	
WW + T #1	302.4	331.7	390.0	384.9	350.0	
WD + T #1	277.4	299.6	346.8	336.4	303.8	
TARE #1	73.2	72.2	81.7	73.1	79.0	
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	12.2	14.1	16.3	18.4	20.6	
DRY DENSITY	99.4	103.3	107.1	105.9	104.1	

TEST RESULTS

Maximum dry density = 107.2 pcf

Optimum moisture = 16.7 %

Material Description

Remarks:
CL soils composite from B-1 and B-2

Project No. G154-21 **Client:** Halff
Project: Spring Creek Dams

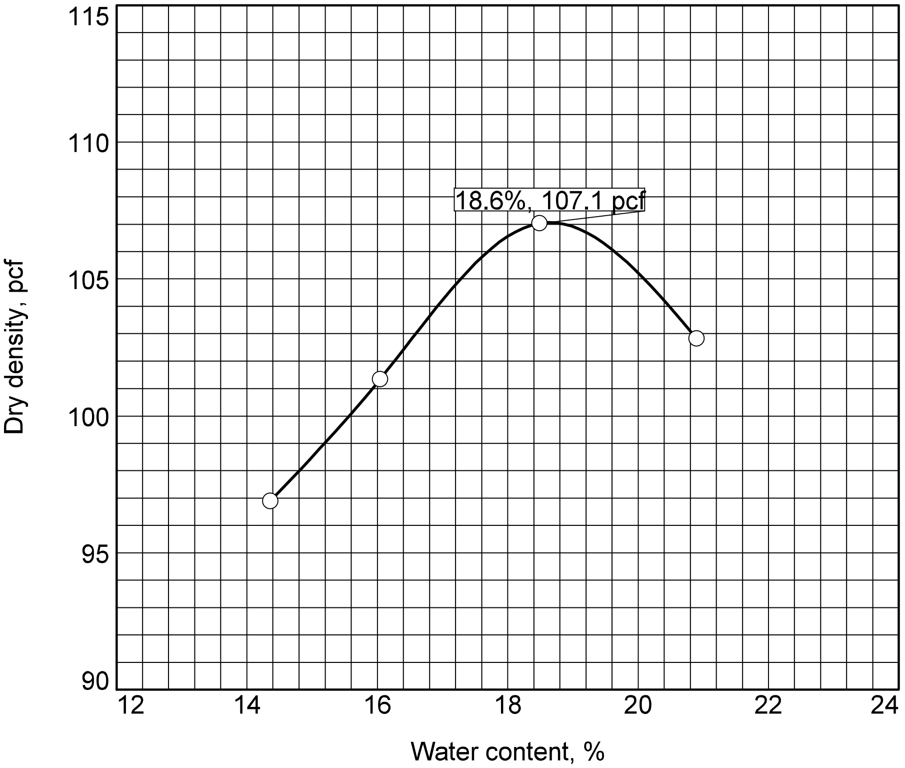
○ **Location:** Composite 1

Aviles Engineering Corp.

Checked by:
Title:

MOISTURE DENSITY RELATIONSHIP

Curve No.



Test Specification:
ASTM D 698 Method A Standard

Preparation Method

Hammer Wt. 5.5 lb.

Hammer Drop 12 in.

Number of Layers three

Blows per Layer 25

Mold Size 0.03333 cu. ft.

Test Performed on Material Passing #4 **Sieve**

NM **LL** 47 **PI** 17

Sp.G. (ASTM D 854)

%>#4 **%<No.200** 70.6

USCS CL **AASHTO**

Date Sampled

Date Tested

Tested By

TESTING DATA

	1	2	3	4	5	6
WM + WS	3668.8	3771.4	3910.9	3873.0		
WM	1993.5	1993.5	1993.5	1993.5		
WW + T #1	172.2	185.0	178.7	277.2		
WD + T #1	160.4	170.0	162.2	242.1		
TARE #1	78.3	76.5	72.7	73.9		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	14.4	16.0	18.5	20.9		
DRY DENSITY	96.9	101.3	107.0	102.8		

TEST RESULTS

Maximum dry density = 107.1 pcf

Optimum moisture = 18.6 %

Material Description

Remarks:
CL soils composite from B-3

Project No. G154-21 **Client:** Halff

Project: Spring Creek Dams

○ **Location:** Composite 2

Checked by:

Title:

Aviles Engineering Corp.

APPENDIX B

Plates B-1 to B-4	Baseline Soil Parameters by Individual Boring
Plates B-5 to B-6	Earth Dam Alternatives, from Black & Veatch email "SJRA Geotech Slides", dated June 14, 2024
Plate B-7	US Bureau of Land Reclamation, Design Standards No. 13: Embankment Dams, Chapter 16 "Cutoff Walls", Table 16.4.5-1 "Typical Gradation Limits for Soil-Bentonite Backfills", Page 16-26, July 2014

Baseline Soil Parameters by Individual Boring - Based on Boring B-1

Elevation/ Depth (ft)	Soil Type	γ_m (pcf)	Undrained Parameters		Effective Stress Parameters		Total Stress Parameters		Basis for Baseline Parameter
			C_u (psf)	ϕ_u (deg)	C' (psf)	ϕ' (deg)	C_{cu} (psf)	ϕ_{cu} (deg)	
250 to 244 (0 to 6)	SM/Firm CL	115	0	26	0	26	0	26	Typical Value
244 to 240 (6 to 10)	Stiff to very stiff CL/Medium dense SC	130	1,800	0	200	30	210	24	Test Results
240 to 236 (10 to 14)	Medium dense SC	129	1,000	0	200	30	210	24	
236 to 228 (14 to 22)	Medium dense SC	125	0	32	0	32	0	32	SPT Data
228 to 223 (22 to 27)	Medium dense SP-SM	120	0	30	0	30	0	30	
223 to 218 (27 to 32)	Medium dense SP-SM	120	0	28	0	28	0	28	
218 to 212 (32 to 38)	Stiff to very stiff CL	119	2,000	0	200	26	225	22	Test Results & Similar Soil
212 to 208 (38 to 42)	Medium dense SC	120	0	28	0	28	0	28	SPT Data
208 to 203 (42 to 47)	Very stiff CL	120	2,400	0	225	26	250	22	Test Results & Similar Soil
203 to 198 (47 to 52)	Medium dense SP-SM	120	0	32	0	32	0	32	SPT Data
198 to 188 (52 to 62)	Very stiff CH	112	2,200	0	300	24	325	18	Test Results & Typical Value
188 to 183 (62 to 67)	Firm to stiff CH	129	800	0	100	18	125	15	
183 to 163 (67 to 87)	Medium dense to dense SM/SP-SM	120	0	32	0	32	0	32	SPT Data
163 to 160 (87 to 90)	Very stiff CL	120	1,800	0	200	26	225	22	Test Results & Typical Value

Notes: (1) γ_m = moist unit weight of soil;

(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;

(3) C' =effective cohesion, ϕ' =effective friction angle, effective stress parameters that were determined from Consolidated-Undrained triaxial tests with pore water pressure measurements;

(4) C_{cu} = cohesion, ϕ_{cu} = friction angle, total stress parameters that were developed from Consolidated-Undrained triaxial tests;

(5) SM = Silty Sand, CL = Lean Clay, SC = Clayey Sand, SP-SM = Poorly Graded Sand with Silt, CH = Fat Clay.

Baseline Soil Parameters by Individual Boring - Based on Boring B-2

Elevation/ Depth (ft)	Soil Type	γ_m (pcf)	Undrained Parameters		Effective Stress Parameters		Total Stress Parameters		Basis for Baseline Parameter
			C_u (psf)	ϕ_u (deg)	C' (psf)	ϕ' (deg)	C_{cu} (psf)	ϕ_{cu} (deg)	
230 to 224 (0 to 6)	Very loose SM	115	0	26	0	26	0	26	SPT Data
224 to 216 (6 to 14)	Loose to medium dense SM	115	0	28	0	28	0	28	
216 to 214 (14 to 16)	Medium dense SM	115	0	32	0	32	0	32	
214 to 198 (16 to 32)	Stiff CL/Medium dense SC/Loose SP-SM	115	0	28	0	28	0	28	SPT Data & Typical Values
198 to 188 (32 to 42)	Medium dense SP-SM	120	0	30	0	30	0	30	
188 to 183 (42 to 47)	Hard CH	125	3,000	0	300	24	325	18	Test Results & Similar Soil
183 to 173 (47 to 57)	Hard CL	131	3,000	0	300	26	325	22	
173 to 159 (57 to 71)	Stiff to very stiff CL	131	1,600	0	200	26	225	22	
159 to 153 (71 to 77)	Medium dense SM	120	0	28	0	28	0	28	SPT Data
153 to 148 (77 to 82)	Stiff to very stiff CL	129	1,600	0	175	26	200	22	Test Results & Similar Soil
148 to 133 (82 to 97)	Very stiff to hard CL	131	3,000	0	300	26	325	22	
133 to 110 (97 to 120)	Dense to very dense SM/SP-SM	120	0	34	0	34	0	34	SPT Data

Notes: (1) γ_m = moist unit weight of soil;

(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;

(3) C' =effective cohesion, ϕ' =effective friction angle, effective stress parameters that were determined from Consolidated-Undrained triaxial tests with pore water pressure measurements;

(4) C_{cu} = cohesion, ϕ_{cu} = friction angle, total stress parameters that were developed from Consolidated-Undrained triaxial tests;

(5) SM = Silty Sand, CL = Lean Clay, SC = Clayey Sand, SP-SM = Poorly Graded Sand with Silt, CH = Fat Clay.

Baseline Soil Parameters by Individual Boring - Based on Boring B-3

Elevation/ Depth (ft)	Soil Type	γ_m (pcf)	Undrained Parameters		Effective Stress Parameters		Total Stress Parameters		Basis for Baseline Parameter
			C_u (psf)	ϕ_u (deg)	C' (psf)	ϕ' (deg)	C_{cu} (psf)	ϕ_{cu} (deg)	
230 to 226 (0 to 4)	Loose SM	115	0	28	0	28	0	28	SPT Data & Test Results
226 to 216 (4 to 14)	Medium dense SC	133	800	0	175	30	200	24	
216 to 198 (14 to 32)	Very loose to medium dense SC/SM	115	0	26	0	26	0	26	
198 to 188 (32 to 42)	Very stiff to hard CH	121	3,000	0	100	18	120	15	Test Results
188 to 183 (42 to 47)	Medium dense SM	120	0	32	0	32	0	32	SPT Data
183 to 178 (47 to 52)	Hard CL	120	3,000	0	300	26	325	22	Test Results & Similar Soil
178 to 173 (52 to 57)	Very stiff CL	120	1,800	0	200	26	225	22	
173 to 168 (57 to 62)	Very soft to stiff CL	115	800	0	75	26	90	22	
168 to 163 (62 to 67)	Medium dense SP-SM	120	0	30	0	30	0	30	SPT Data
163 to 148 (67 to 82)	Dense SP-SM	120	0	32	0	32	0	32	SPT Data
148 to 133 (82 to 97)	Medium dense SC	120	800	0	100	30	125	24	Test Results & Similar Soil
133 to 127 (97 to 103)	Very dense SP-SM	125	0	34	0	34	0	34	SPT Data
127 to 118 (103 to 112)	Medium dense to dense SC	120	0	32	0	32	0	32	SPT Data
118 to 113 (112 to 117)	Very dense SM	125	0	34	0	34	0	34	
113 to 110 (117 to 120)	Very stiff CL-ML	120	0	28	0	28	0	28	Typical Values

Notes: (1) γ_m = moist unit weight of soil;

(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;

(3) C' =effective cohesion, ϕ' =effective friction angle, effective stress parameters that were determined from Consolidated-Undrained triaxial tests with pore water pressure measurements;

(4) C_{cu} = cohesion, ϕ_{cu} = friction angle, total stress parameters that were developed from Consolidated-Undrained triaxial tests;

(5) SM = Silty Sand, CL = Lean Clay, SC = Clayey Sand, SP-SM = Poorly Graded Sand with Silt, CH = Fat Clay, CL-ML = Silty Clay with Sand.

Baseline Soil Parameters by Individual Boring - Based on Boring B-4

Elevation/ Depth (ft)	Soil Type	γ_m (pcf)	Undrained Parameters		Effective Stress Parameters		Total Stress Parameters		Basis for Baseline Parameter
			C_u (psf)	ϕ_u (deg)	C' (psf)	ϕ' (deg)	C_{cu} (psf)	ϕ_{cu} (deg)	
245 to 239 (0 to 6)	Medium dense SC	135	1,000	0	100	30	125	24	Test Results & Similar Soil
239 to 233 (6 to 12)	Dense SC-SM	120	0	32	0	32	0	32	SPT Data
233 to 218 (12 to 27)	Medium dense SC-SM/SP-SM	120	0	30	0	30	0	30	
218 to 213 (27 to 32)	Medium dense SP-SM	120	0	28	0	28	0	28	
213 to 208 (32 to 37)	Medium dense SP-SM	120	0	32	0	32	0	32	
208 to 203 (37 to 42)	Medium dense SP-SM	120	0	28	0	28	0	28	
203 to 198 (42 to 47)	Medium dense SP-SM	120	0	32	0	32	0	32	
198 to 193 (47 to 52)	Hard CH	123	3,000	0	120	18	145	15	Test Results & Similar Soil
193 to 183 (52 to 62)	SC	135	1,200	0	100	30	125	24	
183 to 168 (62 to 77)	Dense SP-SM	120	0	32	0	32	0	32	SPT Data
168 to 163 (77 to 82)	Dense SP	120	0	32	0	32	0	32	
163 to 155 (82 to 90)	Dense to very dense SP	125	0	34	0	34	0	34	

Notes: (1) γ_m = moist unit weight of soil;

(2) C_u =undrained cohesion, ϕ_u = angle of internal friction, under short term conditions. UU = strength parameters that were determined from Unconsolidated-Undrained triaxial tests;

(3) C' =effective cohesion, ϕ' =effective friction angle, effective stress parameters that were determined from Consolidated-Undrained triaxial tests with pore water pressure measurements;

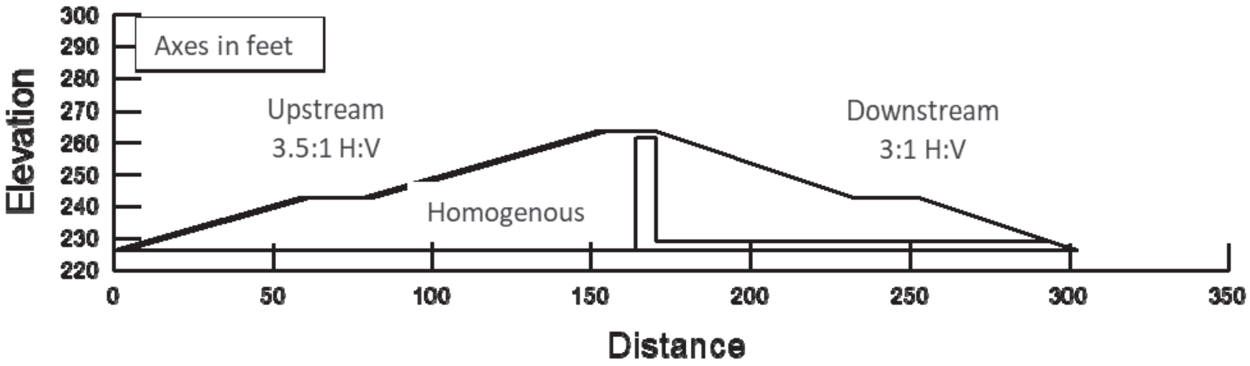
(4) C_{cu} = cohesion, ϕ_{cu} = friction angle, total stress parameters that were developed from Consolidated-Undrained triaxial tests;

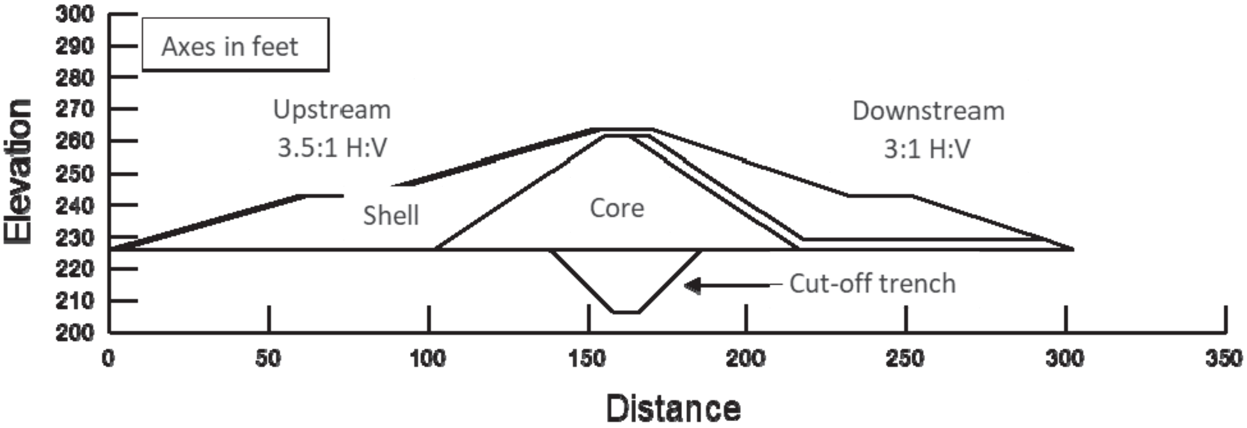
(5) SM = Silty Sand, CL = Lean Clay, SC = Clayey Sand, SP-SM = Poorly Graded Sand with Silt, CH = Fat Clay, CL-ML = Silty Clay with Sand.

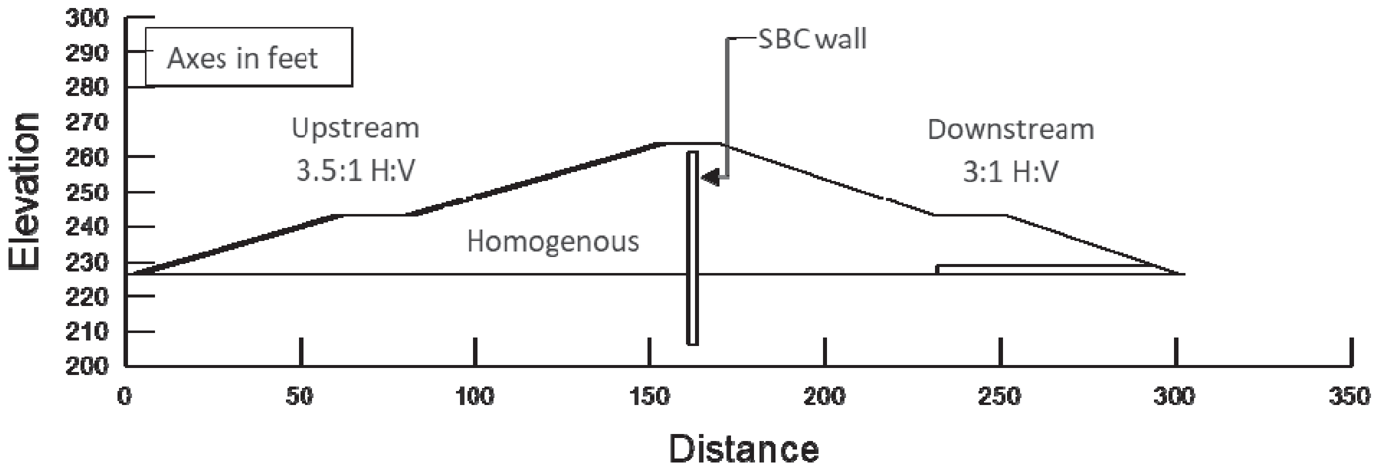
From: Pedarla, Aravind <PedarlaA@bv.com>
Sent: Friday, June 14, 2024 3:15 PM
To: wwang@avilesengineering.com
Cc: Turkson, Prince
Subject: SJRA Geotech Slides

Wilber,

Here are the various cross sections of the dam. Let us know if you have any questions.







Regards

Aravind Pedarla, Ph.D., P.E.*
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have been used with successful results as shown in table 16.4.5-1. As a general guide, the following gradation ranges that have been used in Europe, Australia, and the United States can be a guide to the designer when determining the appropriate backfill.

Table 16.4.5-1. Typical Gradation Limits for Soil-Bentonite Backfills (Europe, Australia, United States)

	Europe and Australia	United States
Screen size	Percent passing by weight	Percent passing by weight
3 inches	80 - 100	80 - 100
3/4 inch	40 - 100	50 - 100
No. 4	30 - 70	30 - 70
No. 30	20 - 50	25 - 60
No. 200	10 - 25	10 - 30

The designer should understand that these typical gradations are only general guides. Each case will be different, and filter compatibility of the trench backfill and in situ material should be satisfied, especially if high gradients are anticipated across the cutoff wall. In some cases where a cutoff wall is excavated through very coarse material, the excavated material mixed with bentonite slurry will not be an acceptable backfill because it may be too coarse or internally unstable. In some instances, a borrow source or commercial source of soil material can provide an acceptable backfill material that will produce a lower permeability and meet filter criteria for the surrounding in situ material. This was the case at Reclamation’s Diamond Creek Dike [23], Keechelus Dam [39], and Bradbury Dam [24]. The excavated material mixed with bentonite slurry was too coarse and gap-graded to serve as adequate backfill. Therefore, a more compatible backfill was imported, mixed with fresh bentonite slurry, and placed into the trench. If the risk of filter incompatibility leading to loss of backfill is high and/or blowout is a concern, due to high seepage gradients, the designer may want to consider the use of a different type of cutoff wall backfill.

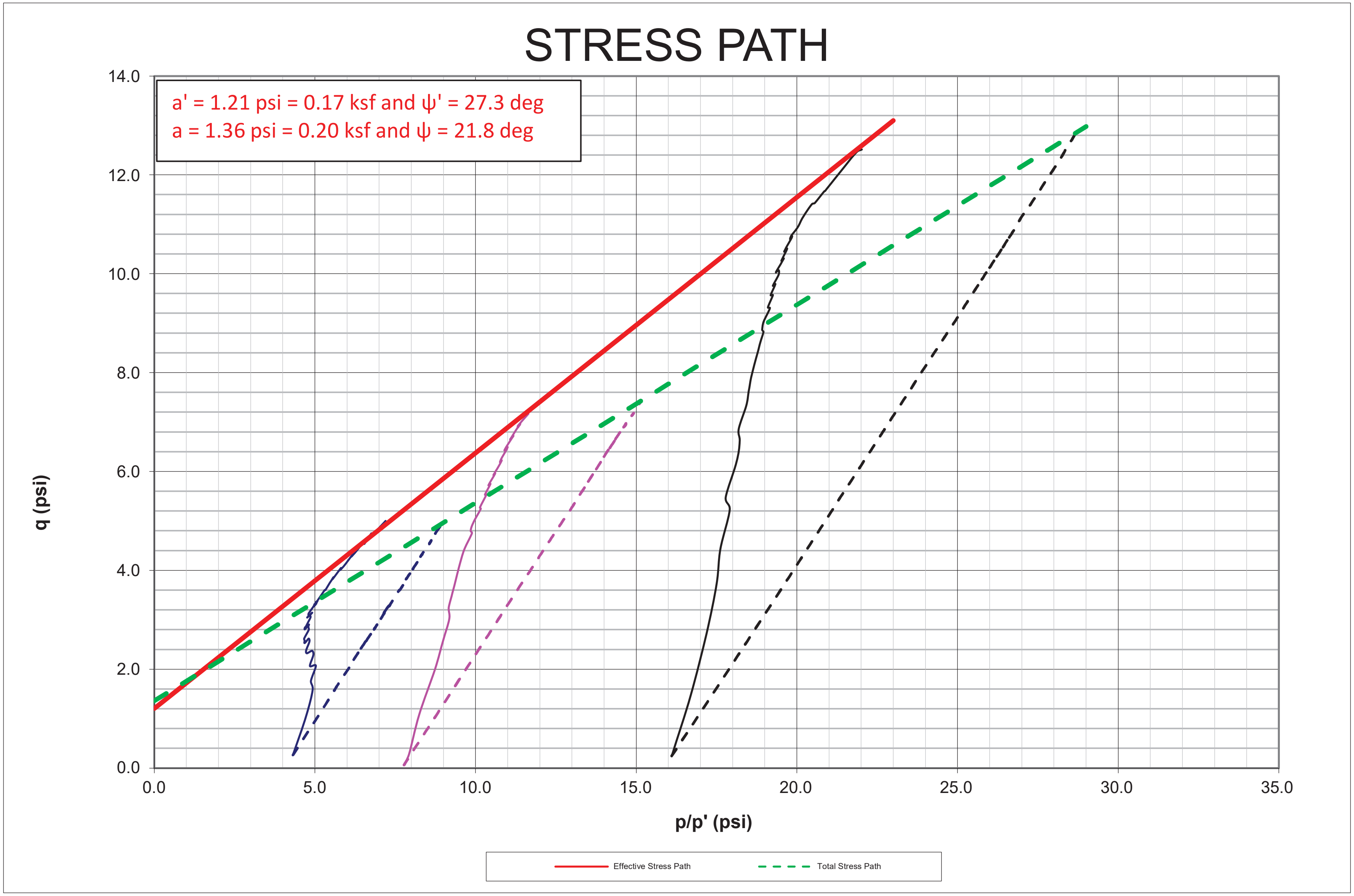
16.4.6 Embankment Core/Earth Backfilled Trench Connection

The connection of the embankment core with the top of the cutoff wall requires particular attention if the integrity of the system is to be maintained. In general, arching effects will severely limit the amount of backfill consolidation that occurs under its own weight. Measured settlements of an inch to less than a few inches are common. Increased settlement, due to embankment loading on the trench, will also be limited to the upper portion of backfill due to arching. However,

APPENDIX C-1

CU Test Results, Boring B-1, 8’-10’

G154-21 Boring B-1, 8'-10'



Notes:

1. Value of p and q at failure is determined considering maximum effective stress obliquity according to ASTM D6747 for each stage of CU test and plot them on the respective p-q curves.
2. Then a best-fit straight line will be drawn to fit the data and the slope (tanψ) and intercept (d) will be determined according to US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4.
3. Then ϕ and c will be determined based on equation D-7 and D-8 (US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4).

Proj. # G154-21 **Boring:** B-1 **Depth (ft):** 8-10

Initial Height Measurements (in.):
Height 1: 5.537
Height 2: 5.57
Height 3: 5.54
Average Height (in.): 5.549

Initial Diameter Measurements (in.):
Diam 1: 2.797
Diam 2: 2.8
Diam 3: 2.801
Average Diameter (in): 2.799333

Initial Dial Gauge Reading (in): 0.15

End of Saturation Dial Gauge Reading (in.): 0.175

First Consolidation: *(if there is no first stage consolidation, enter '0' for initial and final pipette readings, copy DGs to DGc)*
Initial Pipette Reading (mL): 0
Final Pipette Reading (mL): 0
Final Dial Gauge Reading (in.): 0.175

Beginning of First Shear:
Height: 5.524
Diameter: 2.787

End of First Shear:
Dial Gauge Reading at end of shearing (in.): 0.321
Dial Gauge Reading after CV rebound (in): 0.291

Second Stage Consolidation:
Initial Pipette Reading (mL): 22.5
Final Pipette Reading (mL): 20.2
Final Dial Gauge Reading (in.): 0.294

Beginning of Second Shear:
Height: 5.405
Diameter: 2.823

End of Second Shear:
Dial Gauge Reading at end of shearing (in.): 0.382
Dial Gauge Reading after CV rebound (in): 0.357

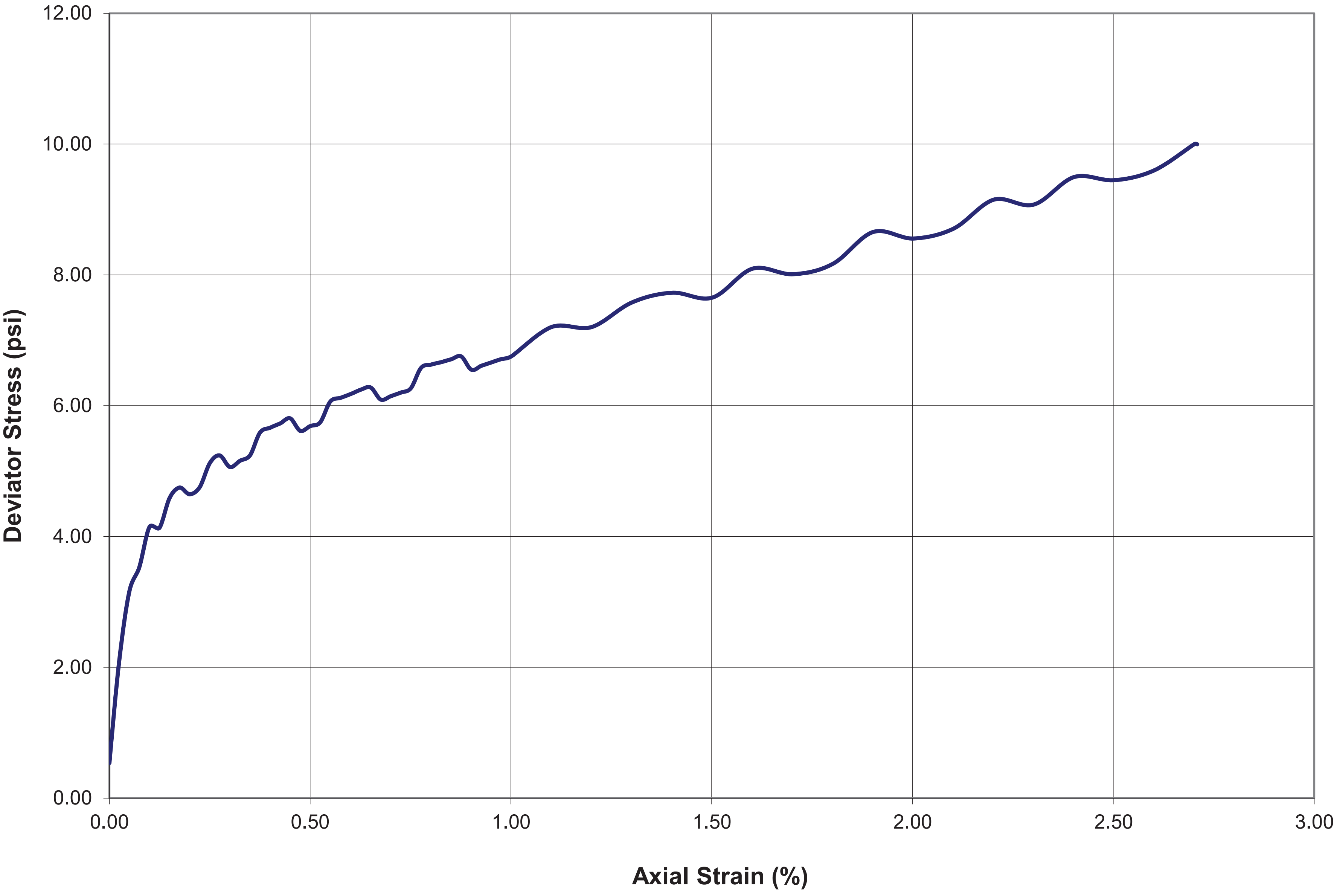
Third Stage Consolidation:
Initial Pipette Reading (mL): 22.95
Final Pipette Reading (mL): 19.3
Final Dial Gauge Reading (in.): 0.362

Beginning of Third Shear:
Height: 5.337
Diameter: 2.850

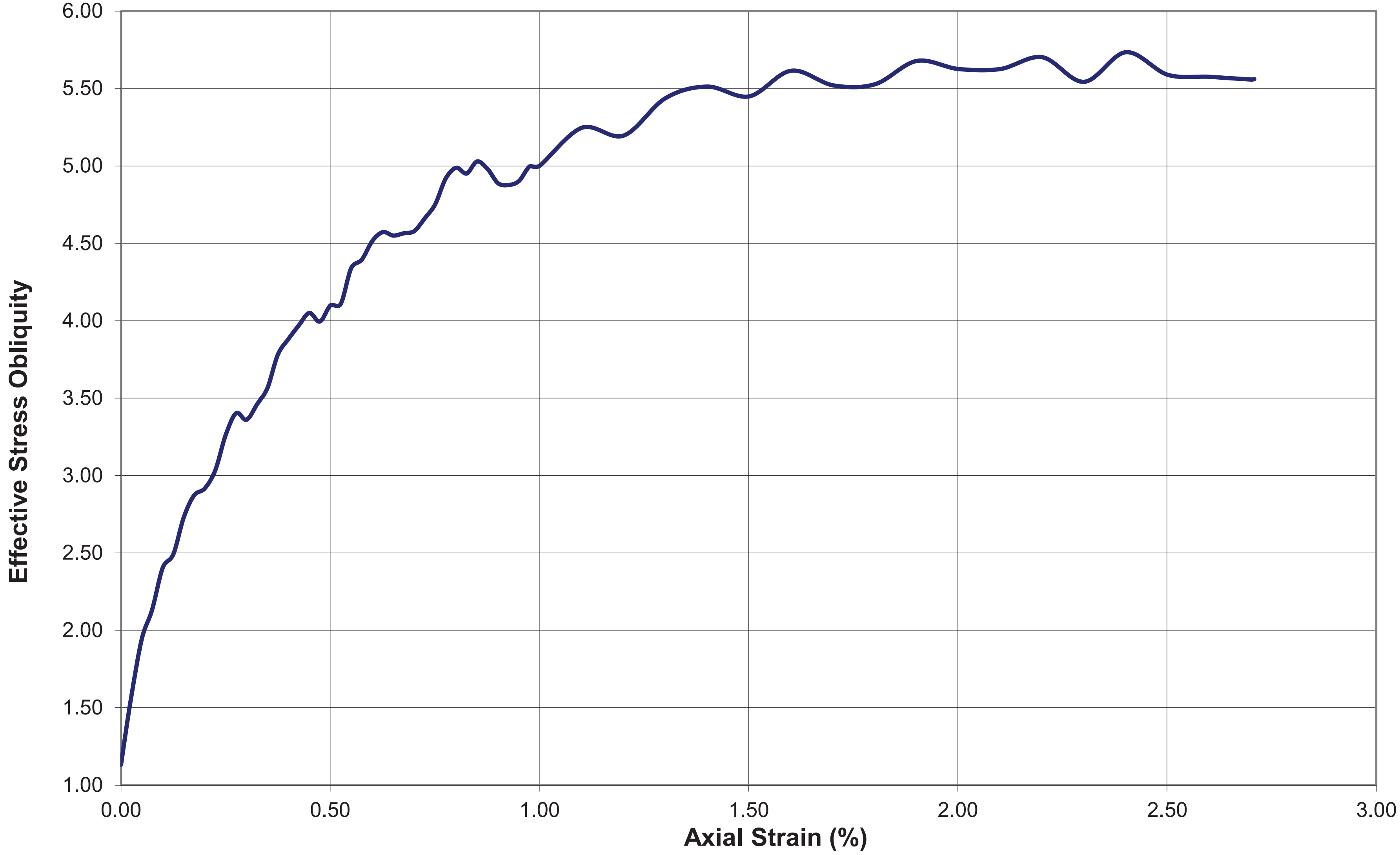
STAGE 1

B-1 8’-10’, at 4 psi confining pressures

G154-21-B-1-8-10 STAGE 1 SHEAR

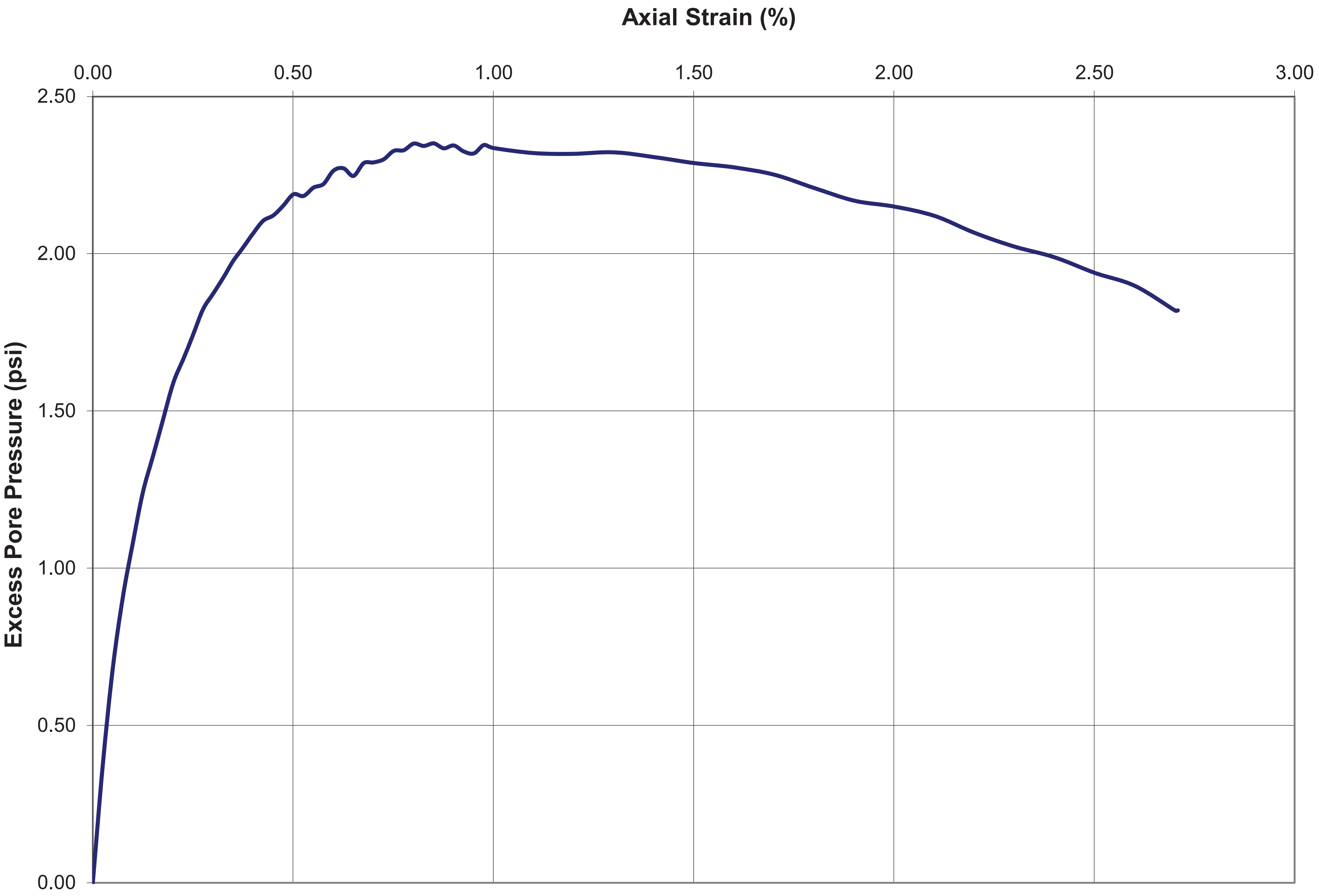


G154-21-B-1-8-10 STAGE 1 SHEAR

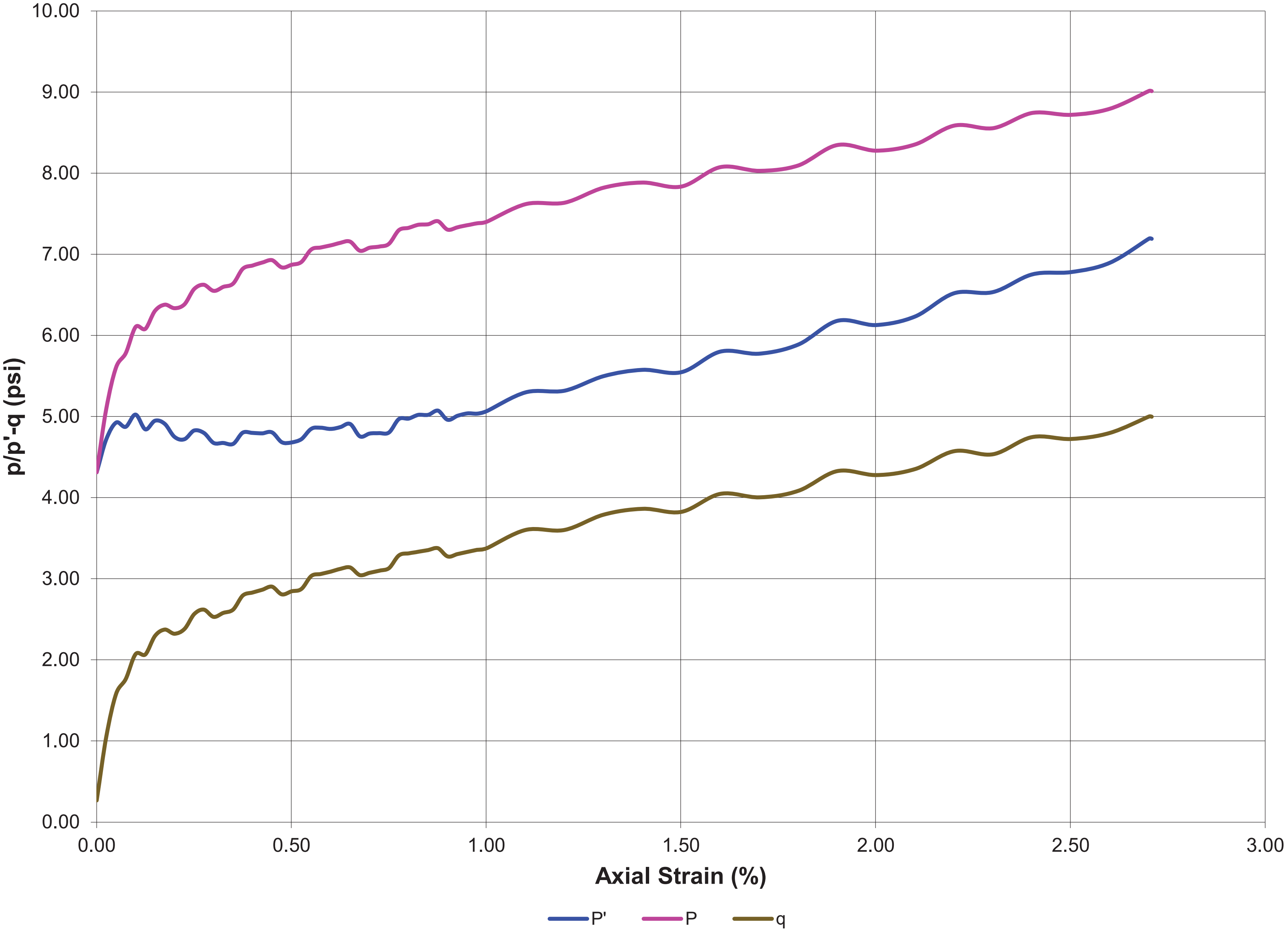


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 2.40%

G154-21-B-1-8-10 STAGE 1 SHEAR



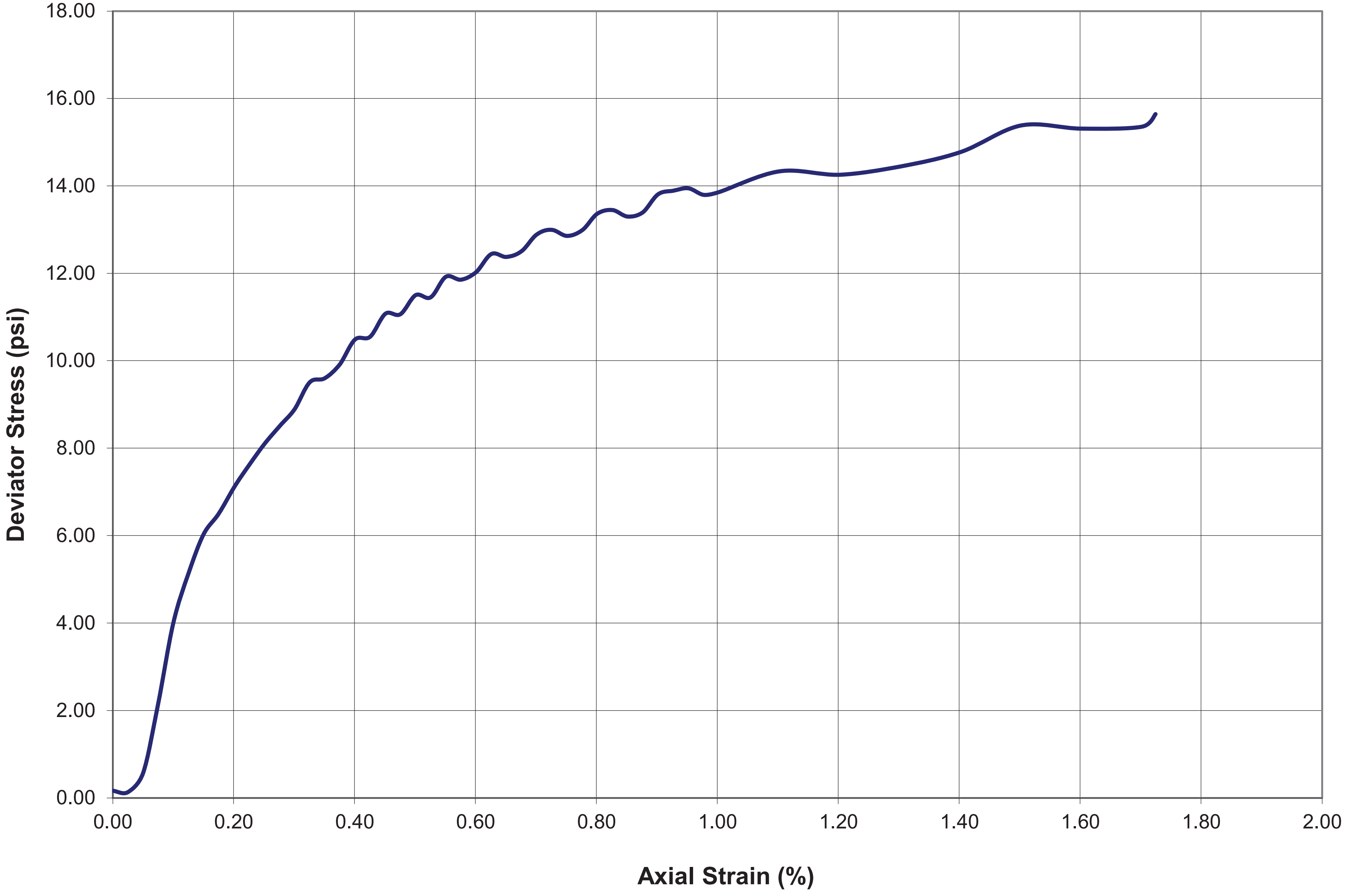
G154-21-B-1-8-10 STAGE 1 SHEAR



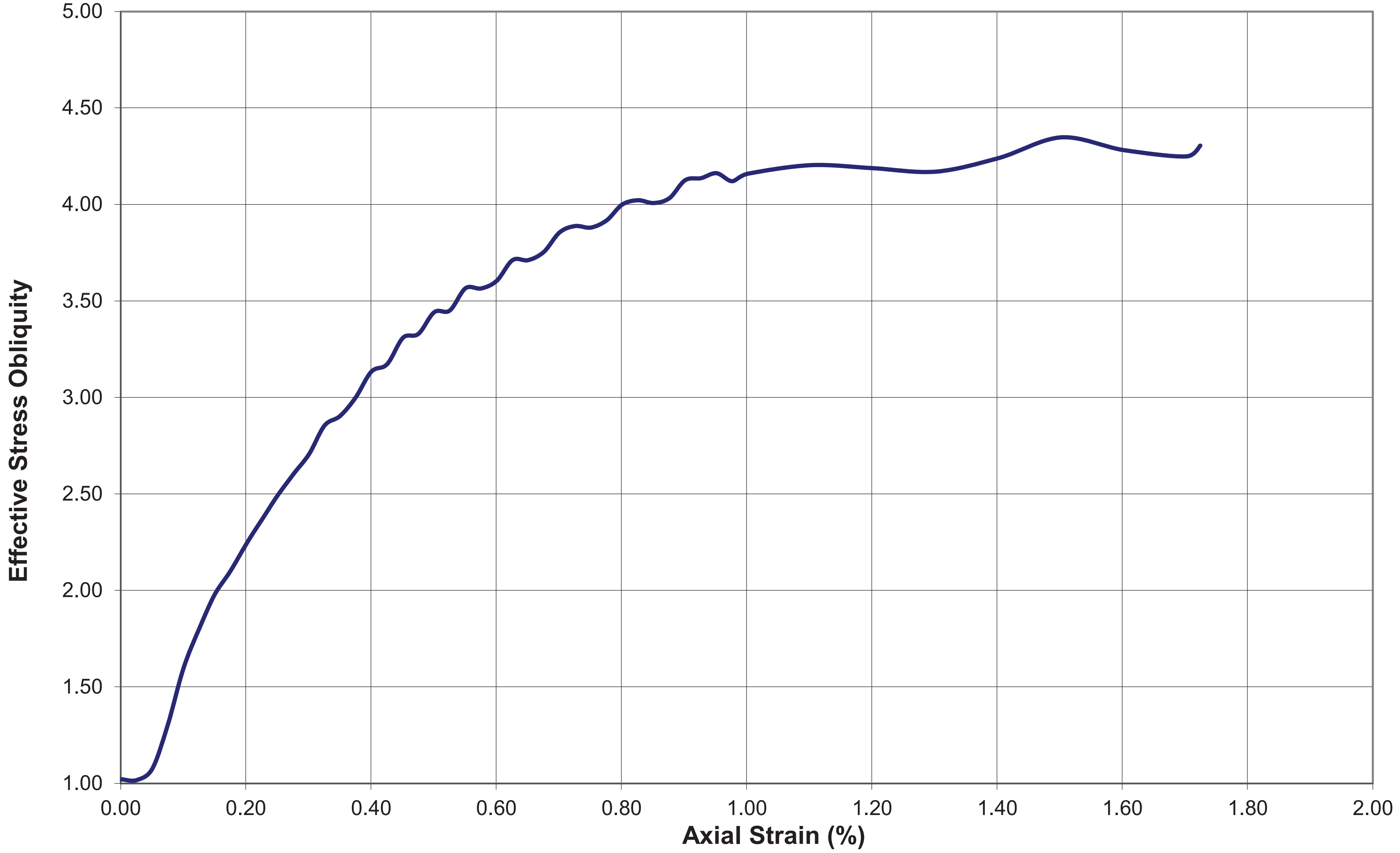
STAGE 2

B-1 8’-10’, at 8 psi confining pressures

G154-21-B-1-8-10 STAGE 2 SHEAR

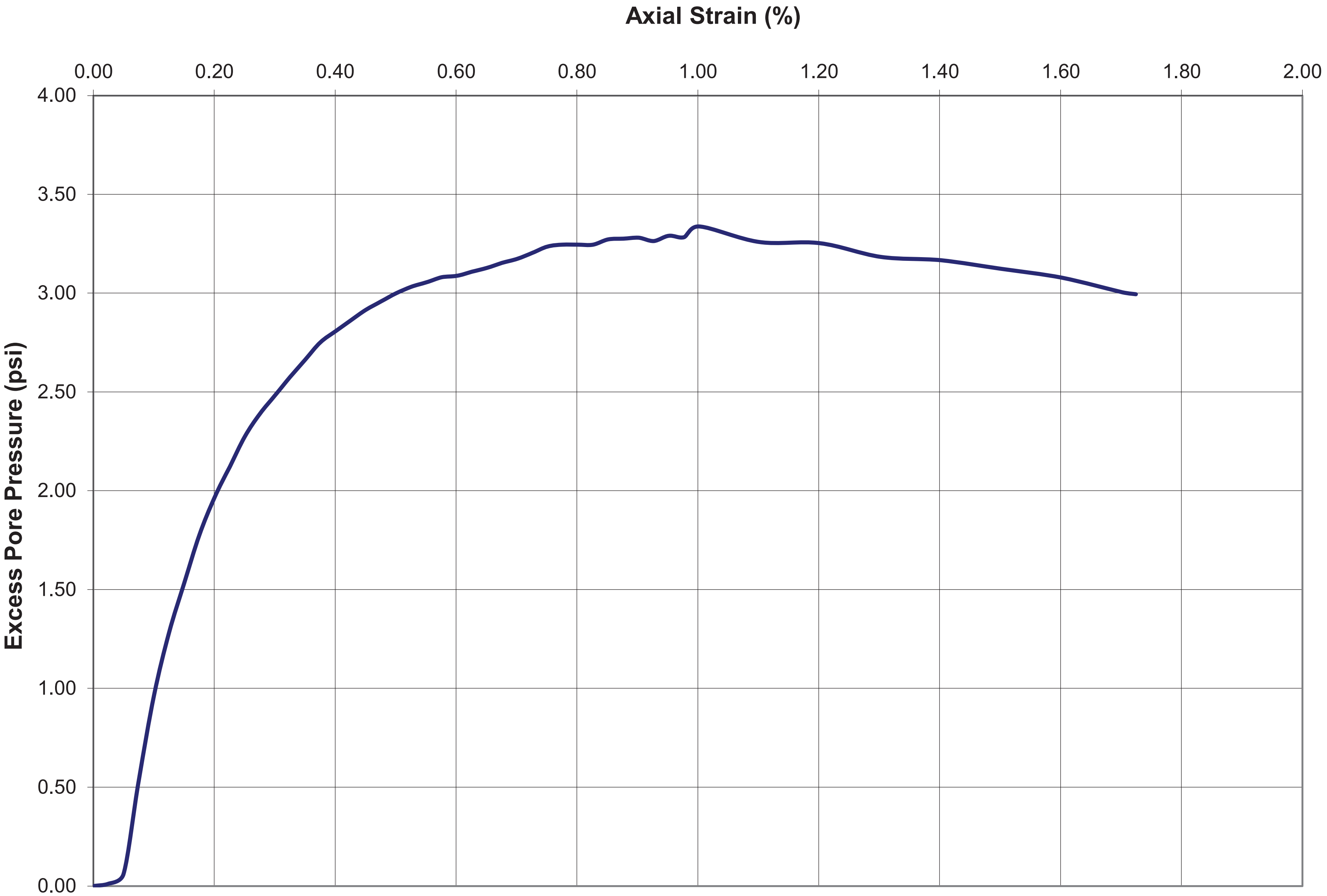


G154-21-B-1-8-10 STAGE 2 SHEAR

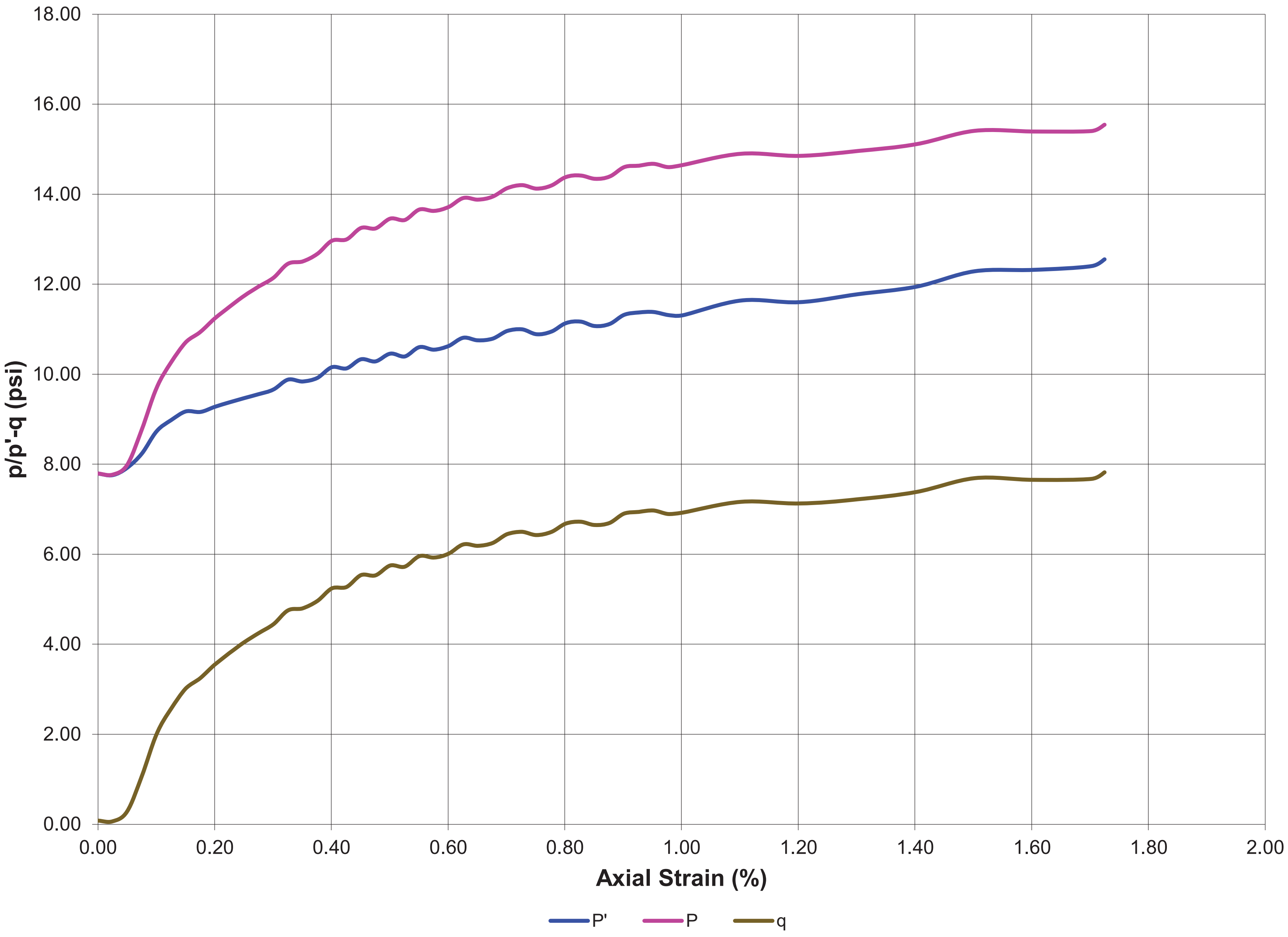


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 1.50%

G154-21-B-1-8-10 STAGE 2 SHEAR

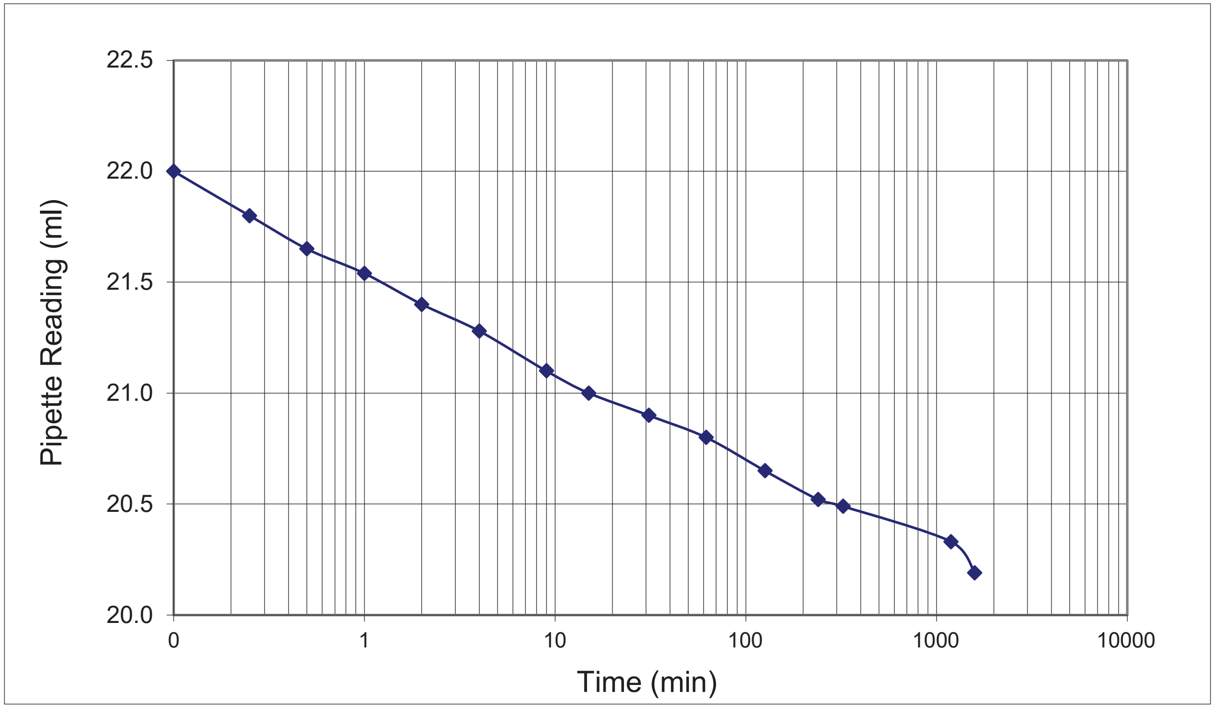


G154-21-B-1-8-10 STAGE 2 SHEAR



G154-21-B-1-8-10 STAGE 2 CONS

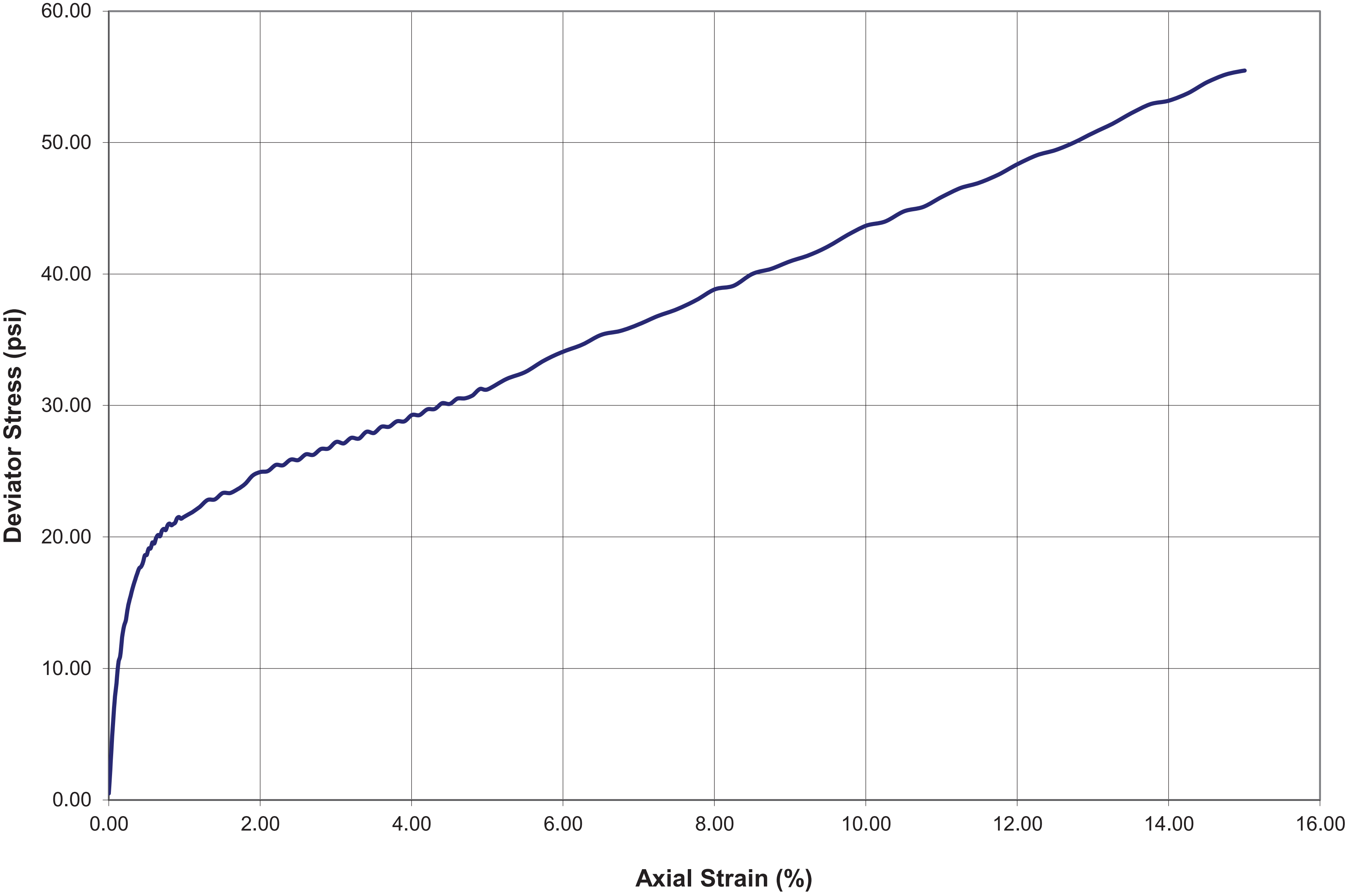
Time (min)	Pipette Reading (ml)
0.00	#REF!
0.10	22.00
0.25	21.80
0.50	21.65
1.00	21.54
2.00	21.40
4.00	21.28
9.00	21.10
15.00	21.00
31.00	20.90
62.00	20.80
126.00	20.65
240.00	20.52
324.00	20.49
1193.00	20.33
1582.00	20.19



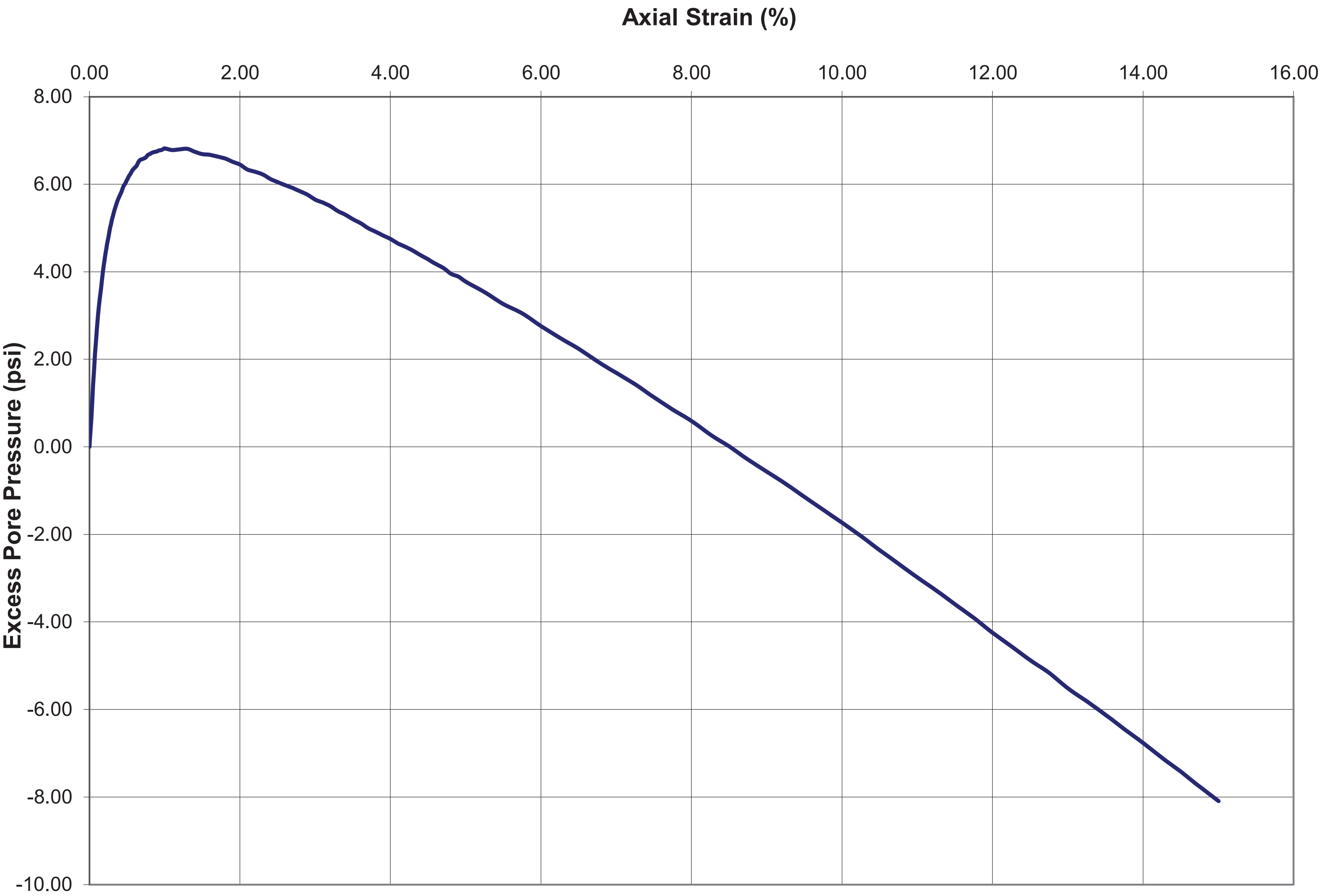
STAGE 3

B-1 8’-10’, at 16 psi confining pressures

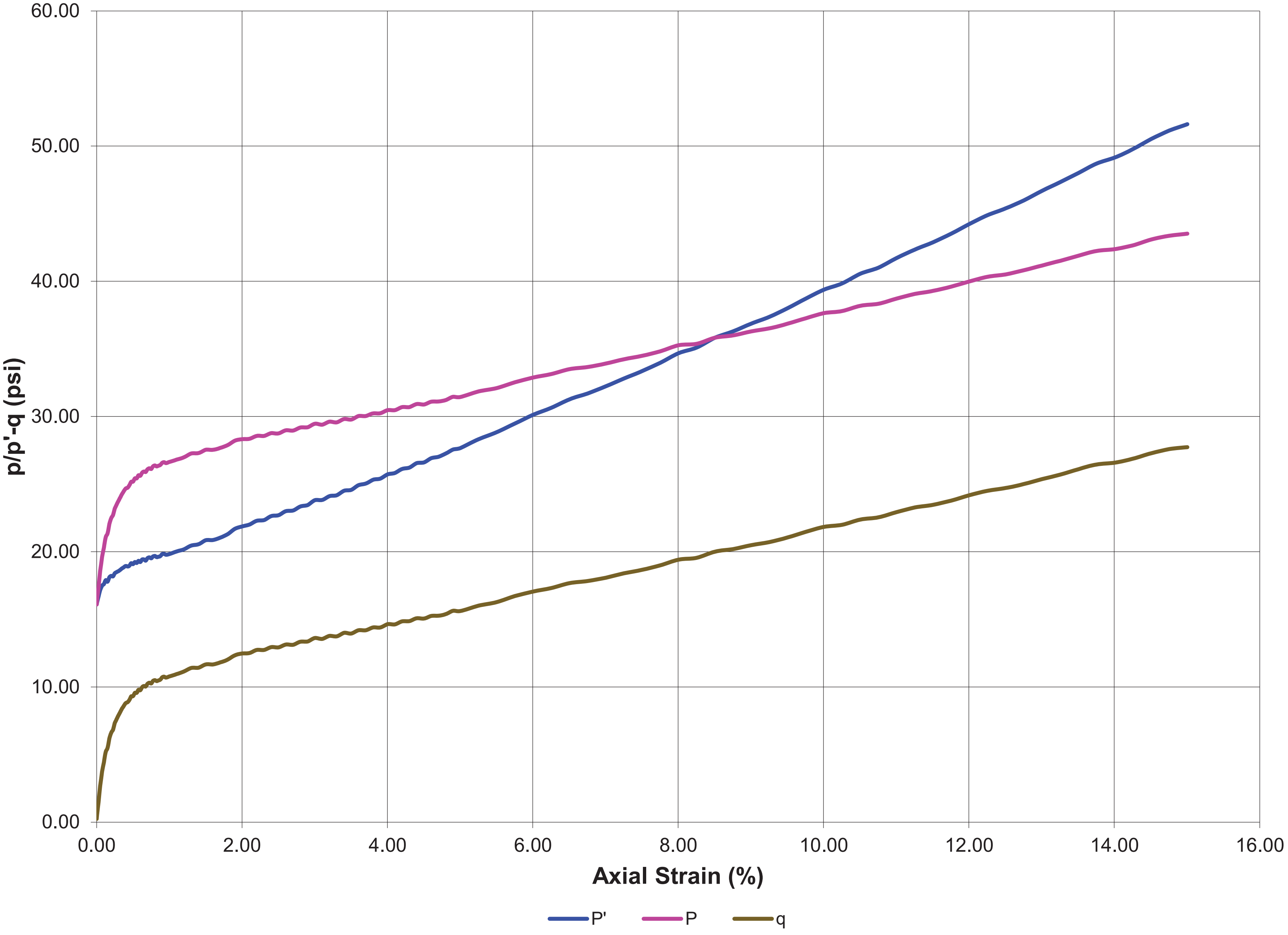
G154-21-B-1-8-10 STAGE 3 SHEAR



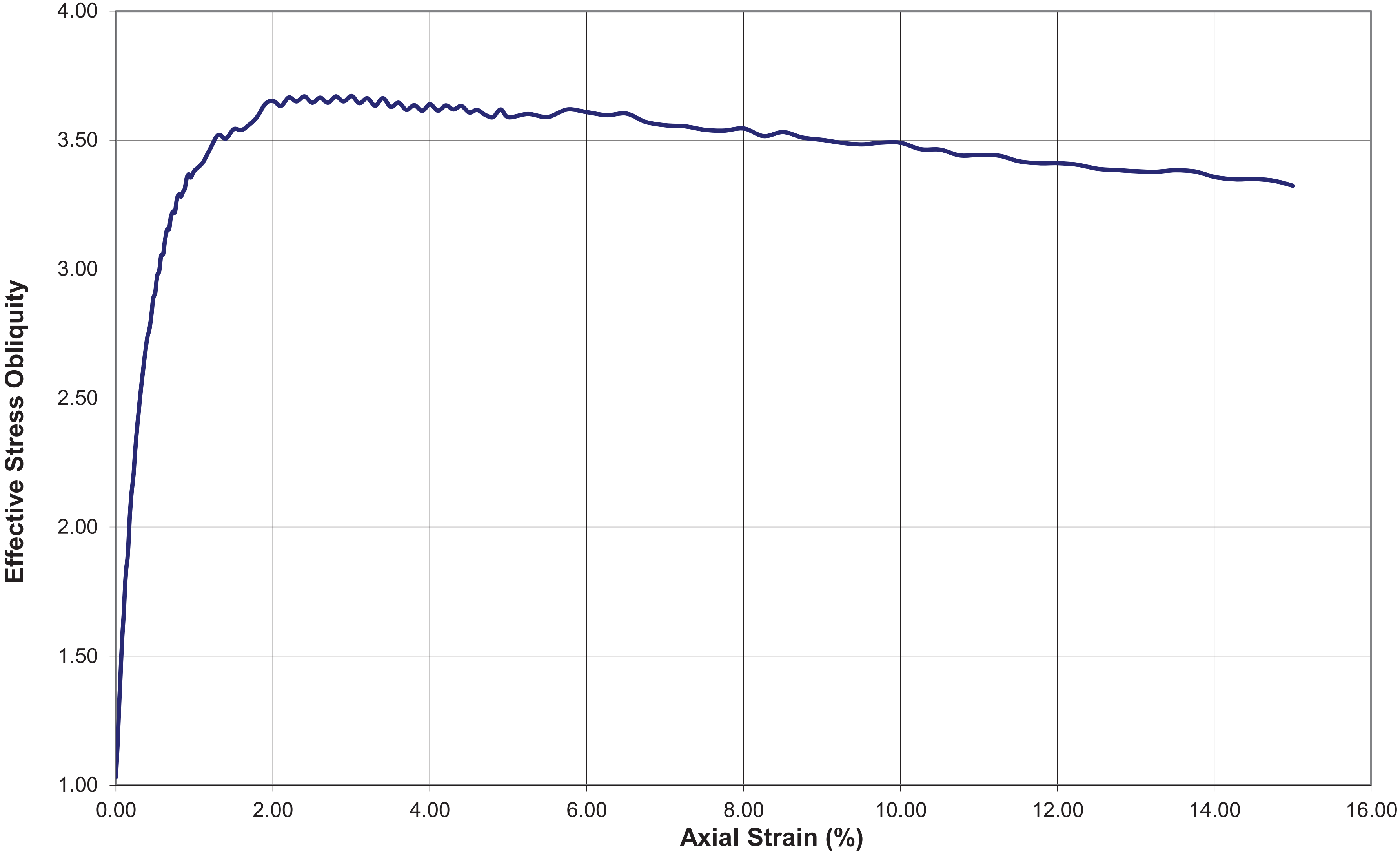
G154-21-B-1-8-10 STAGE 3 SHEAR



G154-21-B-1-8-10 STAGE 3 SHEAR



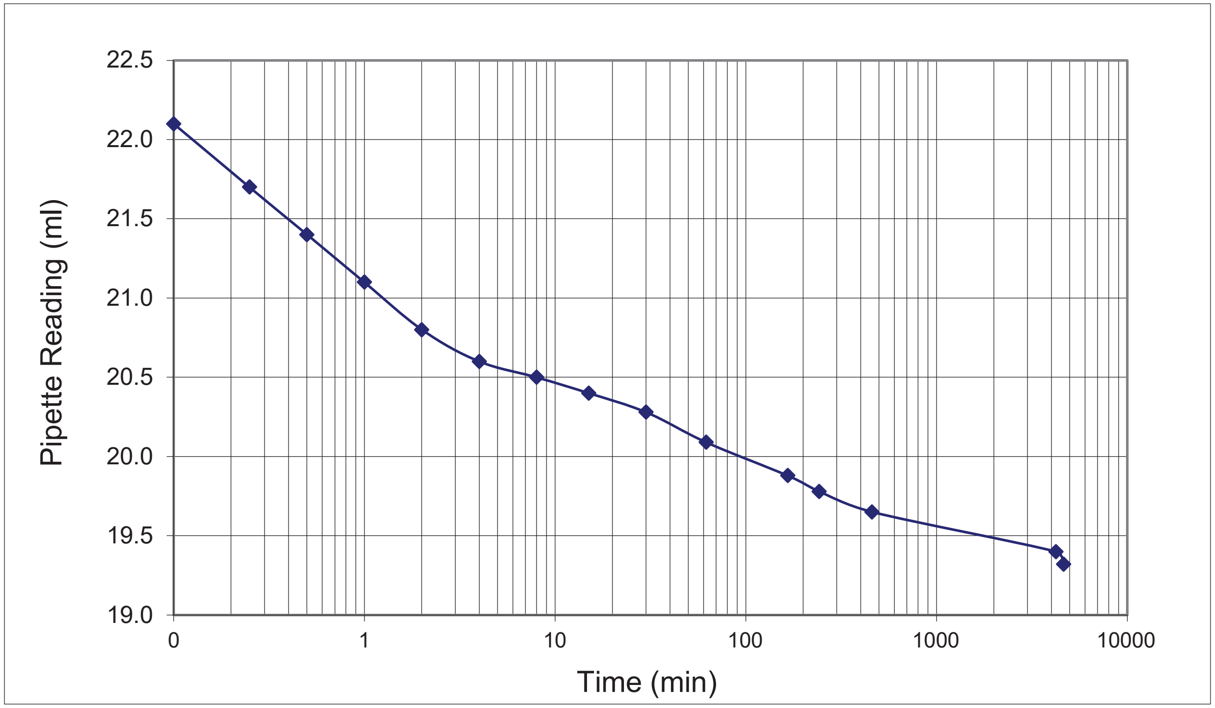
G154-21-B-1-8-10 STAGE 3 SHEAR



Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 2.20%

G154-21-B-1-8-10 STAGE 3 CONS

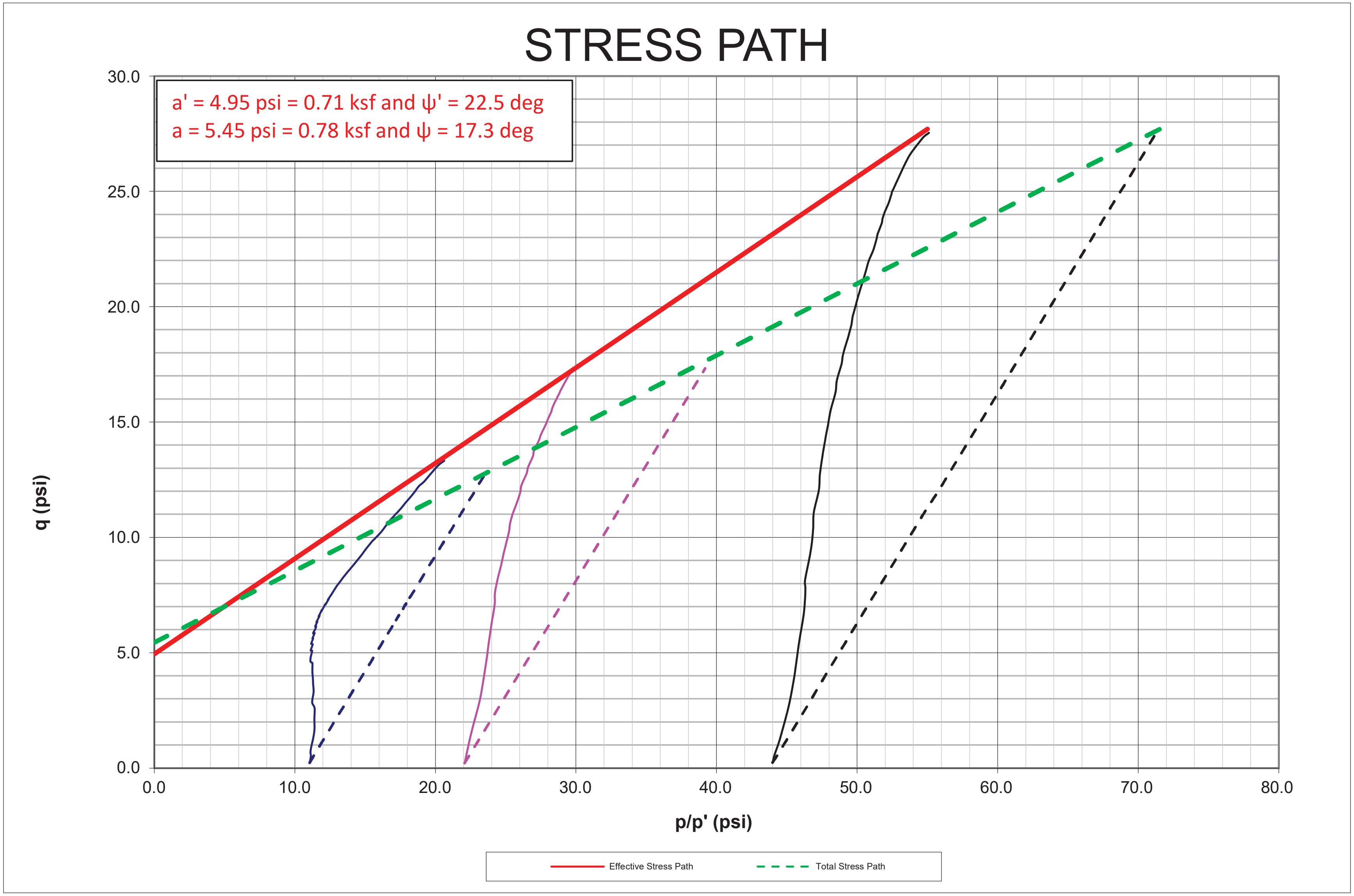
Time (min)	**Pipette Reading (ml)**
0.00	#REF!
0.10	22.10
0.25	21.70
0.50	21.40
1.00	21.10
2.00	20.80
4.00	20.60
8.00	20.50
15.00	20.40
30.00	20.28
62.00	20.09
166.00	19.88
242.00	19.78
459.00	19.65
4225.00	19.40
4645.00	19.32



APPENDIX C-2

CU Test Results, Boring B-2, 43’-45’

G154-21 Boring B-2, 43'-45'



Notes:

1. Value of p and q at failure is determined considering maximum effective stress obliquity according to ASTM D4767 for each stage of CU test and plot them on the respective p-q curves.
2. Then a best-fit straight line will be drawn to fit the data and the slope (tanψ) and intercept (d) will be determined according to US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4.
3. Then ϕ and c will be determined based on equation D-7 and D-8 (US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4).

Proj. # G154-21 **Boring:** B-2 **Depth (ft):** 43-45

Initial Height Measurements (in.):
Height 1: 5.6
Height 2: 5.618
Height 3: 5.613
Average Height (in.): 5.610333

Initial Diameter Measurements (in.):
Diam 1: 2.783
Diam 2: 2.789
Diam 3: 2.796
Average Diameter (in): 2.789333

Initial Dial Gauge Reading (in): 0.15

End of Saturation Dial Gauge Reading (in.): 0.136

First Consolidation: *(if there is no first stage consolidation, enter '0' for initial and final pipette readings, copy DGs to DGc)*
Initial Pipette Reading (mL): 23.1
Final Pipette Reading (mL): 20.15
Final Dial Gauge Reading (in.): 0.149

Beginning of First Shear:
Height: 5.611
Diameter: 2.792

End of First Shear:
Dial Gauge Reading at end of shearing (in.): 0.325
Dial Gauge Reading after CV rebound (in): 0.294

Second Stage Consolidation:
Initial Pipette Reading (mL): 23.1
Final Pipette Reading (mL): 18.84
Final Dial Gauge Reading (in.): 0.302

Beginning of Second Shear:
Height: 5.458
Diameter: 2.842

End of Second Shear:
Dial Gauge Reading at end of shearing (in.): 0.396
Dial Gauge Reading after CV rebound (in): 0.373

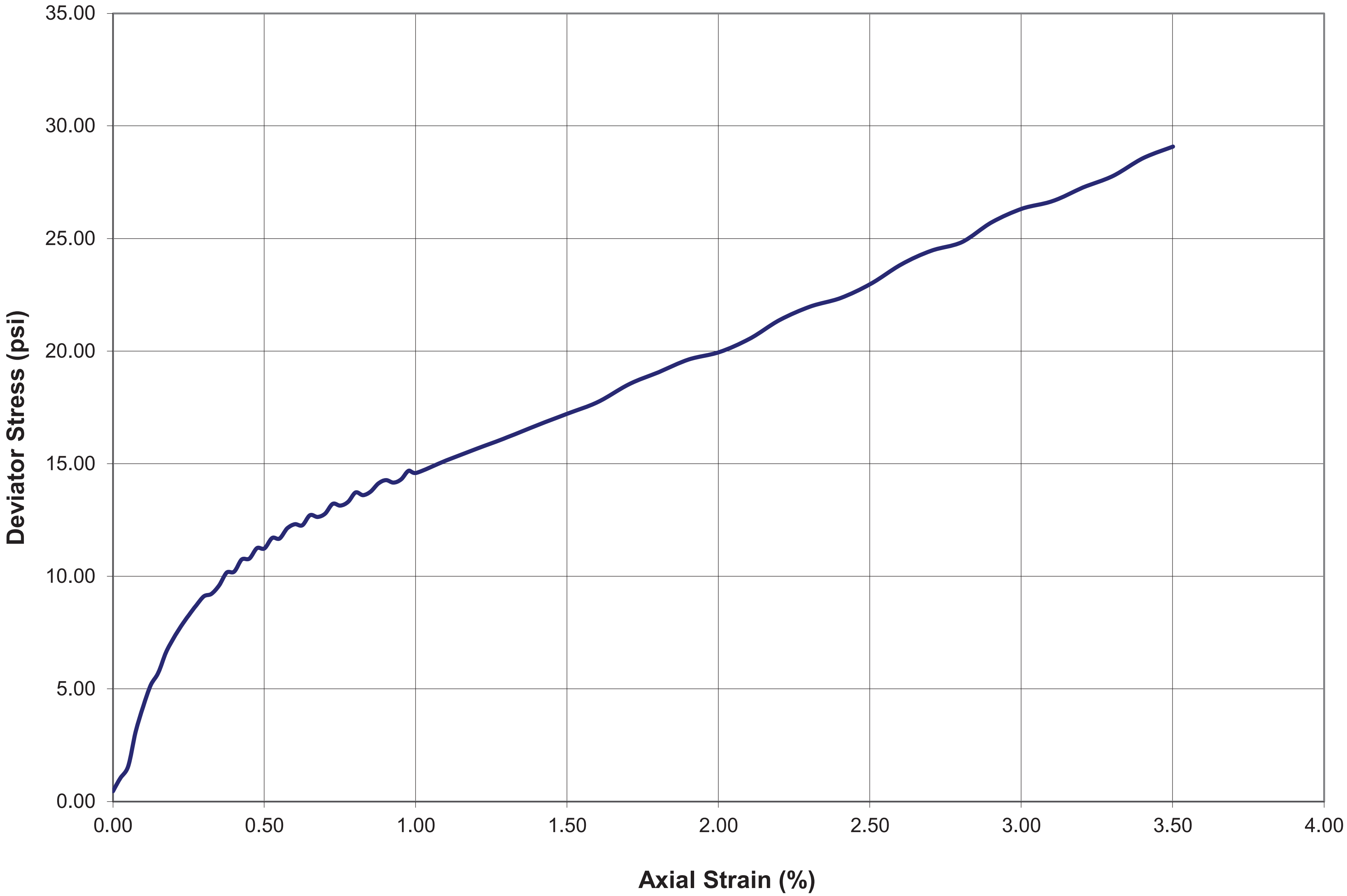
Third Stage Consolidation:
Initial Pipette Reading (mL): 23.5
Final Pipette Reading (mL): 18
Final Dial Gauge Reading (in.): 0.38

Beginning of Third Shear:
Height: 5.380
Diameter: 2.876

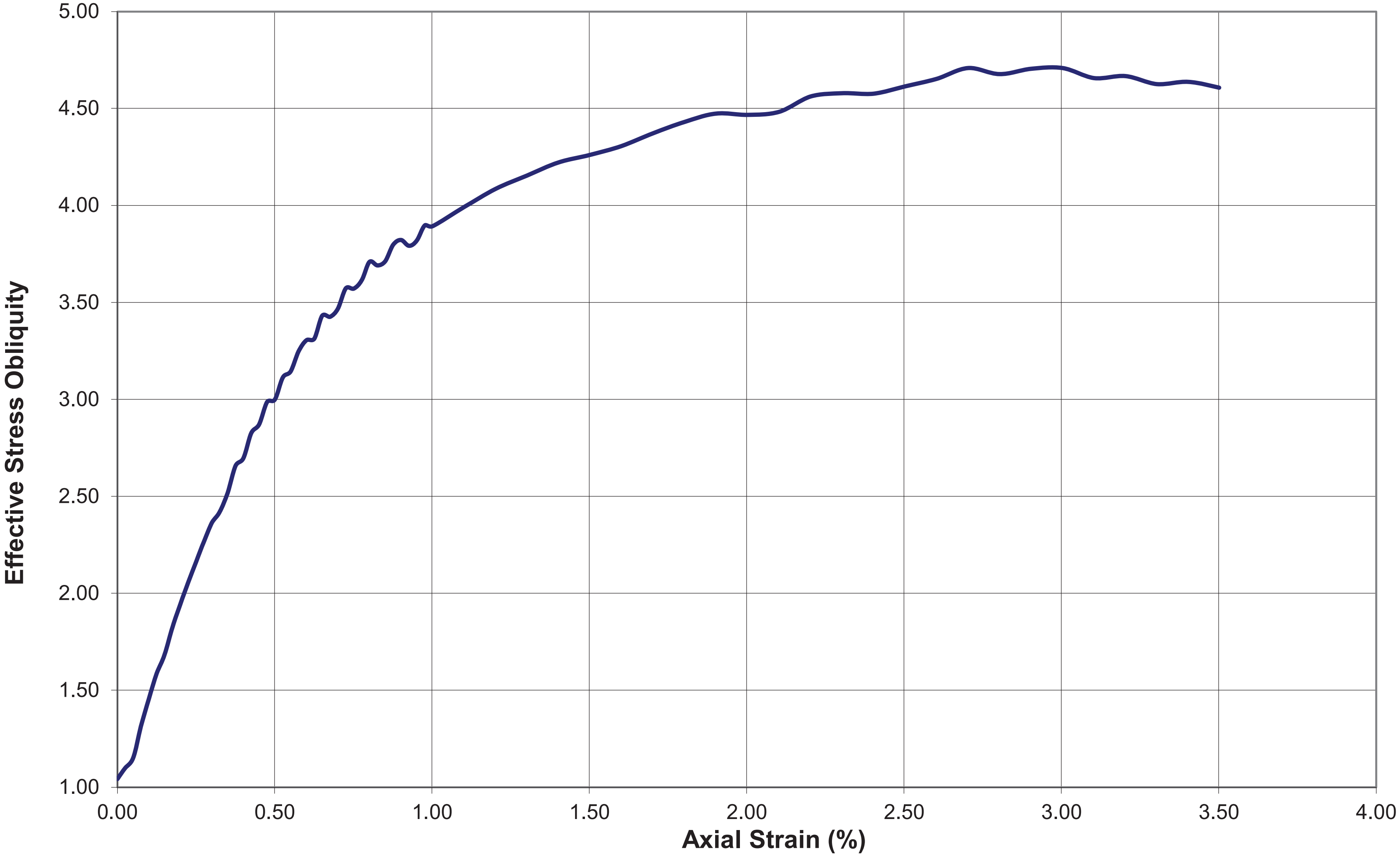
STAGE 1

B-2 43’-45’, at 11 psi confining pressures

G154-21-B-2-43-45 STAGE 1 SHEAR

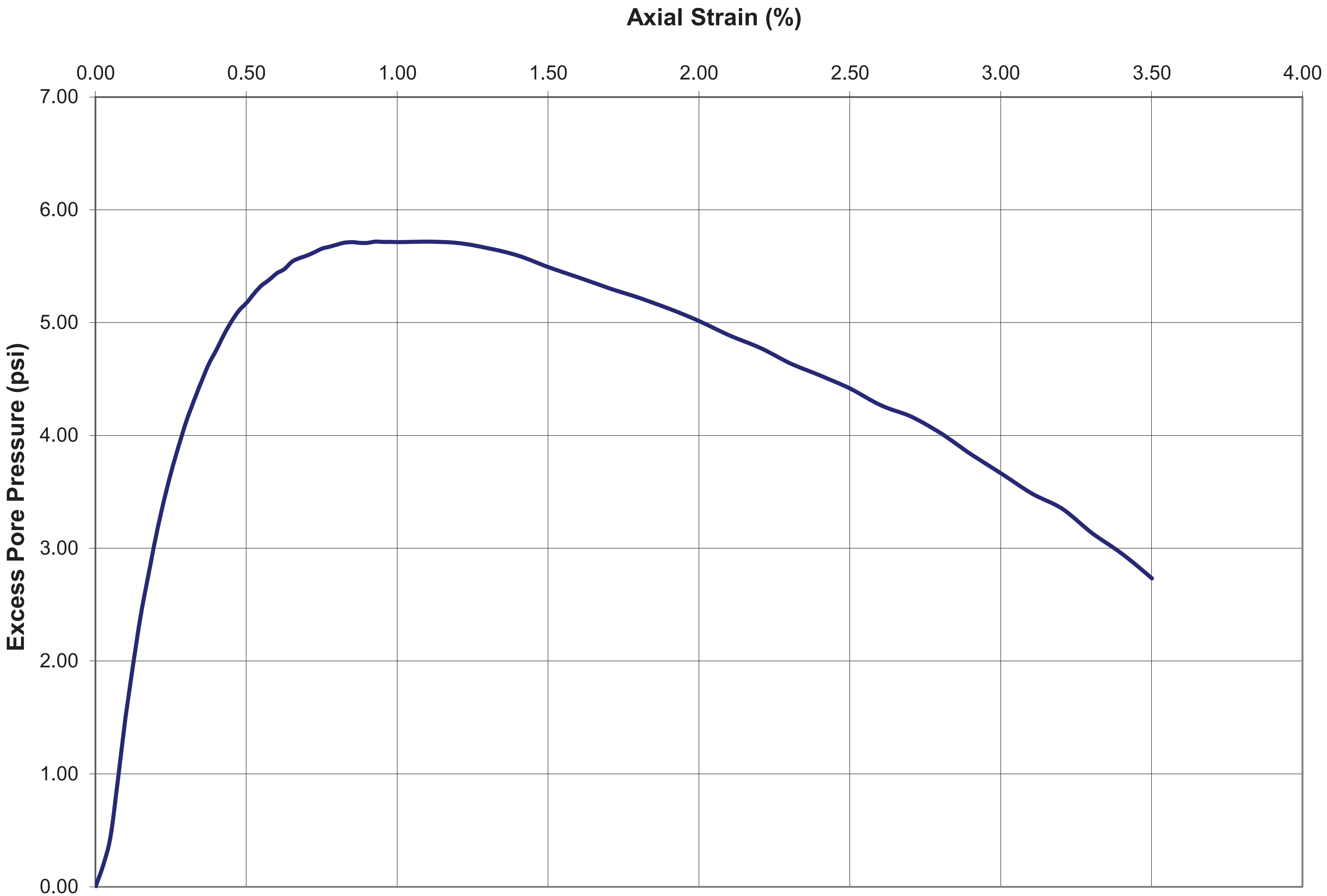


G154-21-B-2-43-45 STAGE 1 SHEAR

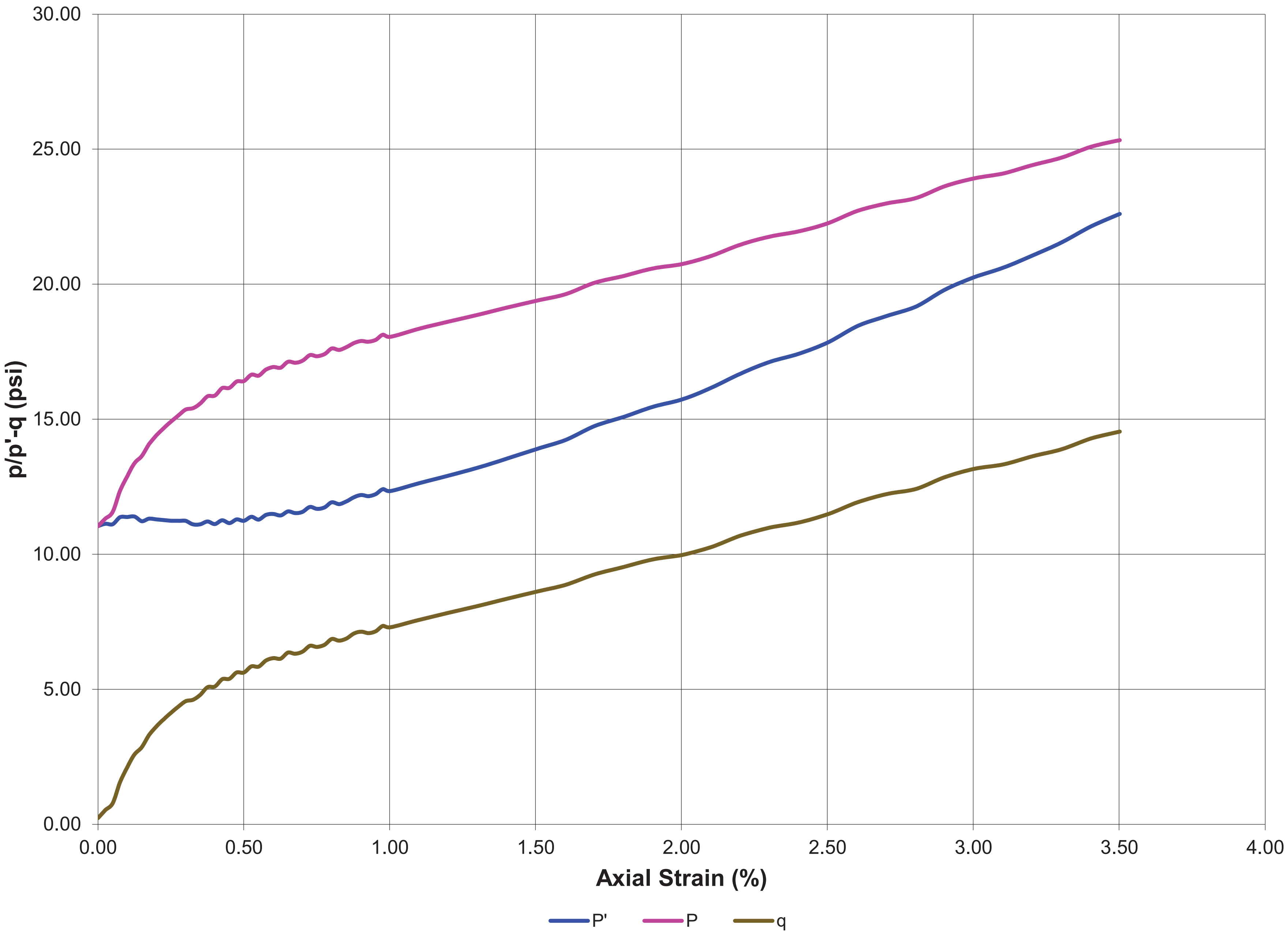


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 2.70%

G154-21-B-2-43-45 STAGE 1 SHEAR

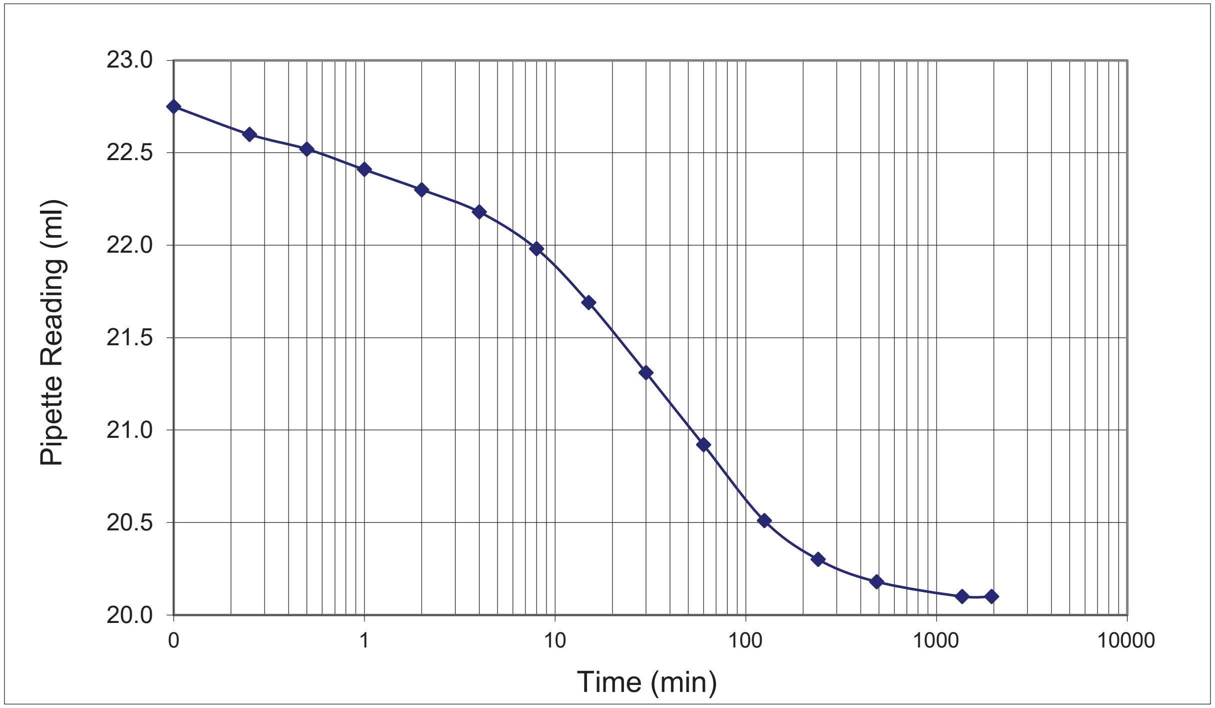


G154-21-B-2-43-45 STAGE 1 SHEAR



G154-21-B-2-43-45 STAGE 1 CONS

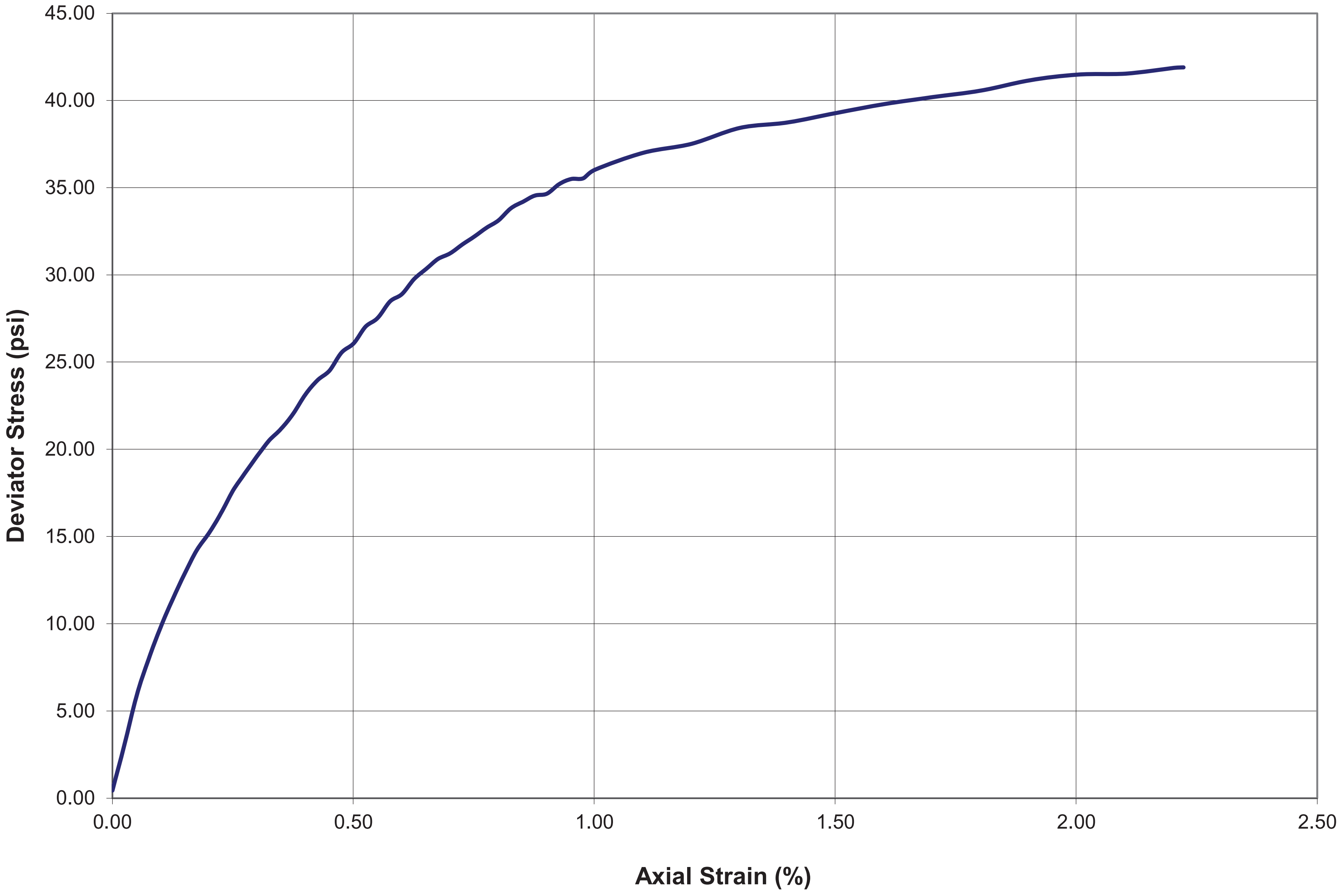
Time (min)	Pipette Reading (ml)
0.00	#REF!
0.10	22.75
0.25	22.60
0.50	22.52
1.00	22.41
2.00	22.30
4.00	22.18
8.00	21.98
15.00	21.69
30.00	21.31
60.00	20.92
125.00	20.51
240.00	20.30
486.00	20.18
1361.00	20.10
1948.00	20.10



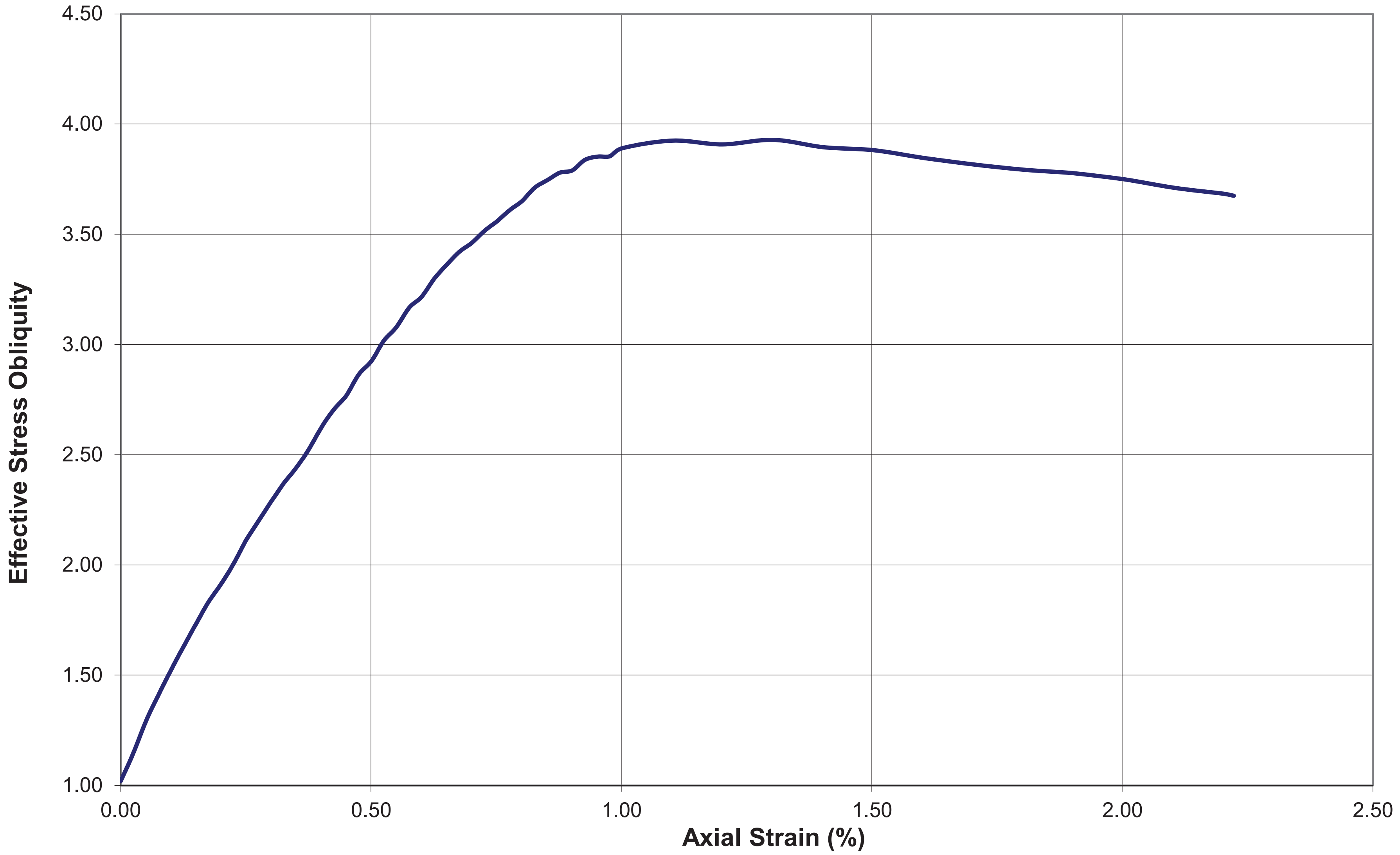
STAGE 2

B-2 43’-45’, at 22 psi confining pressures

G154-21-B-2-43-45 STAGE 2 SHEAR

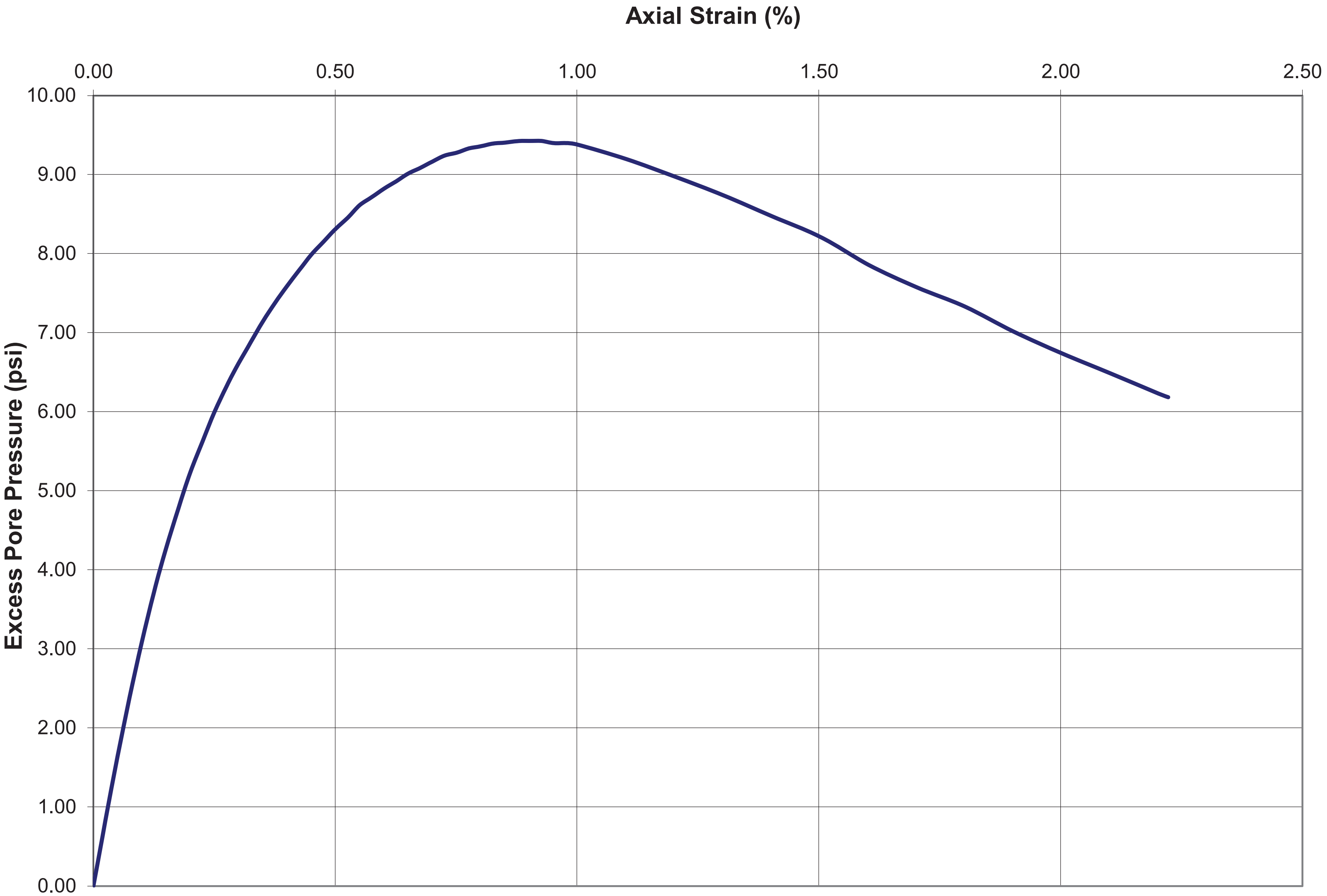


G154-21-B-2-43-45 STAGE 2 SHEAR

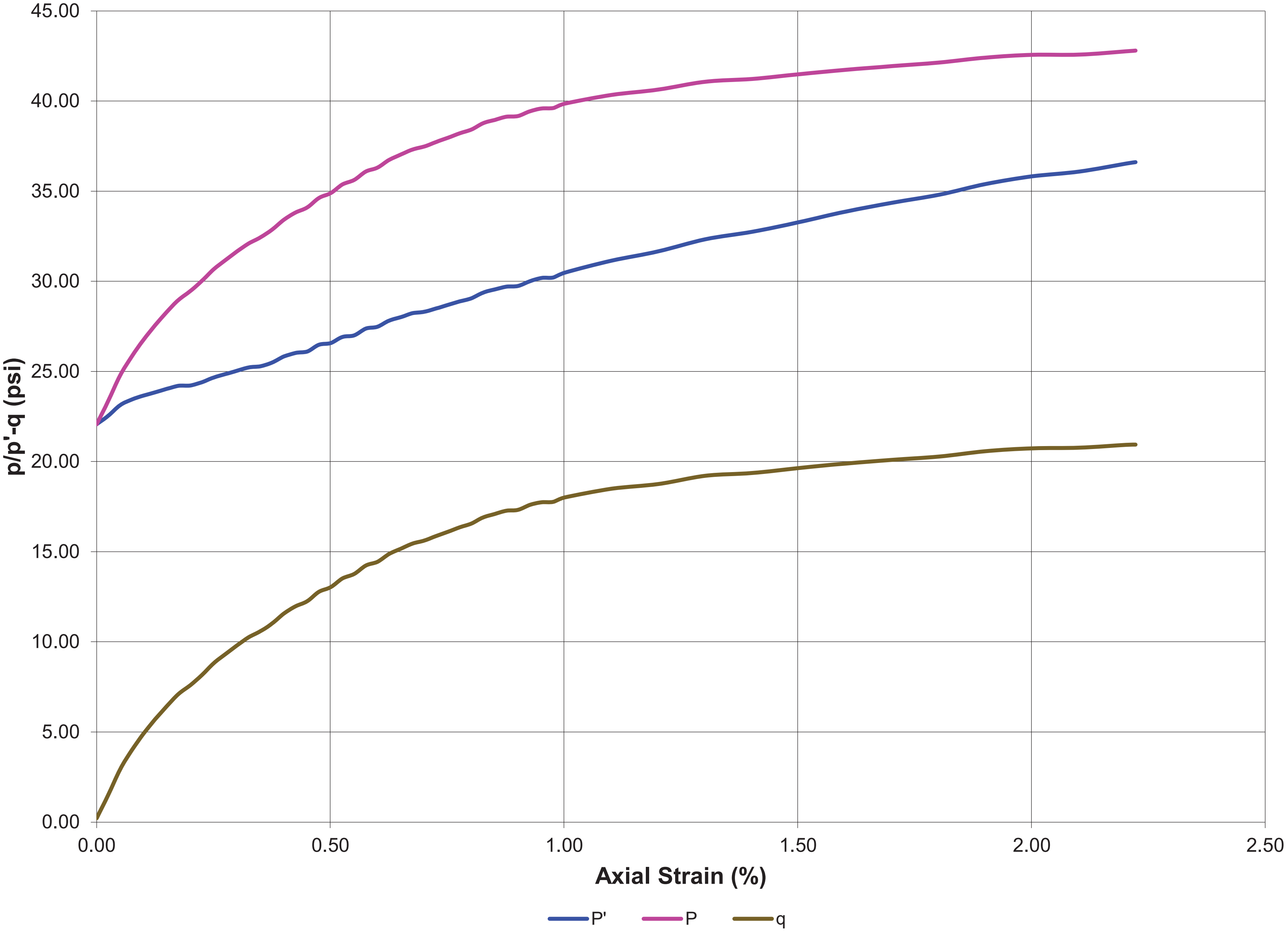


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 1.10%

G154-21-B-2-43-45 STAGE 2 SHEAR

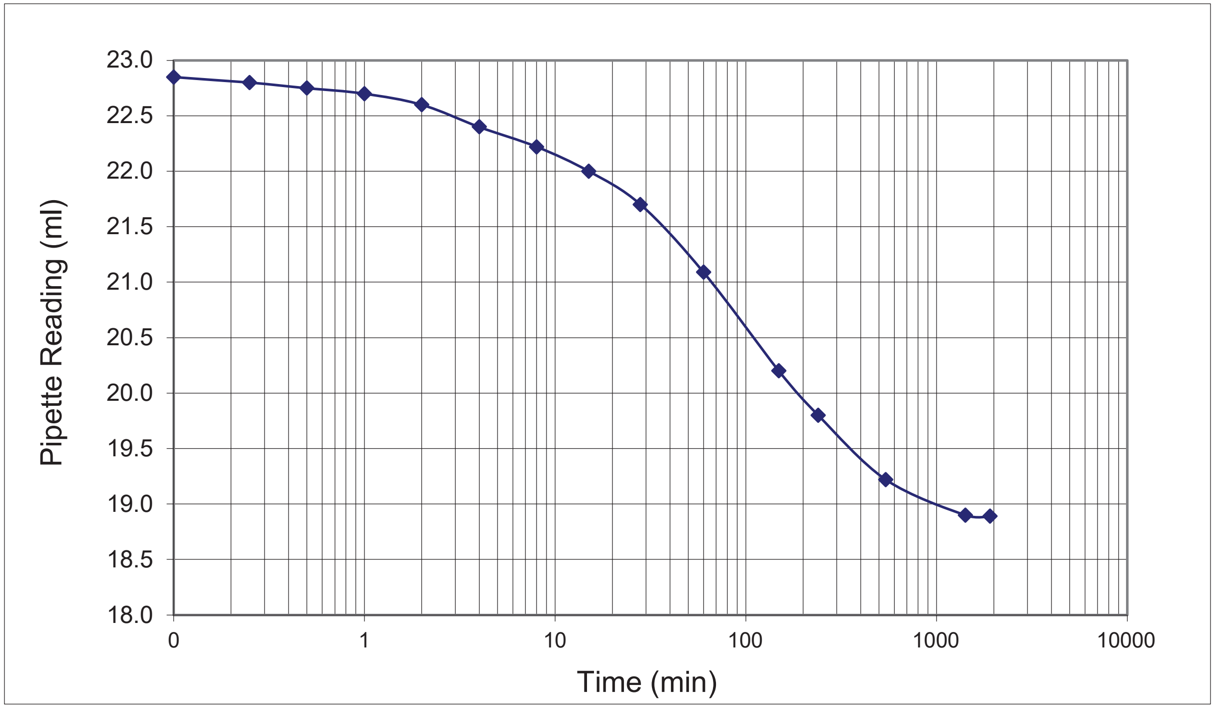


G154-21-B-2-43-45 STAGE 2 SHEAR



G154-21-B-2-43-45 STAGE 2 CONS

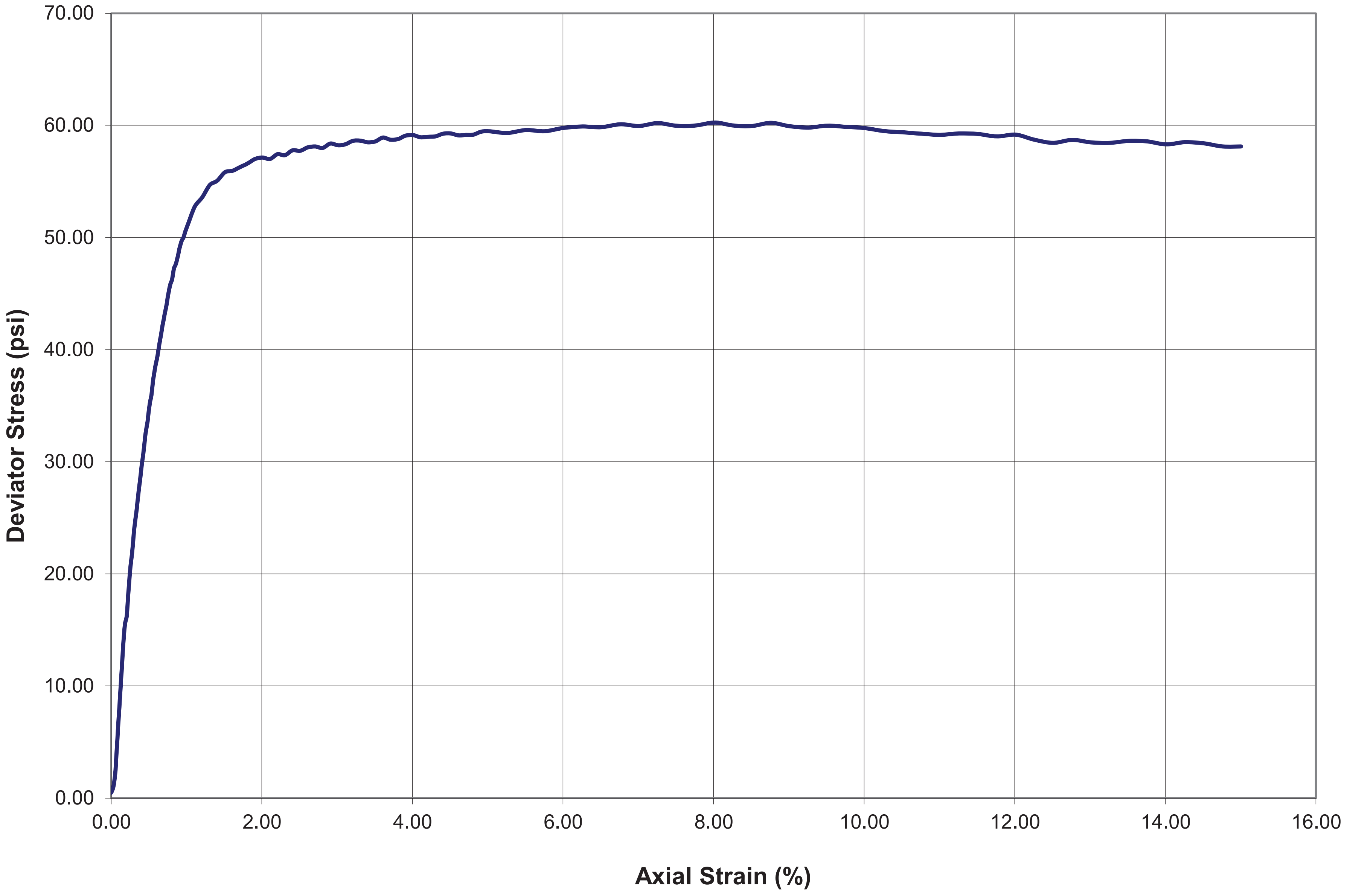
Time (min)	Pipette Reading (ml)
0.00	#REF!
0.10	22.85
0.25	22.80
0.50	22.75
1.00	22.70
2.00	22.60
4.00	22.40
8.00	22.22
15.00	22.00
28.00	21.70
60.00	21.09
149.00	20.20
240.00	19.80
541.00	19.22
1417.00	18.90
1909.00	18.89



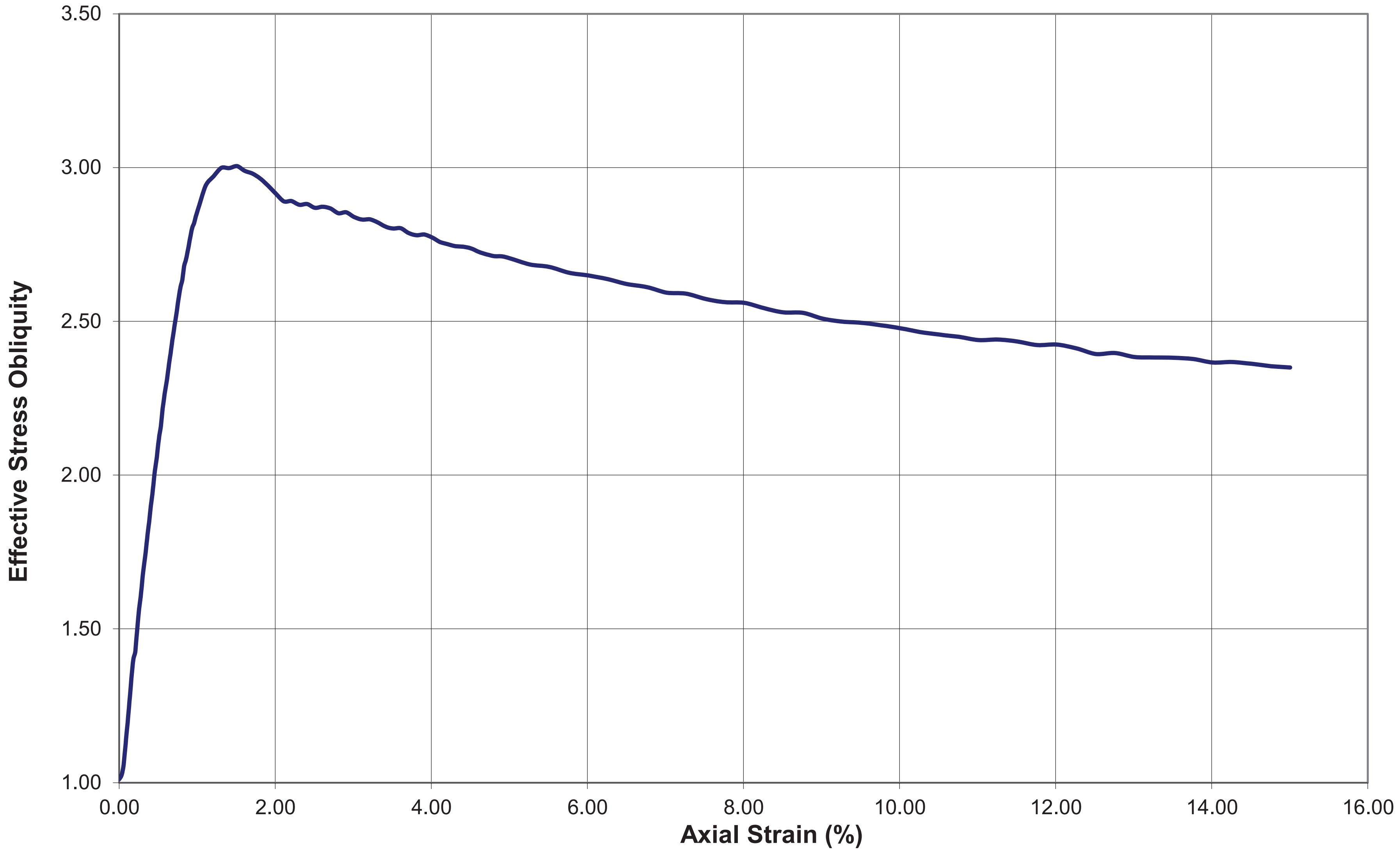
STAGE 3

B-2 43’-45’, at 44 psi confining pressures

G154-21-B-2-43-45 STAGE 3 SHEAR

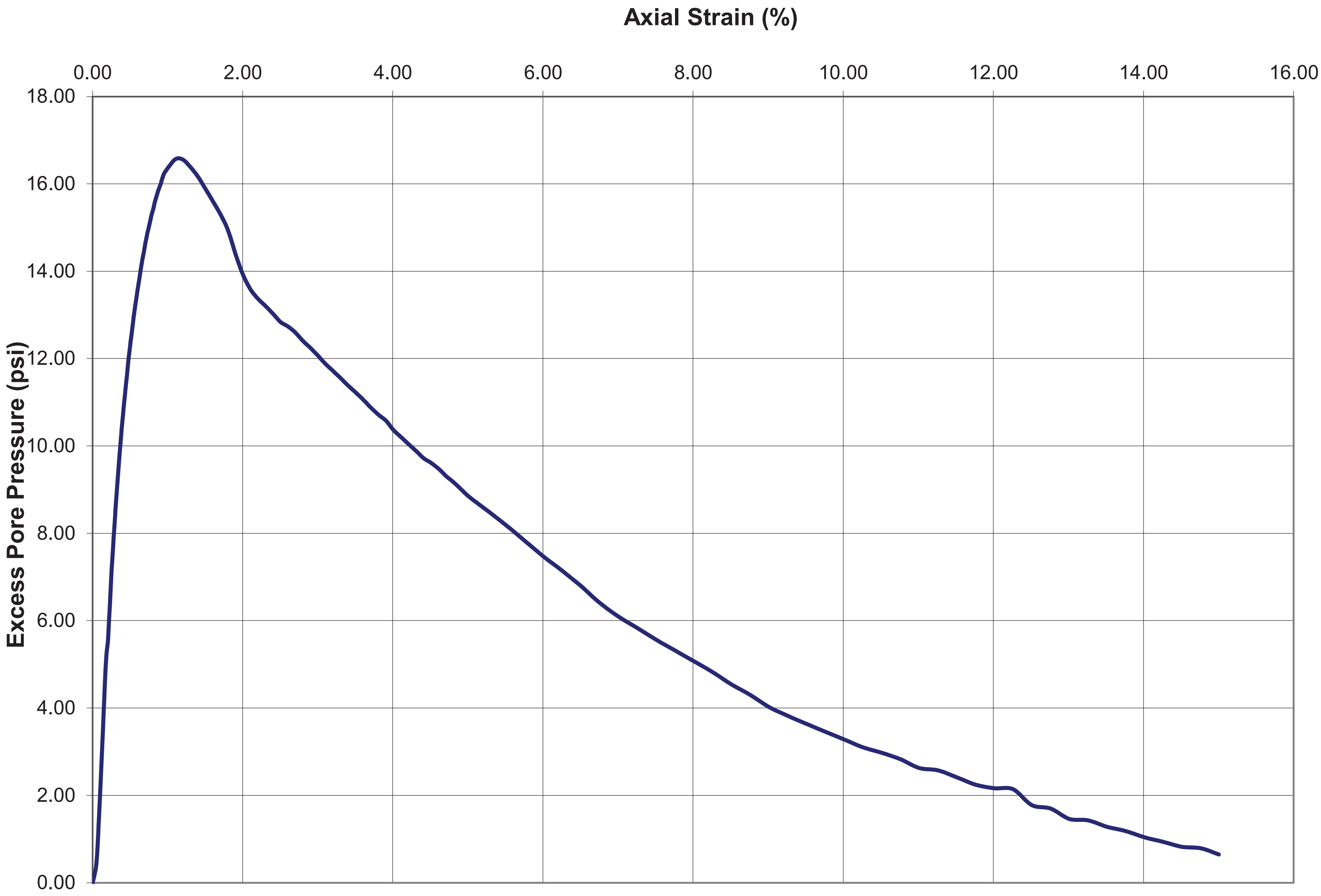


G154-21-B-2-43-45 STAGE 3 SHEAR

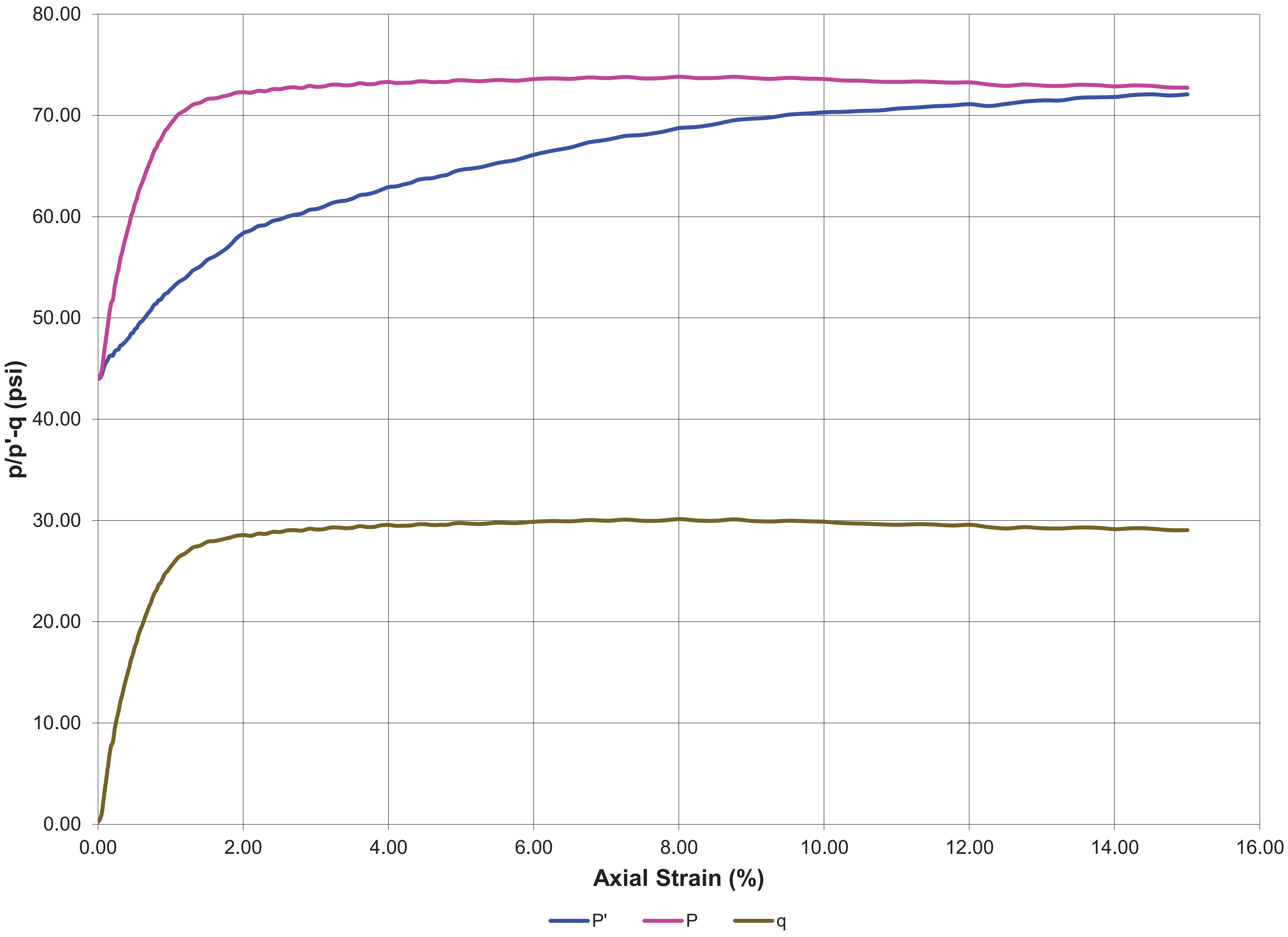


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 1.31%

G154-21-B-2-43-45 STAGE 3 SHEAR

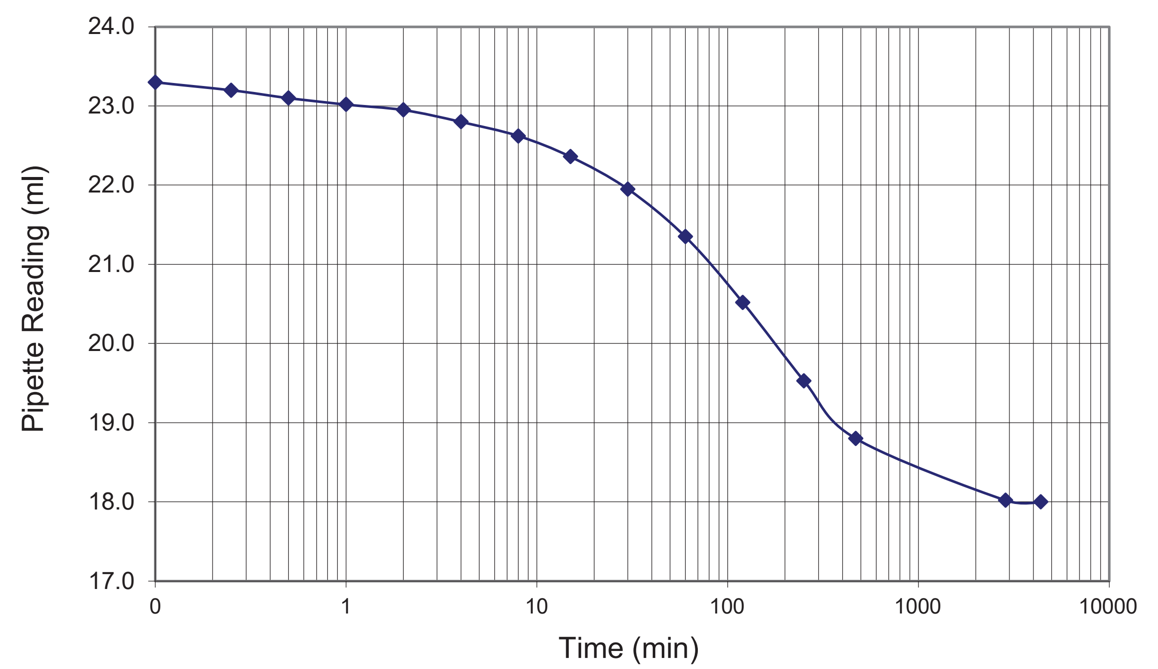


G154-21-B-2-43-45 STAGE 3 SHEAR



G154-21-B-2-43-45 STAGE 3 CONS

Time (min)	Pipette Reading (ml)
0.00	#REF!
0.10	23.30
0.25	23.20
0.50	23.10
1.00	23.02
2.00	22.95
4.00	22.80
8.00	22.62
15.00	22.36
30.00	21.95
60.00	21.35
120.00	20.52
251.00	19.53
470.00	18.80
2863.00	18.02
4381.00	18.00

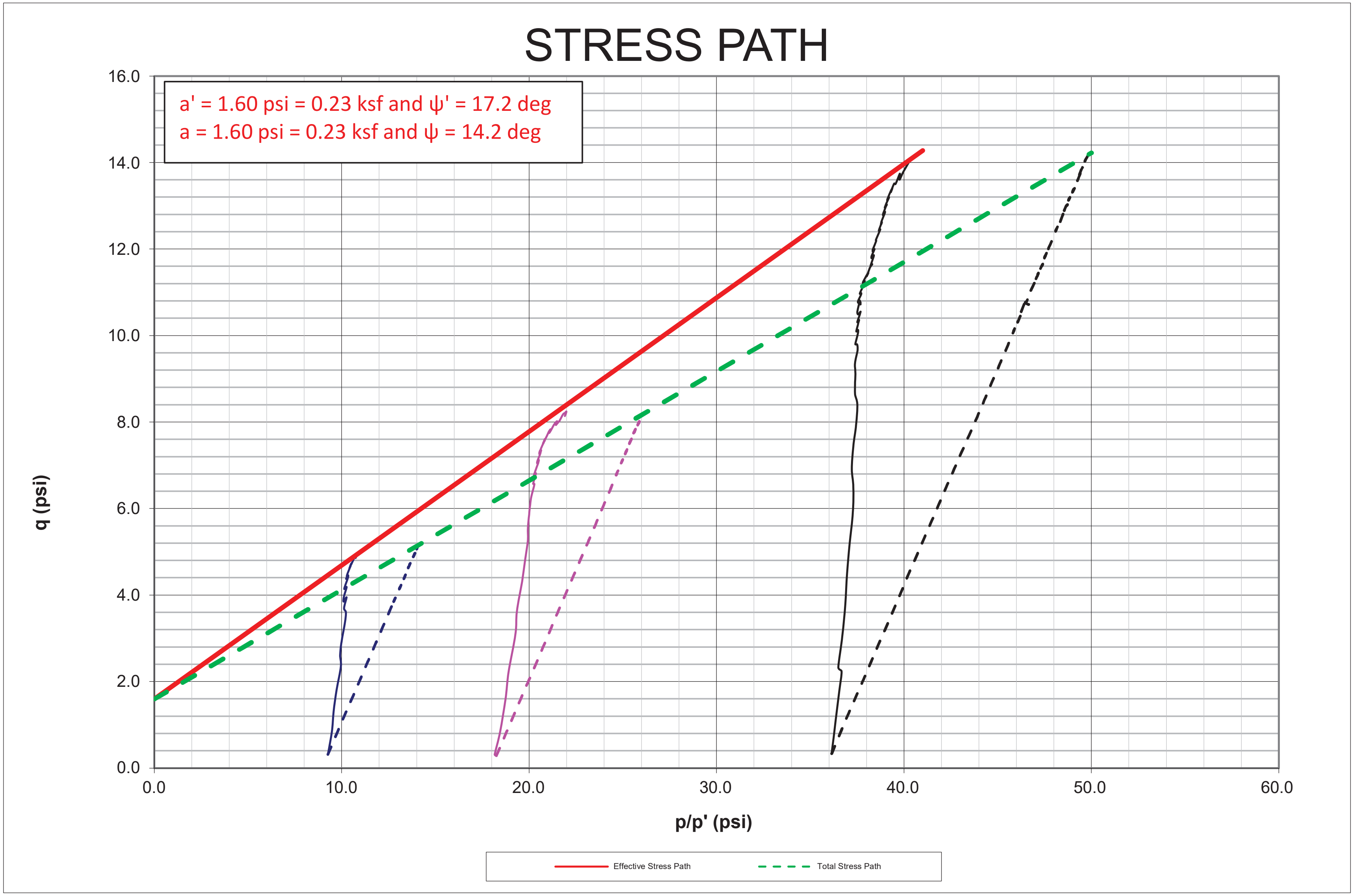


AVILES ENGINEERING CORP.

APPENDIX C-3

CU Test Results, Boring B-3, 33’-35’

G154-21 Boring B-3, 33'-35'



Notes:

1. Value of p and q at failure is determined considering maximum effective stress obliquity according to ASTM D6747 for each stage of CU test and plot them on the respective p-q curves.
2. Then a best-fit straight line will be drawn to fit the data and the slope (tanΨ) and intercept (d) will be determined according to US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4.
3. Then ϕ and c will be determined based on equation D-7 and D-8 (US Army Corps of Engineers Engineering Manual, Appendix D, Section D-4).

Proj. # G154-21 **Boring:** B-3 **Depth (ft):** 33-35

Initial Height Measurements (in.):
Height 1: 5.571
Height 2: 5.577
Height 3: 5.567
Average Height (in.): 5.571667

Initial Diameter Measurements (in.):
Diam 1: 2.752
Diam 2: 2.742
Diam 3: 2.773
Average Diameter (in): 2.755667

Initial Dial Gauge Reading (in): 0.147

End of Saturation Dial Gauge Reading (in.): 0.114

First Consolidation: *(if there is no first stage consolidation, enter '0' for initial and final pipette readings, copy DGs to DGc)*
Initial Pipette Reading (mL): 23.1
Final Pipette Reading (mL): 20.55
Final Dial Gauge Reading (in.): 0.122

Beginning of First Shear:
Height: 5.597
Diameter: 2.767

End of First Shear:
Dial Gauge Reading at end of shearing (in.): 0.205
Dial Gauge Reading after CV rebound (in): 0.184

Second Stage Consolidation:
Initial Pipette Reading (mL): 23
Final Pipette Reading (mL): 16.94
Final Dial Gauge Reading (in.): 0.202

Beginning of Second Shear:
Height: 5.517
Diameter: 2.803

End of Second Shear:
Dial Gauge Reading at end of shearing (in.): 0.321
Dial Gauge Reading after CV rebound (in): 0.298

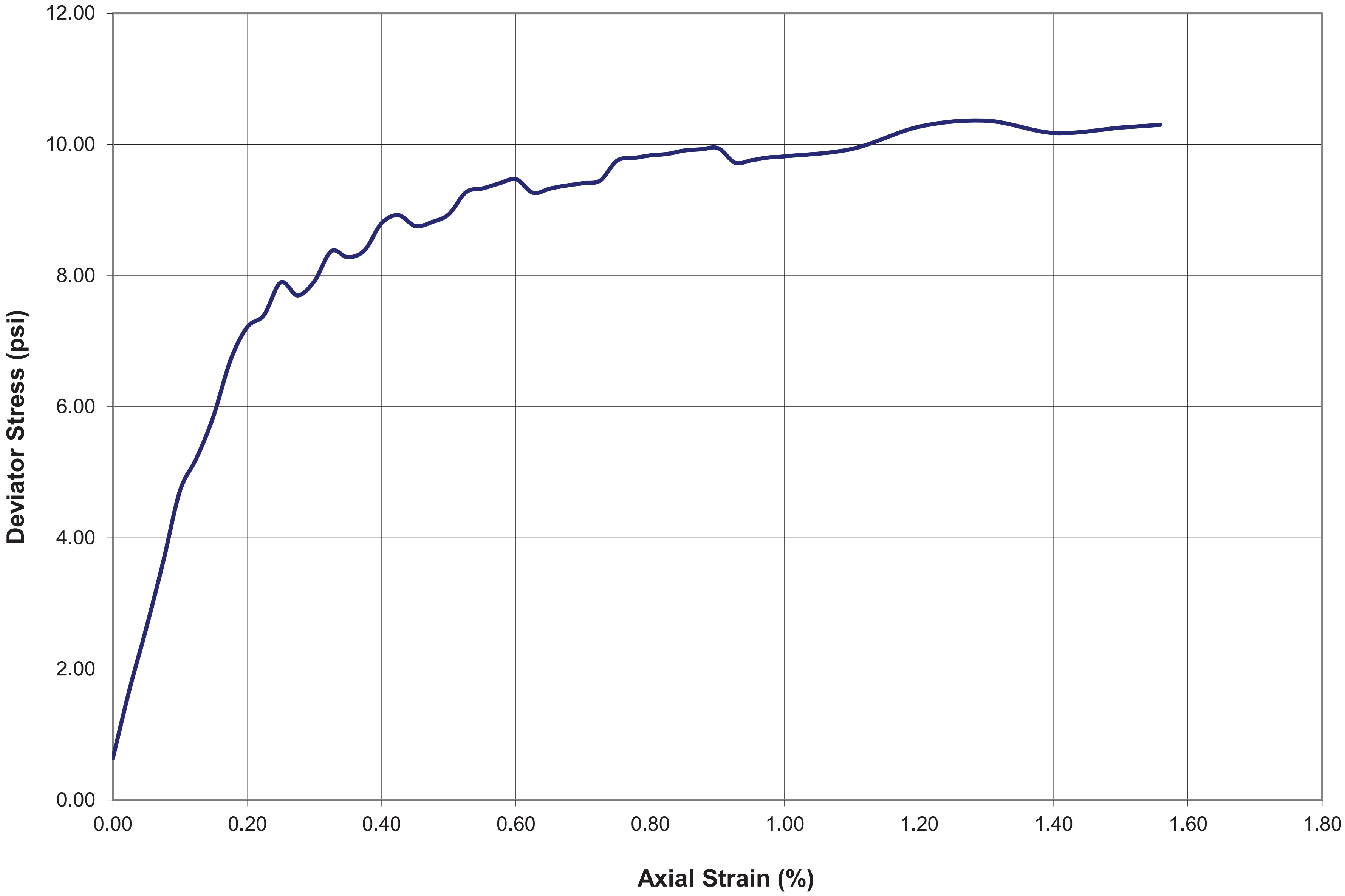
Third Stage Consolidation:
Initial Pipette Reading (mL): 23.05
Final Pipette Reading (mL): 16.45
Final Dial Gauge Reading (in.): 0.316

Beginning of Third Shear:
Height: 5.403
Diameter: 2.849

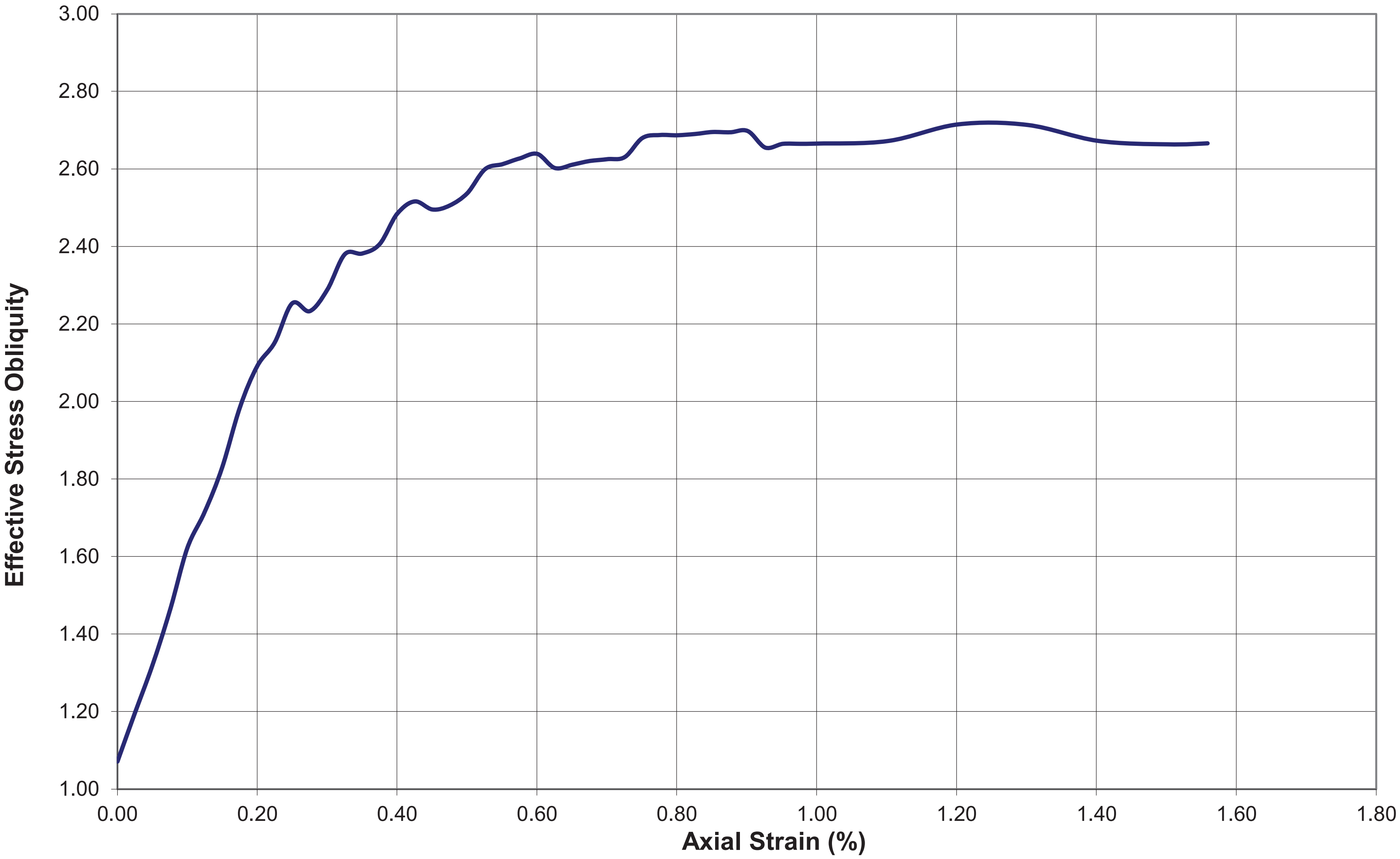
STAGE 1

B-3 33’-35’, at 9 psi confining pressures

G154-21-B-3-33-35 STAGE 1 SHEAR

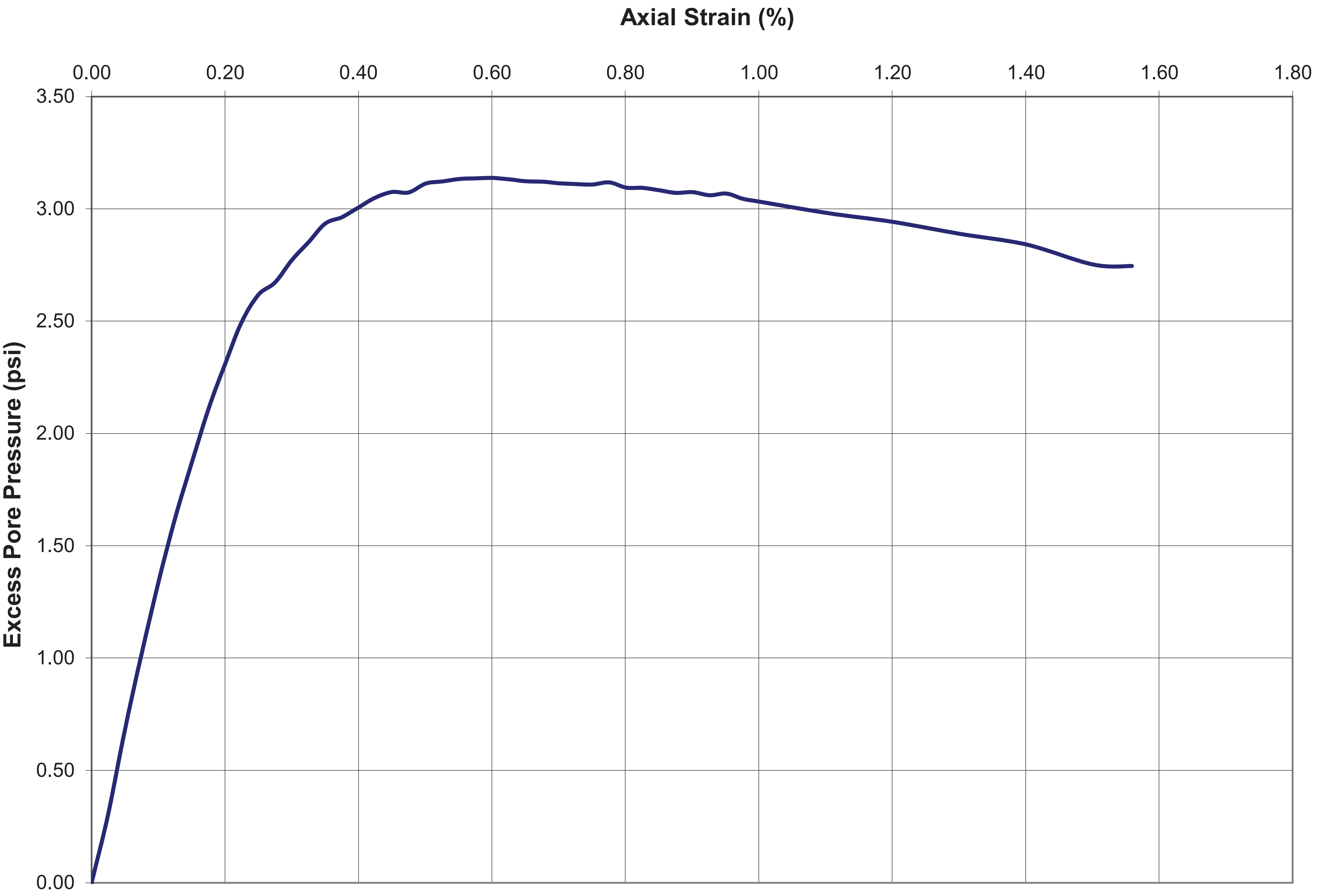


G154-21-B-3-33-35 STAGE 1 SHEAR

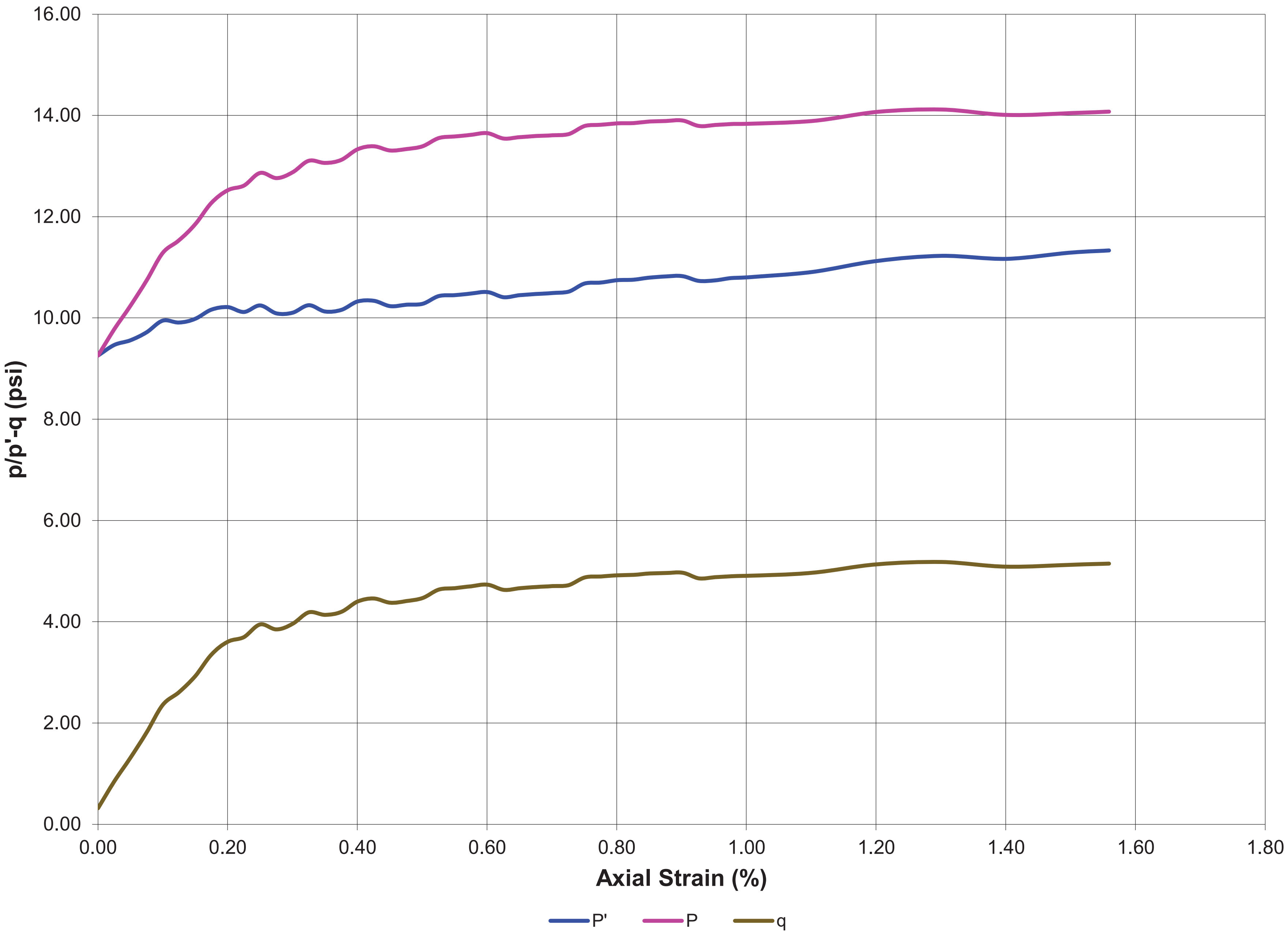


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 1.20%

G154-21-B-3-33-35 STAGE 1 SHEAR

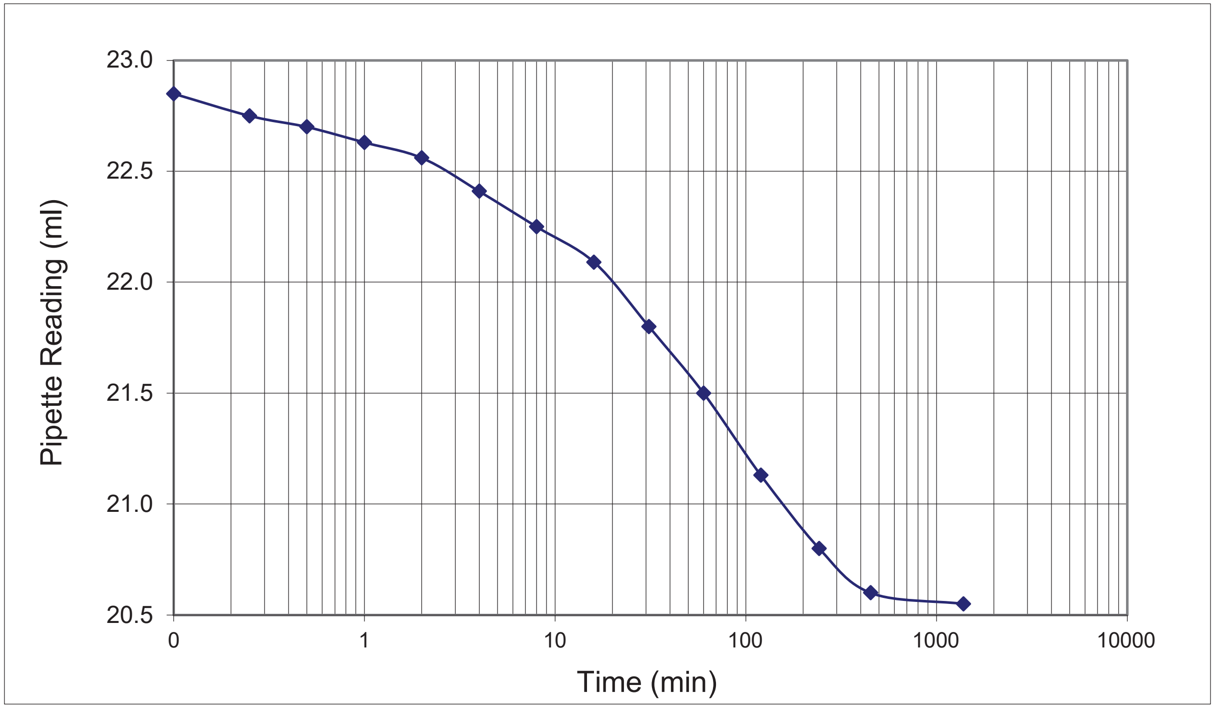


G154-21-B-3-33-35 STAGE 1 SHEAR



G154-21-B-3-33-35 STAGE 1 CONS

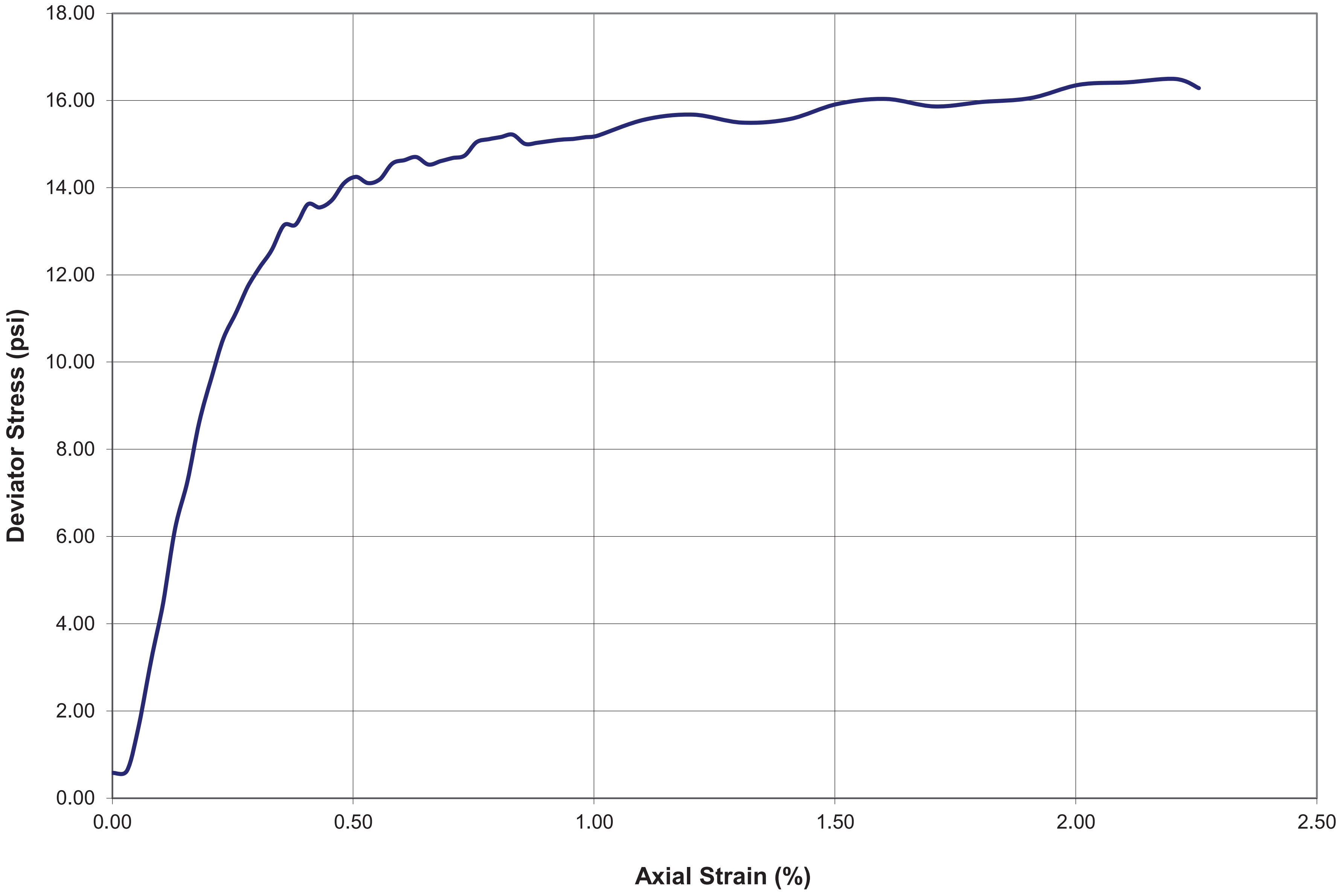
Time (min)	Pipette Reading (ml)
0.00	#REF!
0.10	22.85
0.25	22.75
0.50	22.70
1.00	22.63
2.00	22.56
4.00	22.41
8.00	22.25
16.00	22.09
31.00	21.80
60.00	21.50
120.00	21.13
242.00	20.80
452.00	20.60
1385.00	20.55
0.00	0.00



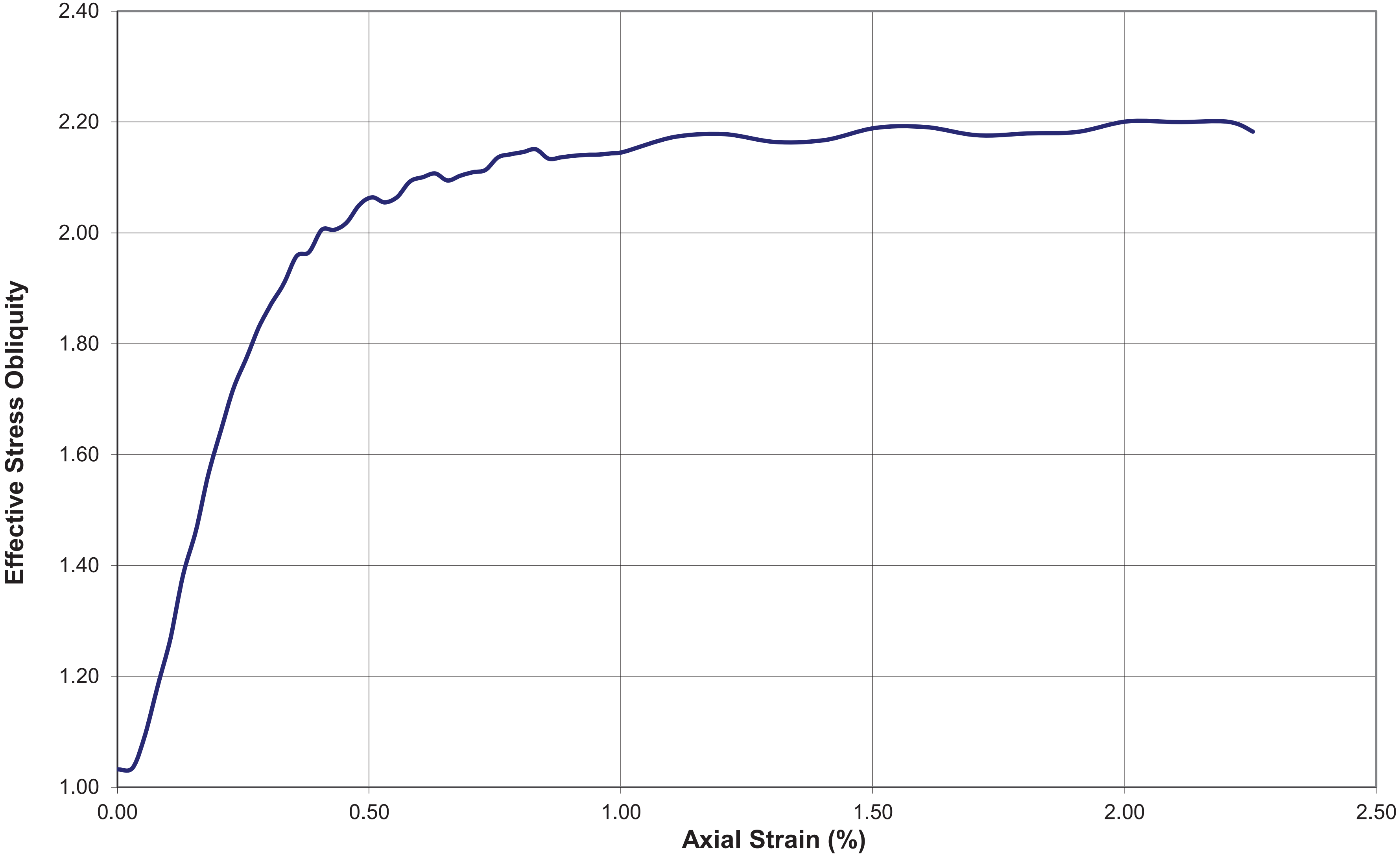
STAGE 2

B-3 33’-35’, at 18 psi confining pressures

G154-21-B-3-33-35 STAGE 2 SHEAR

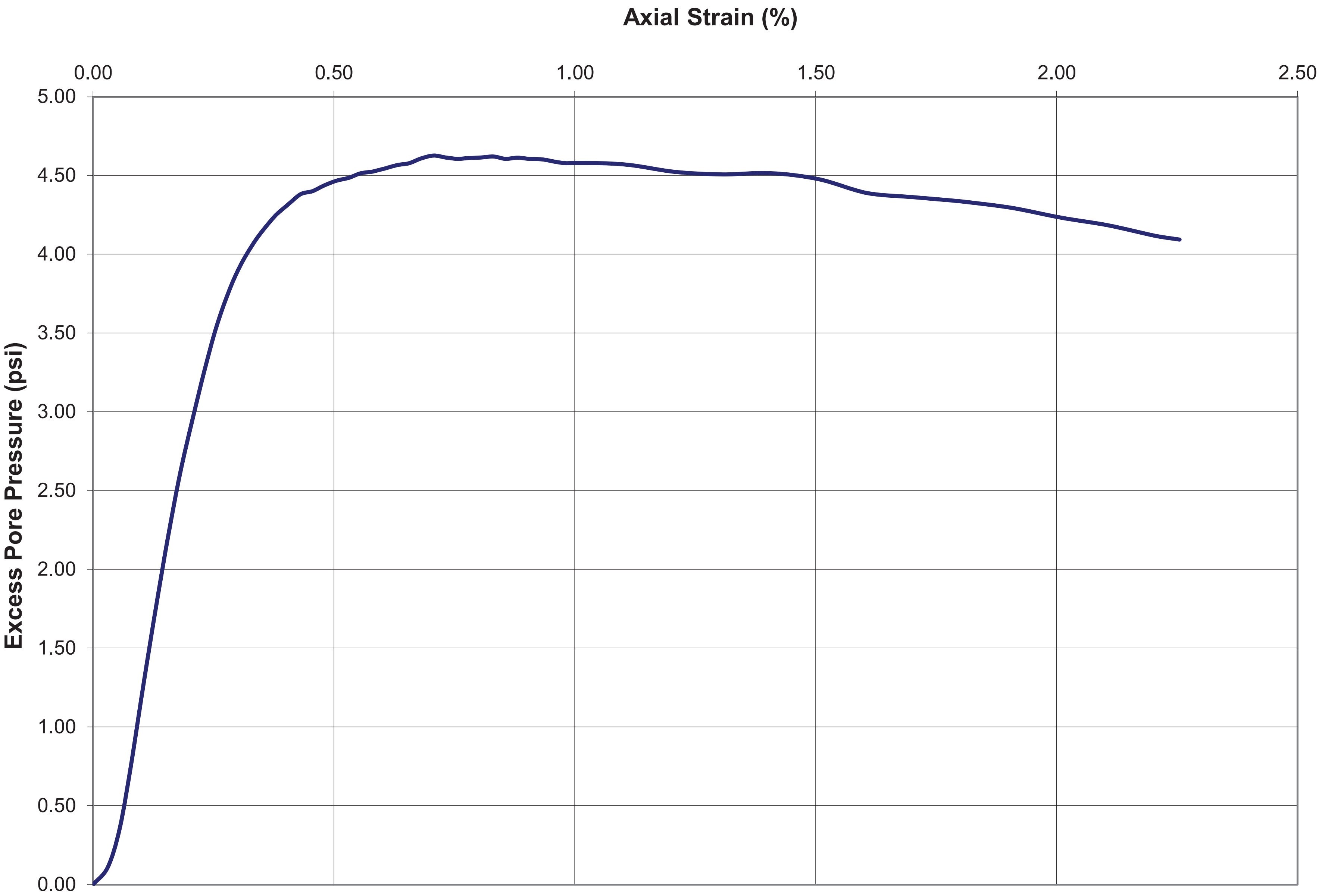


G154-21-B-3-33-35 STAGE 2 SHEAR

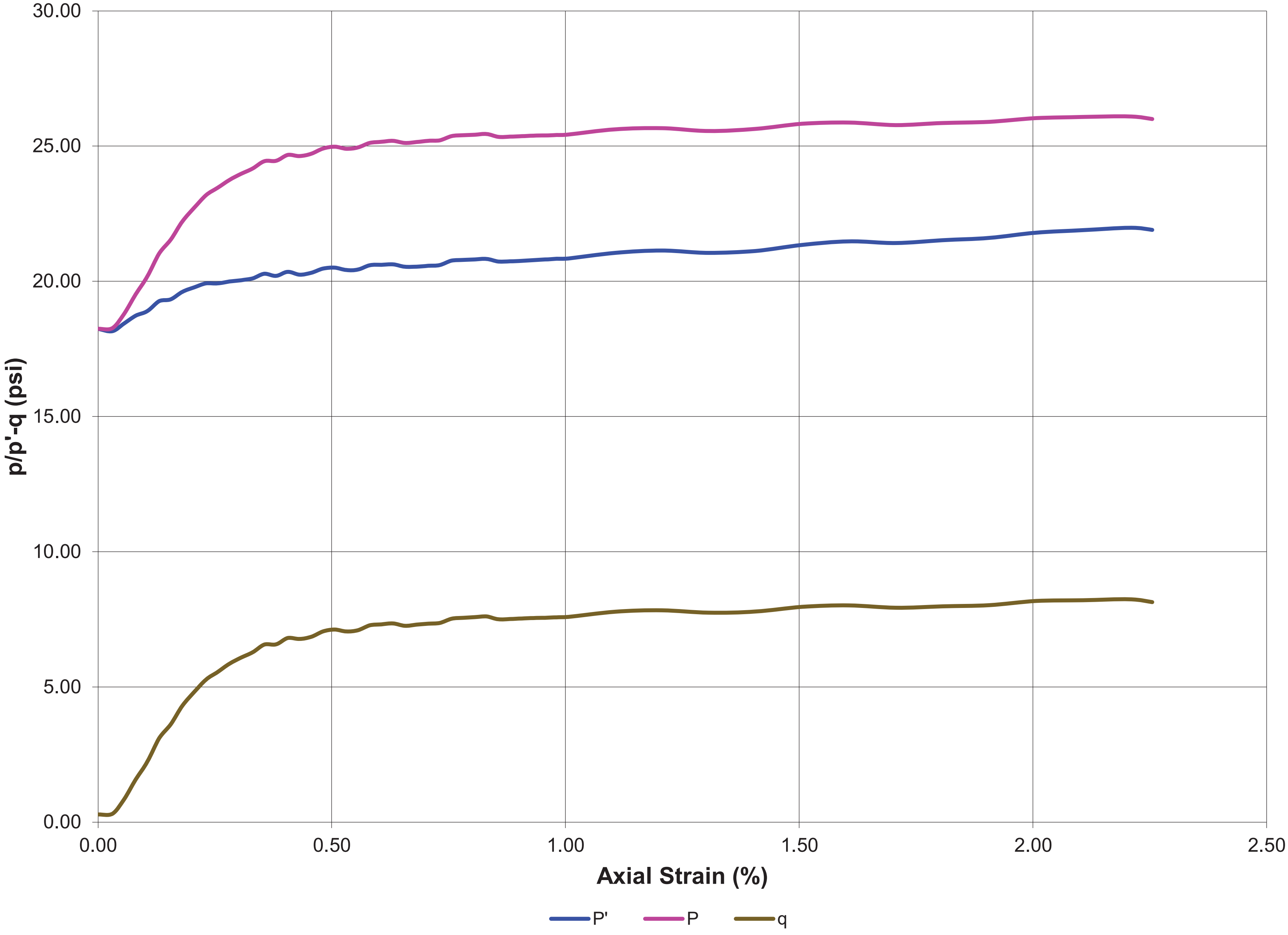


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 2.01%

G154-21-B-3-33-35 STAGE 2 SHEAR

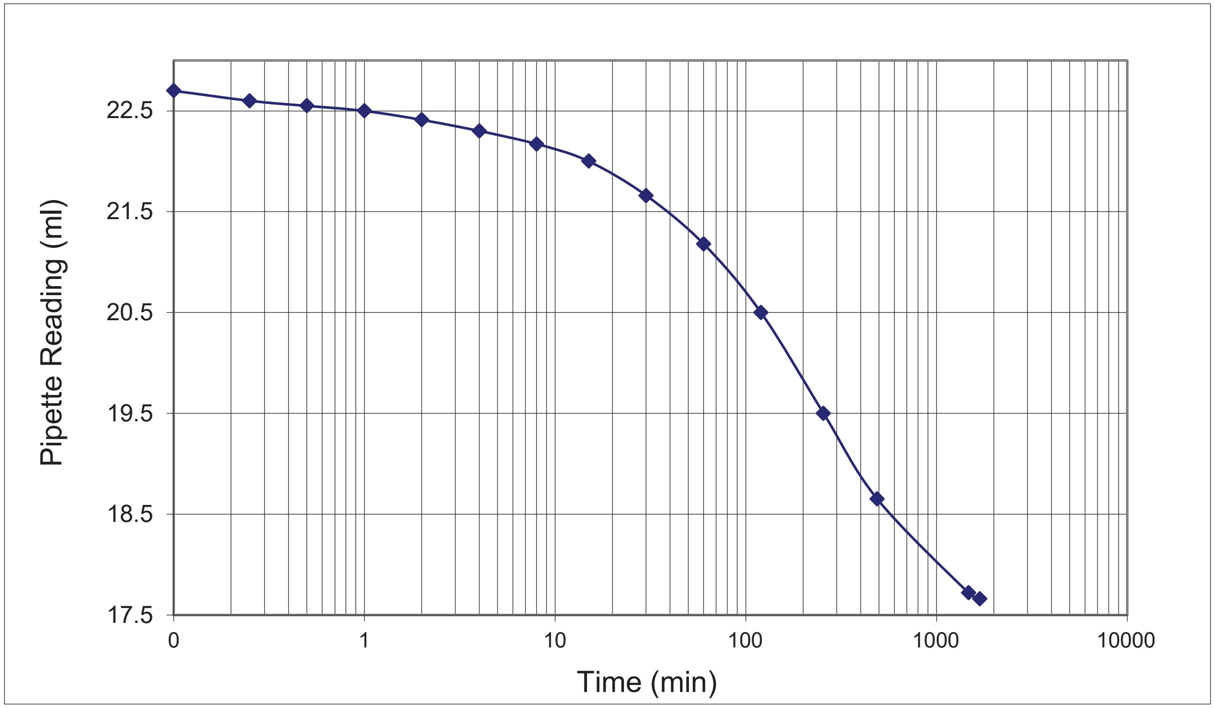


G154-21-B-3-33-35 STAGE 2 SHEAR



G154-21-B-3-33-35 STAGE 2 CONS

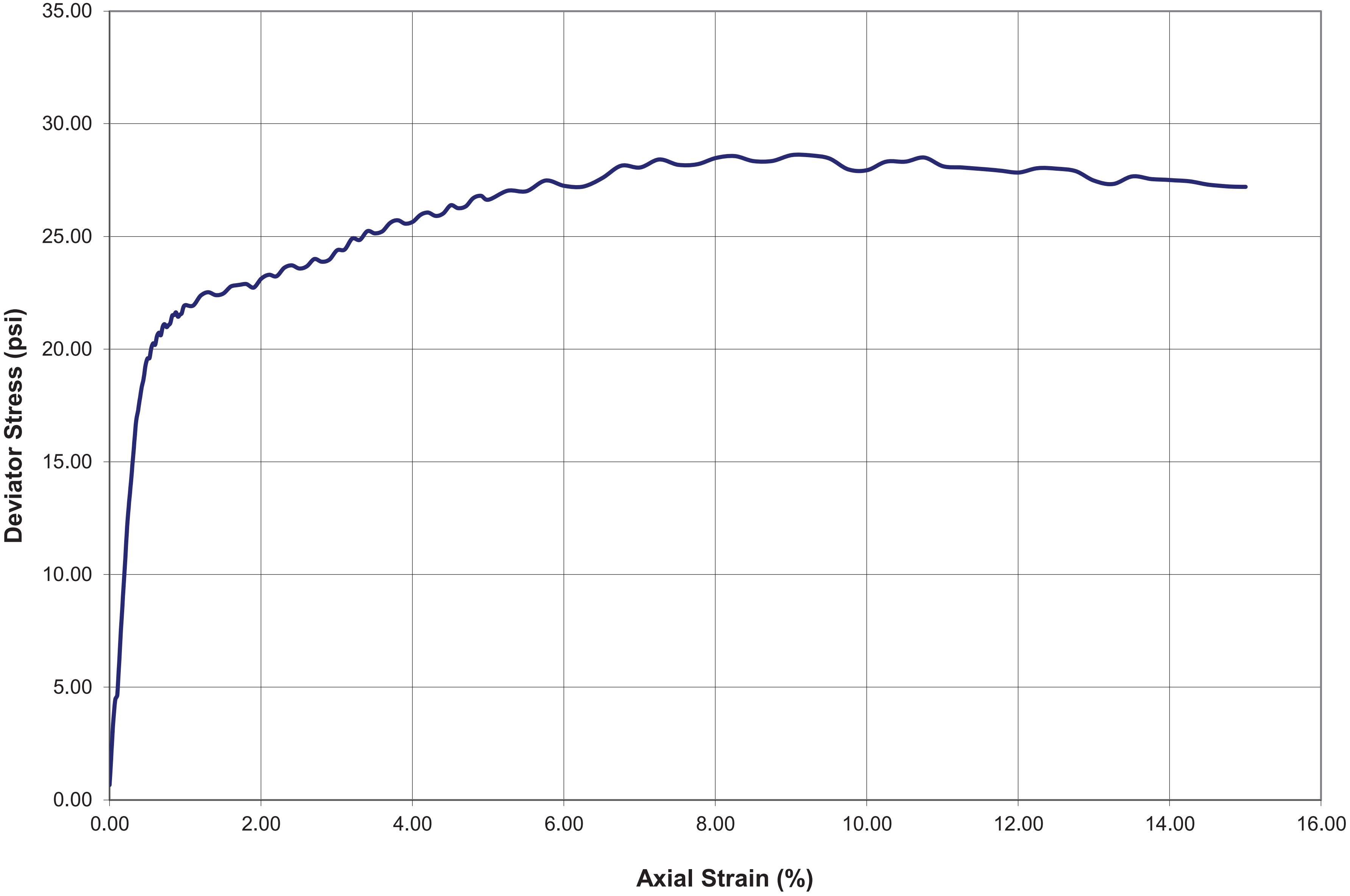
Time (min)	**Pipette Reading (ml)**
0.00	#REF!
0.10	22.70
0.25	22.60
0.50	22.55
1.00	22.50
2.00	22.41
4.00	22.30
8.00	22.17
15.00	22.00
30.00	21.66
60.00	21.18
120.00	20.50
255.00	19.50
488.00	18.65
1472.00	17.72
1686.00	17.66



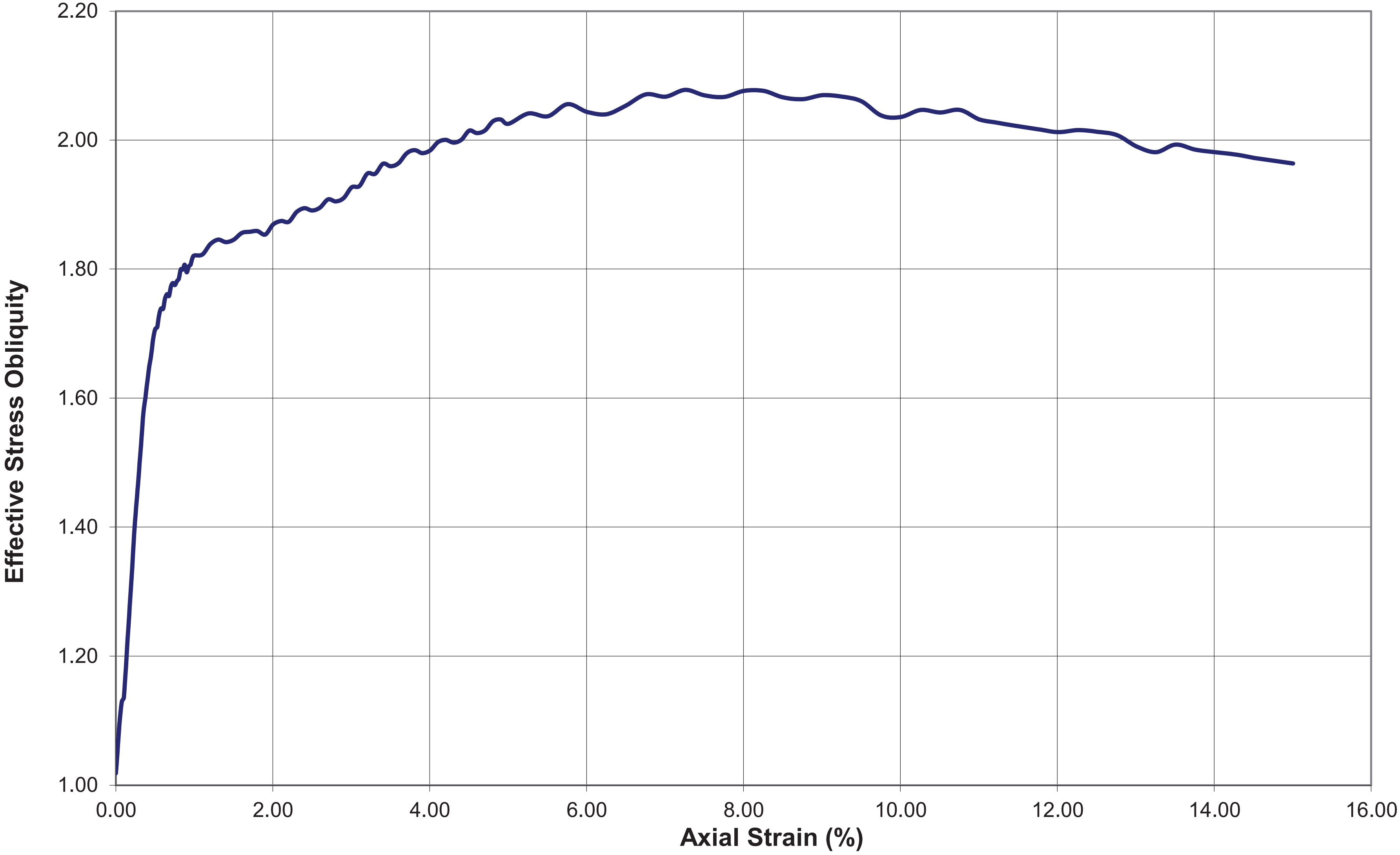
STAGE 3

B-3 33’-35’, at 36 psi confining pressures

G154-21-B-3-33-35 STAGE 3 SHEAR

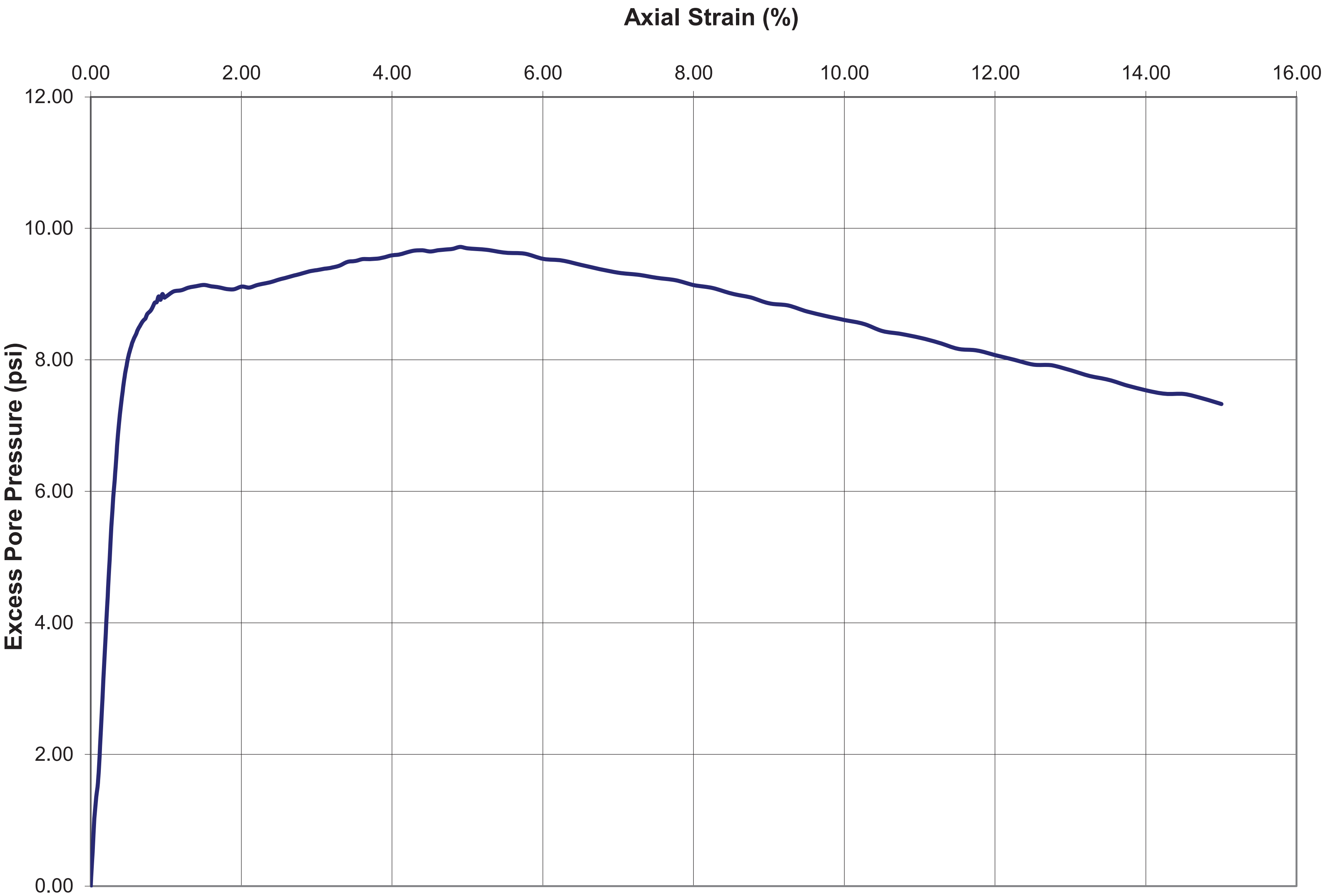


G154-21-B-3-33-35 STAGE 3 SHEAR

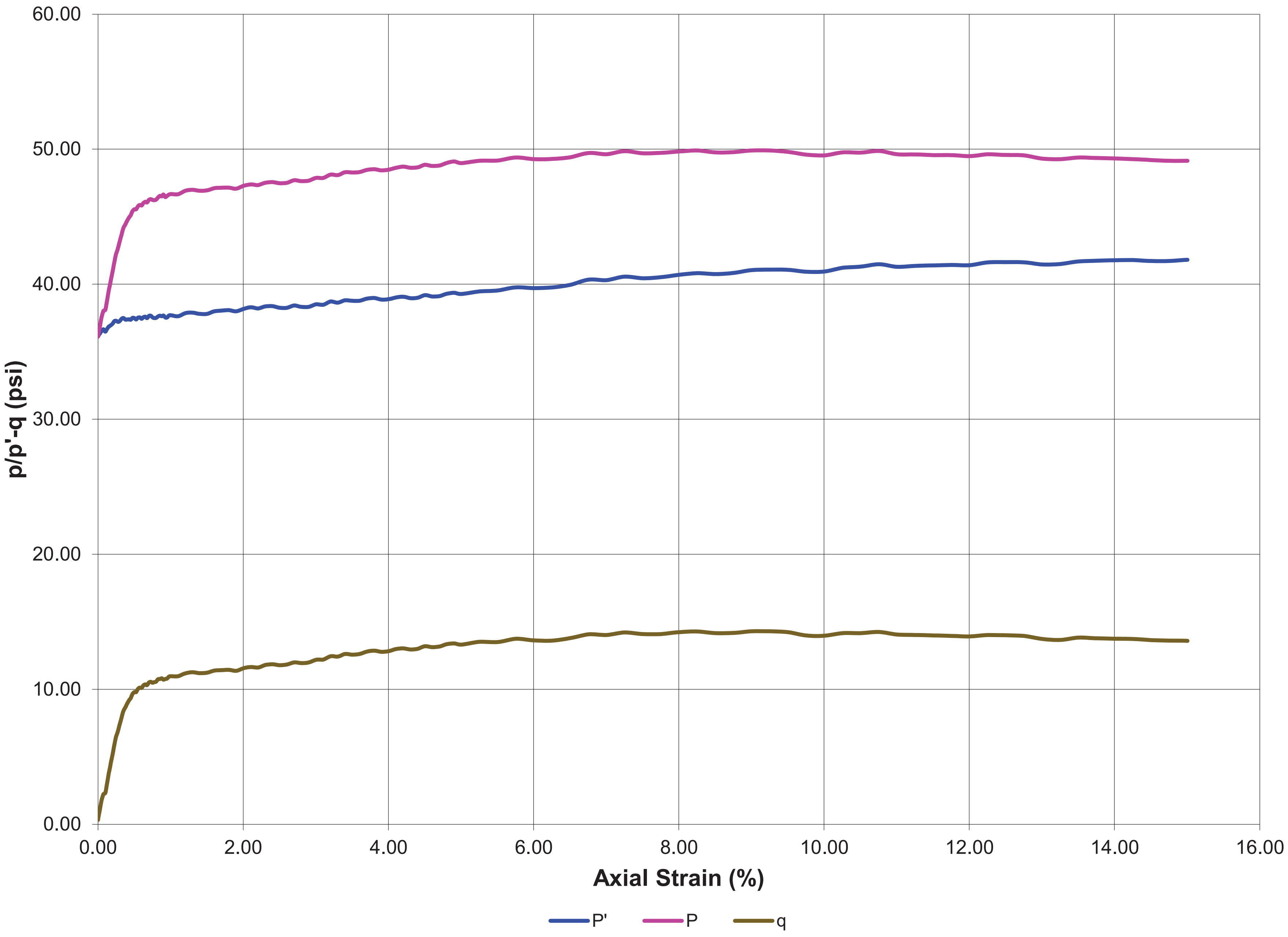


Failure is determined considering maximum effective stress obliquity according to ASTM D6747
Axial Strain at Failure = 7.26%

G154-21-B-3-33-35 STAGE 3 SHEAR

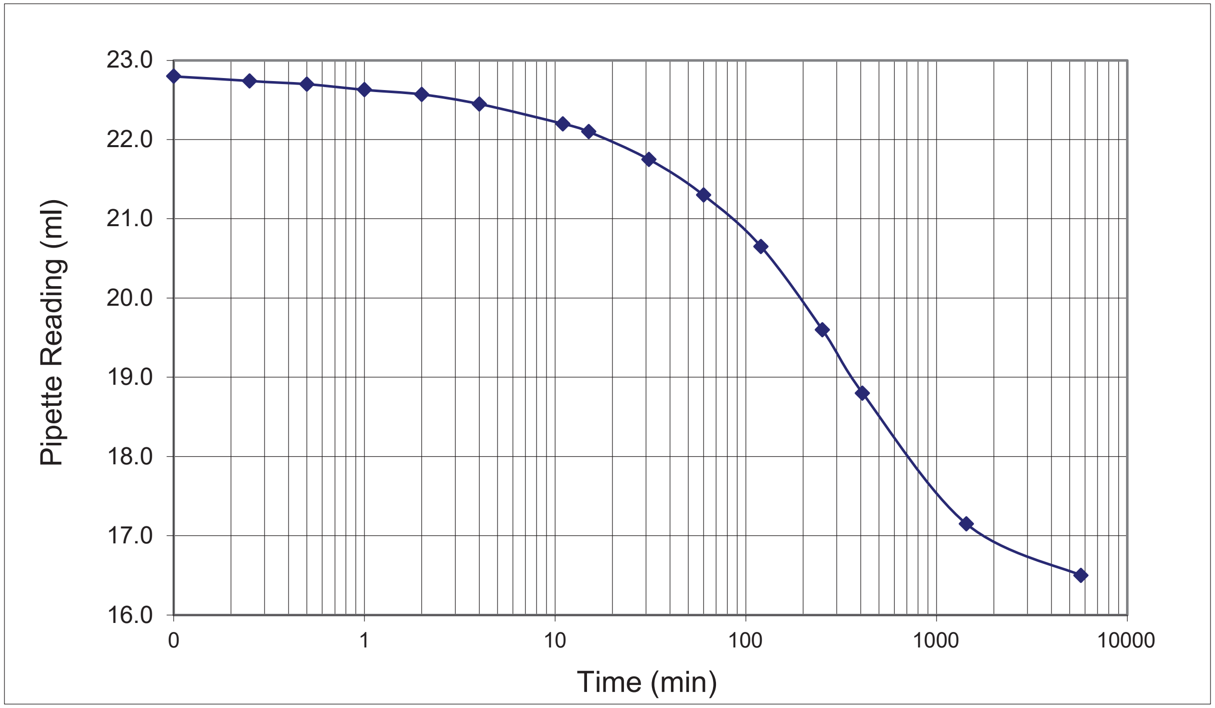


G154-21-B-3-33-35 STAGE 3 SHEAR



G154-21-B-3-33-35 STAGE 3 CONS

Time (min)	**Pipette Reading (ml)**
0.00	#REF!
0.10	22.80
0.25	22.74
0.50	22.70
1.00	22.63
2.00	22.57
4.00	22.45
11.00	22.20
15.00	22.10
31.00	21.75
60.00	21.30
120.00	20.65
252.00	19.60
409.00	18.80
1431.00	17.15
5712.00	16.50

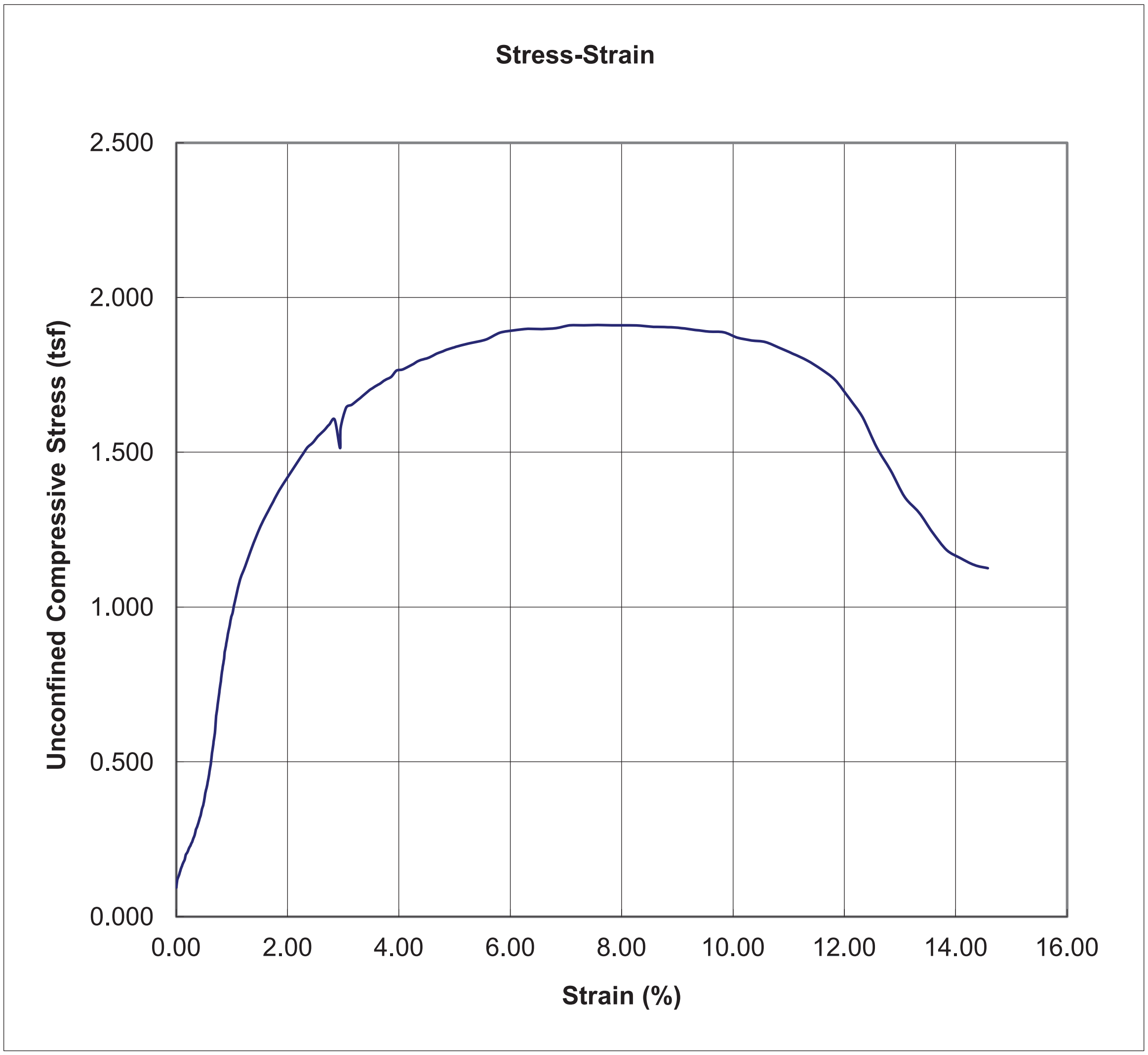


APPENDIX C-4

UC and UU Test Stress-Strain Curves

Project: G154-21
Boring: B-1
Depth (ft): 8
Date: 3/8/2024

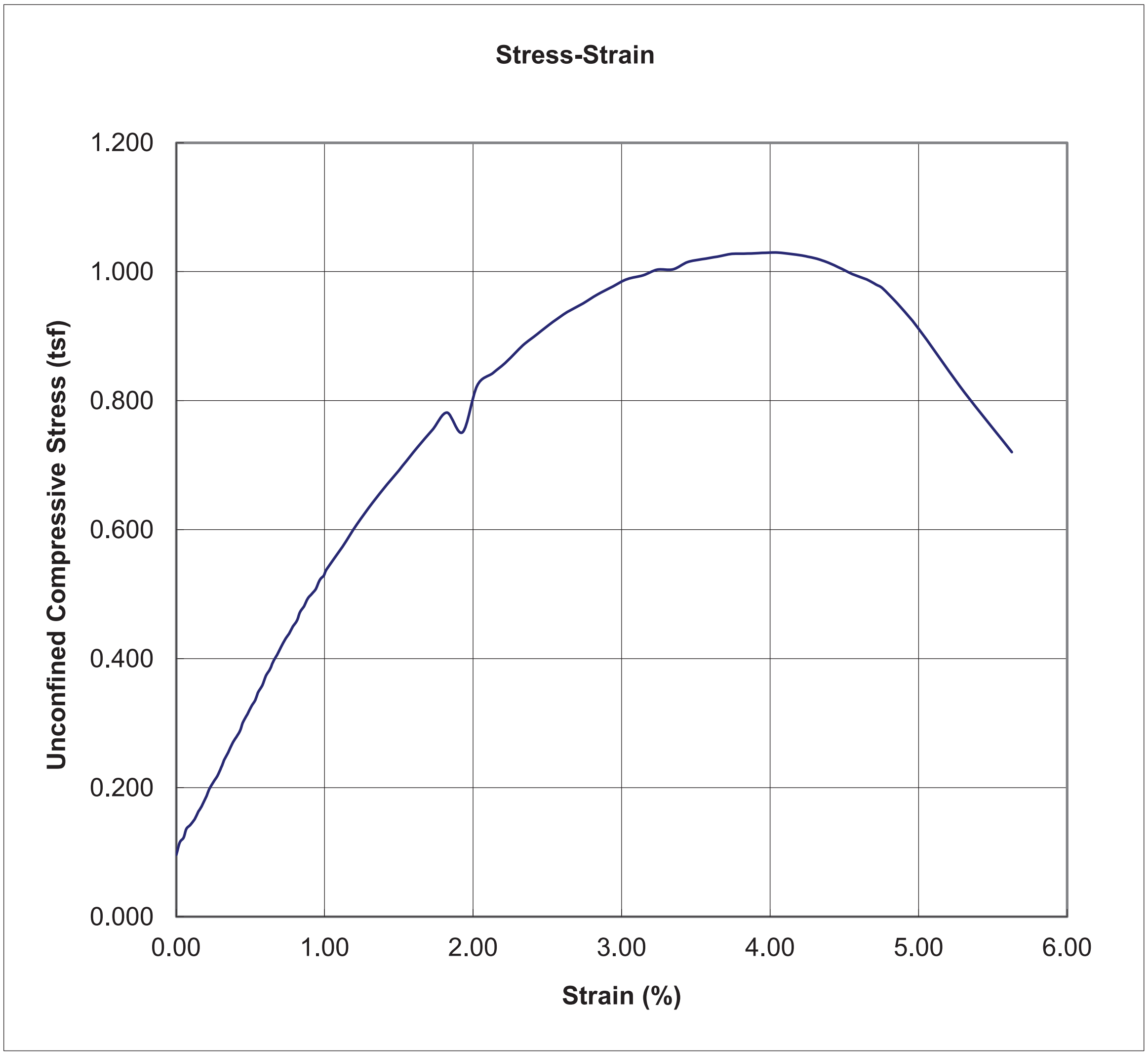
Peak: % Strain: 7.57 Stress (TSF): 1.91



Last Revised: 09/20/23 Rev. 0

Project: G154-21
Boring: B-1
Depth (ft): 14
Date: 3/9/2024

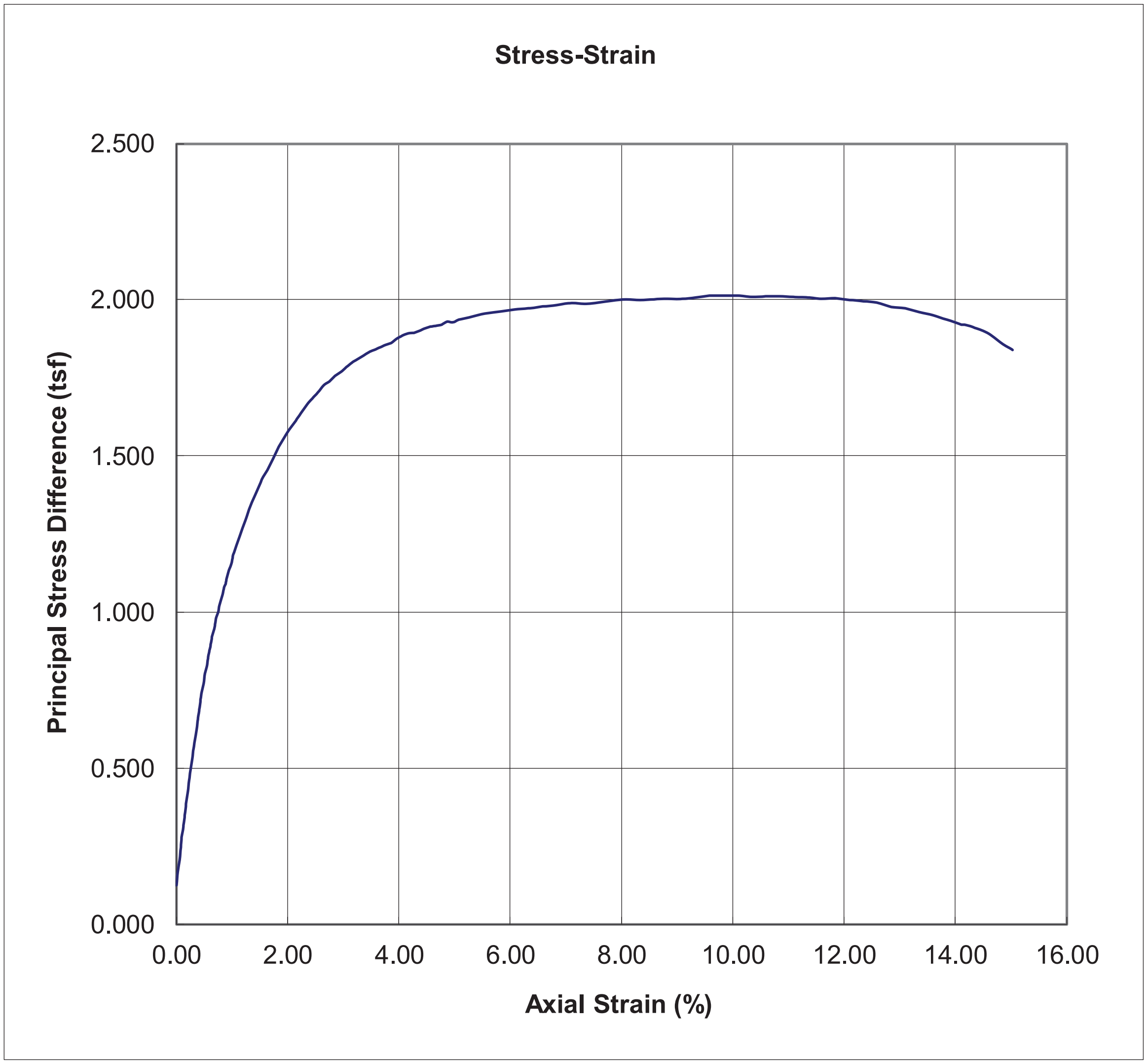
Peak: % Strain: 4.05 **Stress (TSF):** 1.03



Last Revised: 09/20/23 Rev. 0

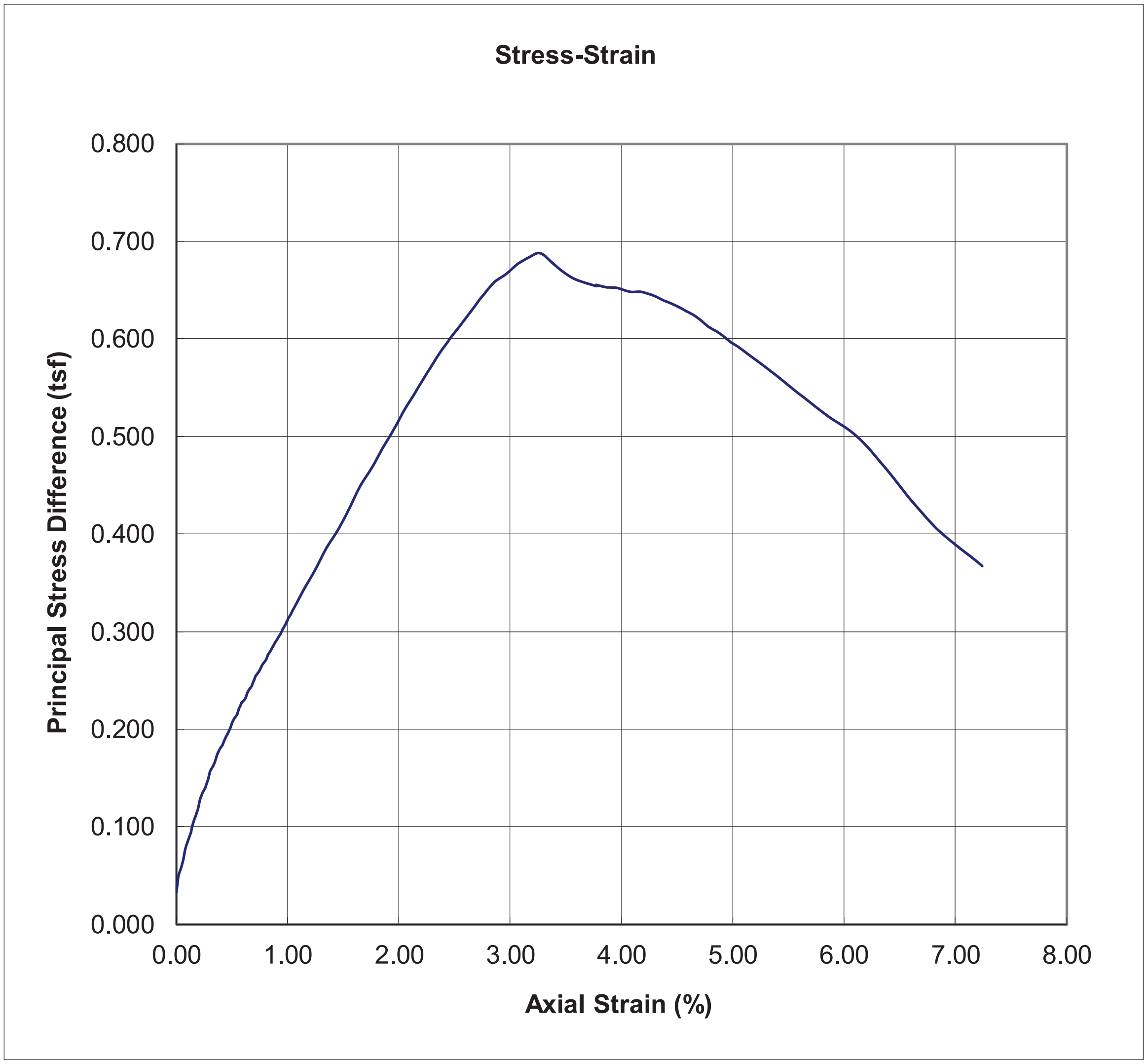
Project: G154-21
Boring: B-1
Depth (ft): 35
Date: 3/13/2024

Peak: %Strain: 10.08 **Stress (TSF):** 2.01



Project: G154-21
Boring: B-1
Depth (ft): 65
Date: 3/12/2024

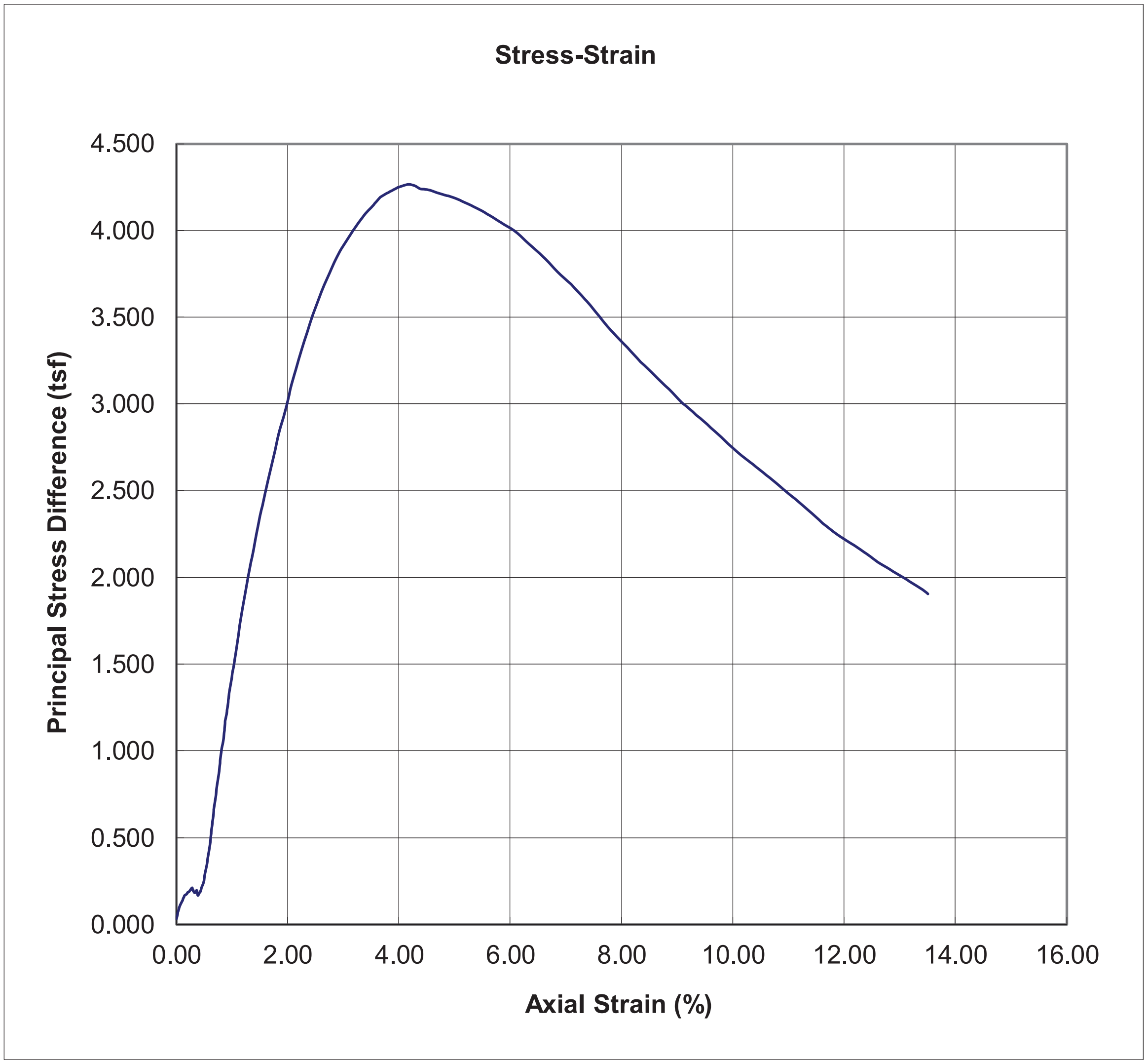
Peak: %Strain: 3.27 **Stress (TSF):** 0.69



Last Revised: 09/20/23 Rev. 0

Project: G154-21
Boring: B-2
Depth (ft): 55
Date: 3/13/2024

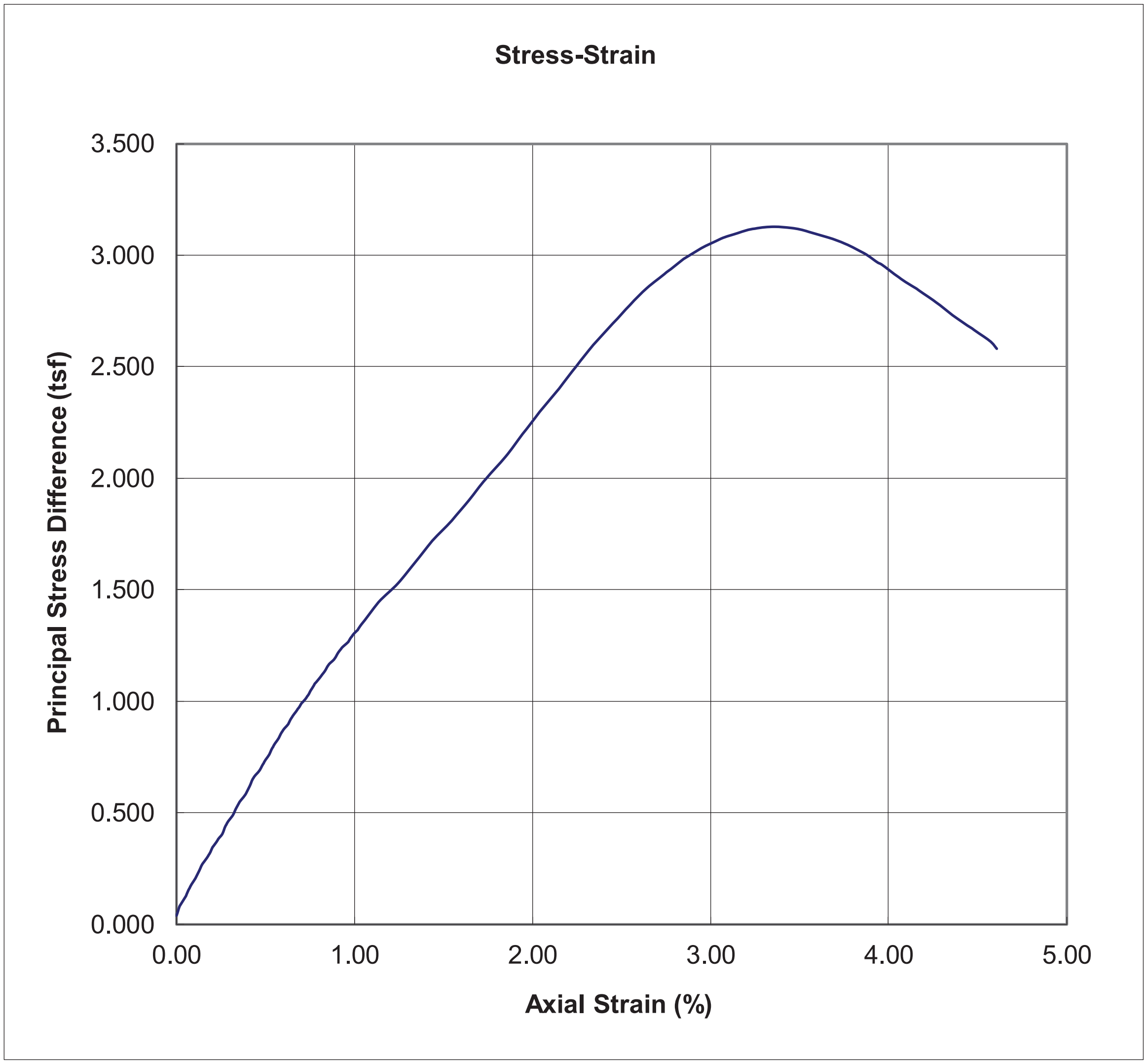
Peak: %Strain: 4.17 **Stress (TSF):** 4.26



Last Revised: 09/20/23 Rev. 0

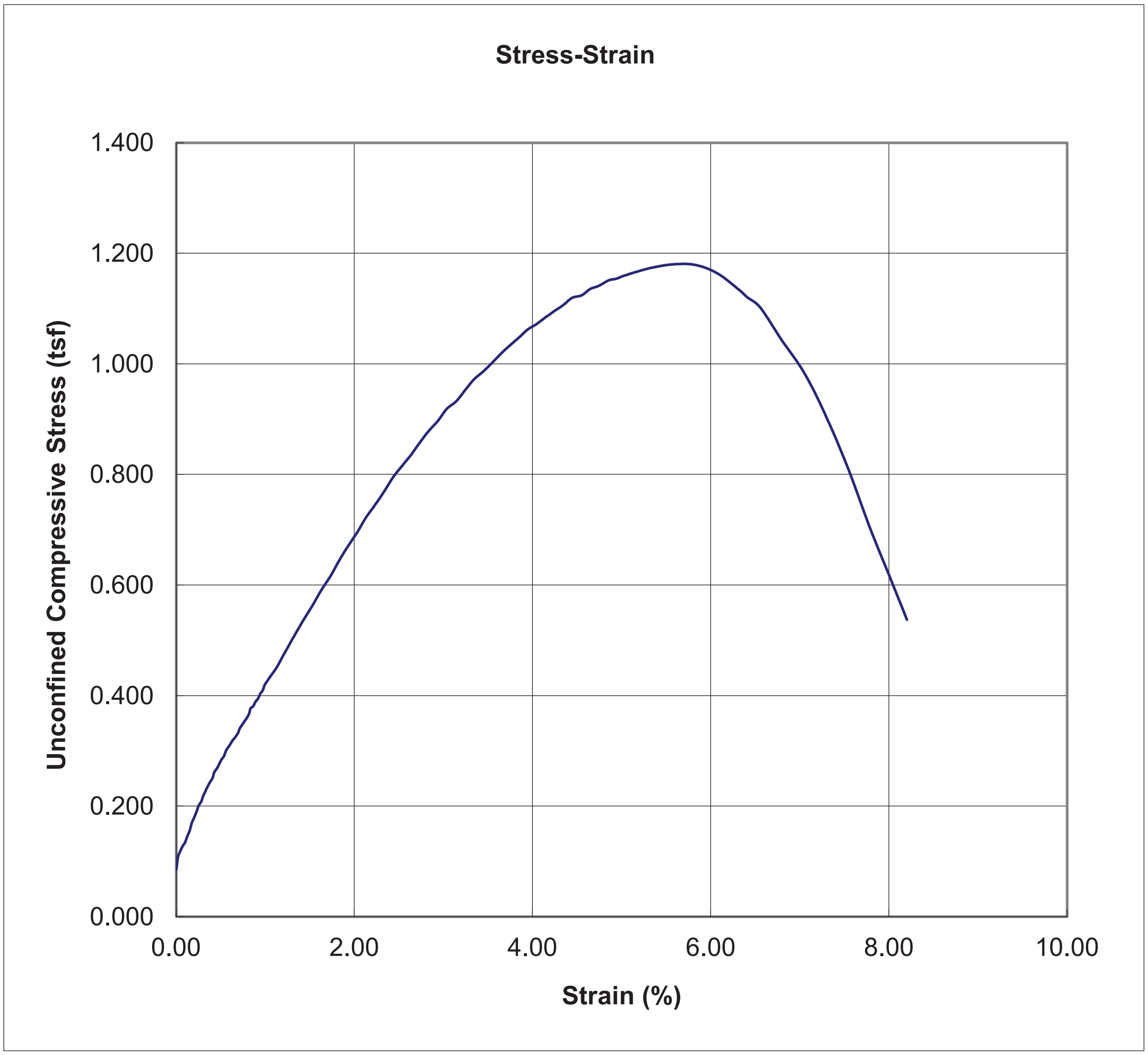
Project: G154-21
Boring: B-2
Depth (ft): 90
Date: 3/12/2024

Peak: %Strain: 3.36 **Stress (TSF):** 3.13



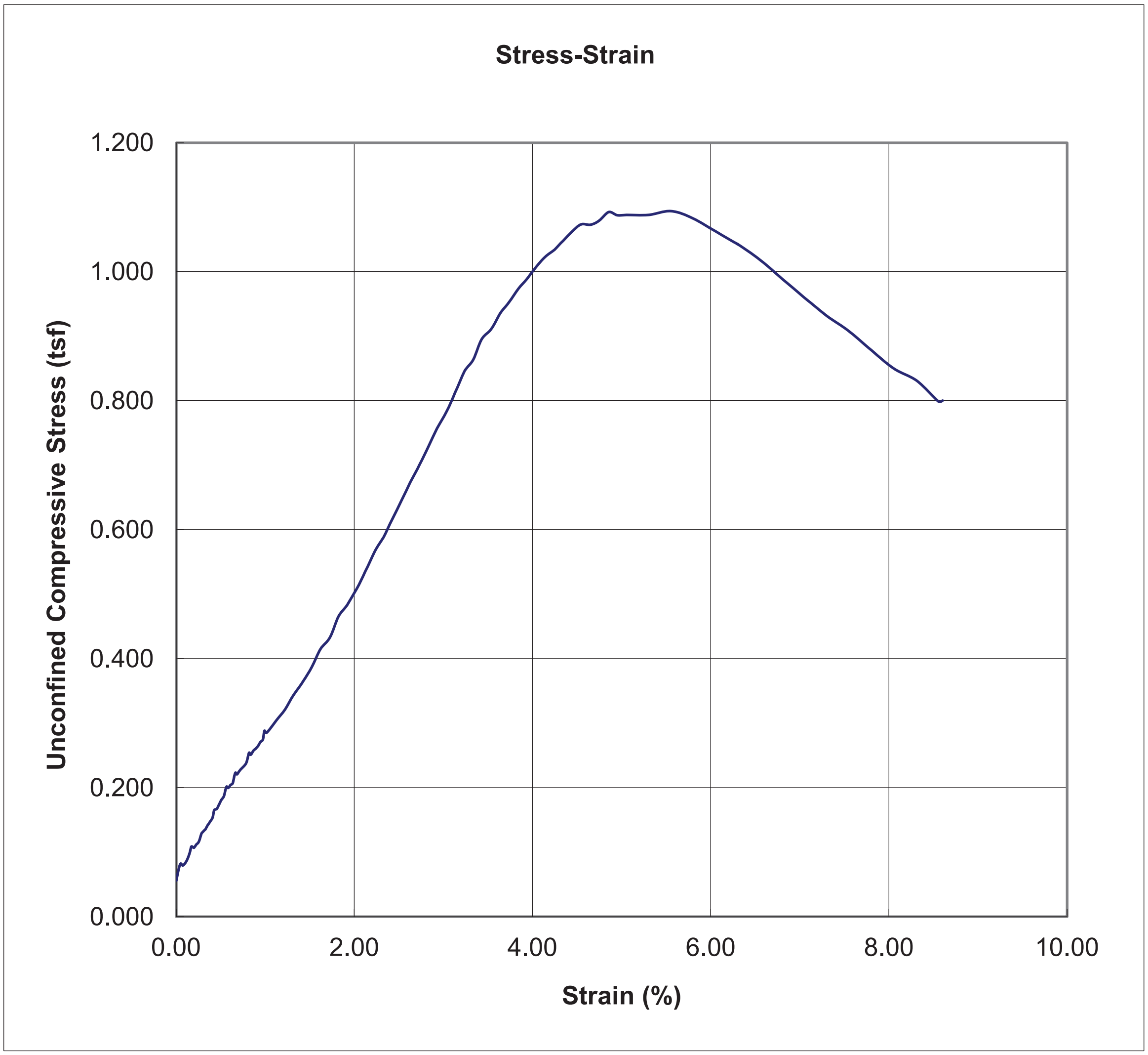
Project: G154-21
Boring: B-3
Depth (ft): 6
Date: 3/9/2024

Peak: % Strain: 5.55 **Stress (TSF):** 1.18



Project: G154-21
Boring: B-4
Depth (ft): 6
Date: 3/8/2024

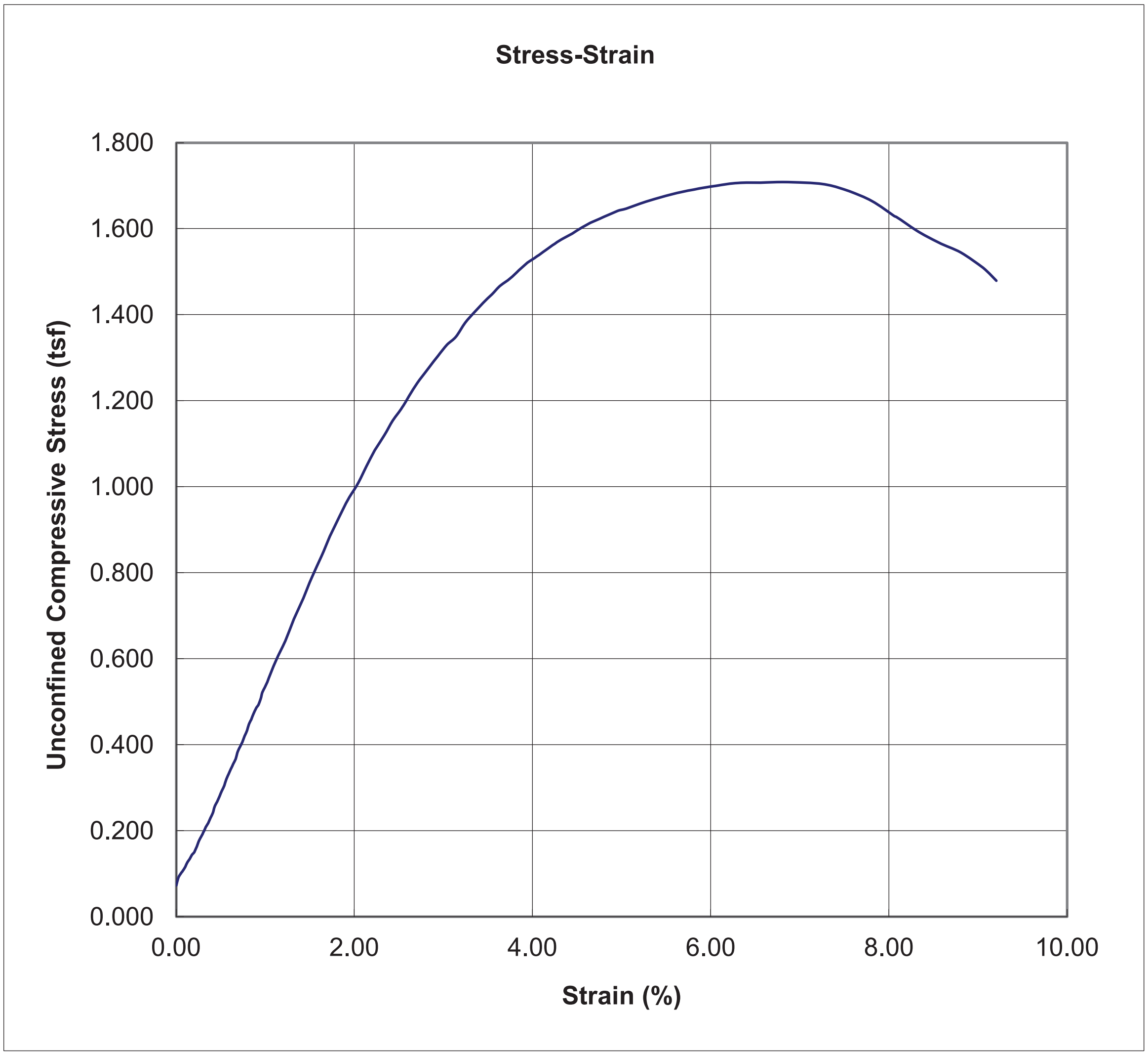
Peak: % Strain: 5.55 Stress (TSF): 1.09



Last Revised: 09/20/23 Rev. 0

Project: G154-21
Boring: B-4
Depth (ft): 60
Date: 3/9/2024

Peak: % Strain: 6.80 **Stress (TSF):** 1.71



APPENDIX C-5

Fugro USA Land, Inc., Automatic Hammer Energy Calibration Memo

FUGRO USA LAND, INC.
6100 Hillcroft Avenue
Houston, Texas 77081
United States of America

Report No. 222213-REP1
March 2, 2023

Van and Sons Drilling Service, Inc.
319 John Alber
Houston, Texas 77076

Attention: Mr. Nicholas Van Antwerp
Vice President of Operations

TCP Hammer Energy Calibrations
Automatic TCP Hammer
Mobile B-60 Truck-Mounted Drill Rig
Houston, Texas

Fugro USA Land, Inc. (Fugro) is pleased to present the results of our Texas Cone Penetrometer (TCP) hammer energy calibrations for the Automatic TCP Hammer mounted on the Mobile B-60 Truck-Mounted Drill Rig. A total of five energy measurements were completed for the purpose of hammer energy calibration on January 4, 2023 in Houston, Texas. This report summarizes the average transferred energies (EMX) and the average energy transfer ratios (ETR) for each data set. A computed correction factor for the automatic TCP hammer is presented.

Objective

The objective for collecting the TCP hammer energy measurements was to determine the energy transfer efficiency and compute a correction factor (hammer calibration) for the automatic TCP hammer on the drill rig.

Energy Measurements

Test Boring and Drilling Method. TCP hammer energy measurements for the Automatic TCP Hammer were obtained from one soil boring completed at a project site in Houston, Texas. The soil boring was drilled using wet rotary drilling techniques. TCP hammer energy measurements were obtained at five depths (99, 103.5, 109, 113.5 and 118.8 ft below existing grade).

Texas Cone Penetrometer Test. The TCP tests were performed in general accordance with the TxDOT Test Procedure Tex-132-E Texas Cone Penetrometer. The TCP test uses a 170-lb drop hammer weight falling from a drop height of 24 inches which provides a theoretical energy (rated energy) of 0.34 kip-ft per hammer blow. The weight of the TCP drop hammer was measured and observed to be 173.3 lbs immediately prior to the hammer calibration. The TCP drop hammer impacts an anvil attached to the top of the drill stem which is attached to a 3-inch diameter penetrometer cone (set on the bottom of the cored hole). The dimensions of the TCP conical driving point (cone) were also measured and observed to be within the specified tolerances stated in Tex-132-E.

PDA Computer and Instrumentation. TCP hammer energy measurements were performed in general accordance with ASTM D4633 "Standard Test Method for Energy Measurement of Dynamic Penetrometers" using a Pile Driving Analyzer (PDA) Model PAX in conjunction with an instrumented 0.61 m (2-ft) section of NWJ drilling rod (SN 333NWJ) manufactured by Pile Dynamics, Inc. (PDI) in Cleveland, Ohio. The instrumented rod section consists of two strain sensors (mounted to cancel any bending in the rod section) and two accelerometers. The PDA computer uses the strain data to obtain a force record and the accelerometer data to obtain a velocity record. The PDA Operator reviews the collected data in real time, performs data quality checks and then digitally saves the data for subsequent analysis in the office.

Energy Calculations. The PDA computer computes the net energy delivered to the instrumented rod section during each hammer blow by integrating the product of force and velocity over the time of the hammer impact. The following equation is used to compute the maximum energy transferred to the rod at the gage location:

$$\mathrm{EMX} = \mathrm{EFV} = \int_a^b F(t) \cdot v(t)\, dt$$

The time "a" corresponds to the start of the record just before impact of the hammer and time "b" is the time the transferred energy reaches a maximum value.

Test Results

Plots and tables prepared from the PDA data for each data set are presented in Attachment 1. Each PDA plot contains three main graphs. The left-hand graph shows CSX (maximum average compressive stress) and CSI (maximum individual gage stress) plotted versus blow number. Together these plots show the difference between the two force measurements. The center plot shows EMX (maximum energy) and ETR (energy transfer ratio which is EMX divided by theoretical hammer energy). These plots indicate the hammer efficiency and consistency during testing. The right-hand graph shows FMX (maximum force) and BPM (blows per minute) to give a relative hammer "operational performance" during testing. The tabulated output for each data set also includes statistical evaluations: average, maximum, and minimum. To arrive at overall performance of the hammer tested, we used the average statistical evaluations from the data sets, shown in Table 1.

The hammer correction factor is based on a "standardized" energy transfer efficiency of 60 percent of the full-rated TCP hammer energy (N_{60}). The uncorrected TCP N-values ($N_{measured}$) can be corrected to the standardized N_{60} using the following equations:

$$N_{60} = N_{measured} \times \text{Correction Factor}$$

where

$$\text{Correction Factor} = (\text{ETR}/60\%)$$

An average of all the ETR values for each data set was used in the above equation to calculate an average correction factor. The correction factor for the TCP automatic hammer is based on our evaluation of all five PDA data sets. Table 1 presents a summary of the overall TCP hammer performance.

Table 1: Summary of Hammer Performance – Automatic TCP Hammer on Mobile B-60 Truck Mounted Drill Rig

Test Date	Data Set No.	Test Depth (ft)	BPM	FMX (kips)	EMX (kip-ft)	ETR (%)	TCP Blow Counts
01-04-2023	DS-1	99	31.3	33.45	0.288	84.7	12/1.5", 50/2.5", 50/1.75"
01-04-2023	DS-3	103.5	32.6	35.29	0.308	90.6	12/1.25", 50/2", 50/1.5"
01-04-2023	DS-4	109	32.8	36.34	0.300	88.4	12/2.5", 47, 20
01-04-2023	DS-4	113.5	31.3	34.11	0.303	89.2	9, 13, 14
01-04-2023	DS-5	118.8	34.8	36.42	0.289	85.1	14/2", 51/0.5", 51/0.1"
Average Overall Performance			**32.6**	**35.12**	**0.298**	**87.6**	**Correction Factor = 1.46**

Conclusions

The TCP hammer energy measurements presented in Table 1 above summarize the average computed transferred energies (EMX) in accordance with ASTM D4633. The correction factor presented in Table 1 was computed based on the average overall energy transfer ratio (ETR) and the standardized 60 percent energy transfer. The average Correction Factor for the Automatic TCP Hammer on the Mobile B-60 Truck-Mounted Drill Rig is 1.46. The average energy transfer ratio (ETR) of the TCP hammer was computed to be 87.6 percent.

Limitations

Fugro USA Land, Inc. warrants that it's services for this study were performed with a degree of care and skill equal to that ordinarily exercised under similar conditions by reputable members of our profession. No other warranty, express or implied, is made or intended.

* * *

The following attachment is included and complete this report.

Attachment

Attachment – PDA Data Summary Plots and Tables
Automatic TCP Hammer on Mobile B-60 Drill Rig.......................... (18 pages)

Closing

We are pleased to be of assistance on the hammer calibration for the Automatic TCP Hammer on the Mobile B-60 Truck-Mounted Drill Rig. Please do not hesitate to contact us should you have questions regarding the content of this report, or if we may be of further service.

Sincerely,
FUGRO USA LAND, INC.
Texas Engineering Firm F-299

Wickrama (Wick) B. Galagoda, P.E.
Project Manager

Copies Submitted; Addressee (1) - by Email

ATTACHMENT

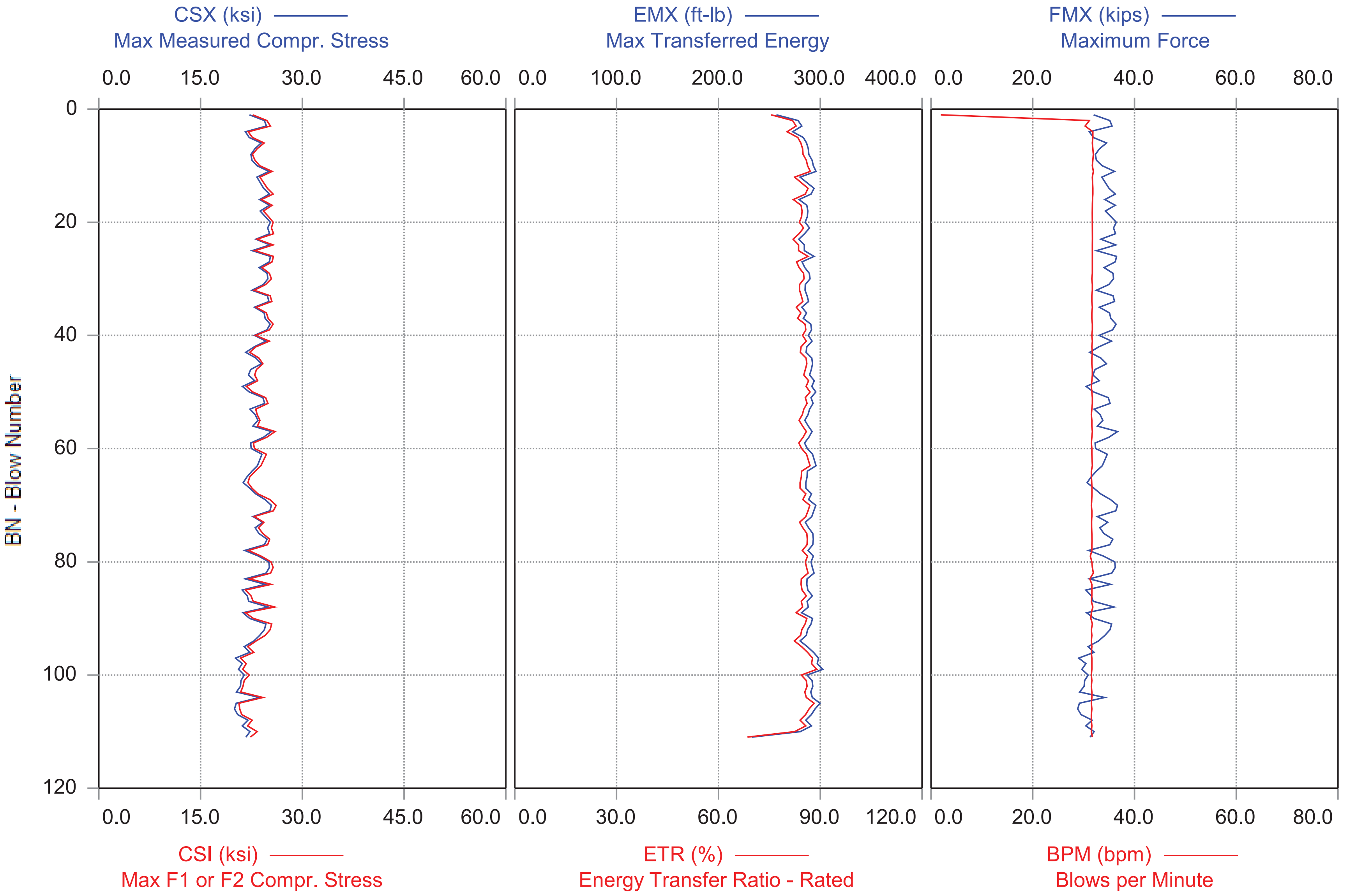
PDA Data Summary Plots and Tables
Automatic TCP Hammer on Mobile B-60 Truck-Mounted Drill Rig

Fugro Consultants, Inc. - PDIPLOT2 Ver 2022.1.62.0 - Case Method & iCAP® Results

Printed: 05-January-2023

Test started: 04-January-2023

VAN AND SONS - DS-1 99FT



Case Method & iCAP® Results

VAN AND SONS - DS-1 99FT A
OP: SW Date: 04-January-2023

AR: 1.44 in^2 SP: 0.492 k/ft^3
LE: 103.50 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio - Rated
FMX: Maximum Force
BPM: Blows per Minute

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
1	99.01	96.00	22.20	22.68	256.854	75.5	31.96	1.9
2	99.02	96.00	24.40	24.79	278.262	81.8	35.14	31.1
3	99.03	96.00	24.71	25.31	281.953	82.9	35.58	30.3
4	99.04	96.00	21.62	22.04	272.880	80.3	31.14	31.8
5	99.05	96.00	22.17	22.64	283.533	83.4	31.93	31.8
6	99.06	96.00	23.94	24.39	286.628	84.3	34.48	31.7
7	99.07	96.00	23.01	23.37	288.417	84.8	33.13	31.8
8	99.08	96.00	22.42	22.65	288.864	85.0	32.28	31.9
9	99.09	96.00	22.57	23.01	292.080	85.9	32.50	31.8
10	99.10	96.00	23.33	23.70	293.235	86.2	33.60	31.7
11	99.11	96.00	25.06	25.55	296.029	87.1	36.09	31.9
12	99.13	96.00	23.30	23.74	280.318	82.4	33.56	31.7
14	99.13	240.00	24.31	24.85	293.831	86.4	35.01	31.8
15	99.14	240.00	25.15	25.70	290.964	85.6	36.21	31.8
16	99.14	240.00	23.71	23.98	279.174	82.1	34.15	31.7
17	99.15	240.00	25.16	25.57	286.781	84.3	36.23	31.7
18	99.15	240.00	23.79	24.28	287.894	84.7	34.26	31.7
19	99.15	240.00	24.58	25.04	287.440	84.5	35.39	31.7
20	99.16	240.00	25.30	25.71	285.189	83.9	36.43	31.7
21	99.16	240.00	24.91	25.43	289.442	85.1	35.87	31.7
22	99.17	240.00	25.18	25.77	284.745	83.7	36.26	31.7
23	99.17	240.00	23.16	23.37	278.921	82.0	33.36	31.7
24	99.18	240.00	25.21	25.70	284.428	83.7	36.31	31.7
25	99.18	240.00	22.60	23.02	284.440	83.7	32.55	31.7
26	99.18	240.00	25.31	25.75	293.874	86.4	36.45	31.7
27	99.19	240.00	25.15	25.54	282.287	83.0	36.22	31.7
28	99.19	240.00	23.63	24.04	284.833	83.8	34.03	31.7
29	99.20	240.00	24.83	25.16	289.361	85.1	35.76	31.7
30	99.20	240.00	24.93	25.45	289.924	85.3	35.89	31.6
31	99.20	240.00	24.28	24.62	285.455	84.0	34.96	31.7
32	99.21	240.00	22.57	22.90	285.247	83.9	32.50	31.7
33	99.21	240.00	24.86	25.27	287.045	84.4	35.80	31.6
34	99.22	240.00	25.07	25.50	288.476	84.8	36.09	31.6
35	99.22	240.00	22.96	23.19	281.976	82.9	33.06	31.7
36	99.23	240.00	24.38	24.72	286.636	84.3	35.10	31.6
37	99.23	240.00	24.53	24.96	283.341	83.3	35.33	31.6
38	99.23	240.00	25.26	25.70	290.789	85.5	36.37	31.7
39	99.24	240.00	24.77	25.16	291.538	85.7	35.67	31.7
40	99.24	240.00	22.94	23.08	288.357	84.8	33.03	31.6
41	99.25	240.00	24.64	25.13	291.956	85.9	35.48	31.7
42	99.25	240.00	22.92	23.18	286.820	84.4	33.01	31.6
43	99.25	240.00	21.64	22.25	286.026	84.1	31.17	31.7
44	99.26	240.00	23.17	23.60	291.848	85.8	33.37	31.6
45	99.26	240.00	23.97	24.20	292.500	86.0	34.52	31.6
46	99.27	240.00	22.41	23.23	291.156	85.6	32.27	31.7
47	99.27	240.00	22.08	22.98	289.621	85.2	31.80	31.7
48	99.28	240.00	22.98	23.42	294.224	86.5	33.09	31.6

Case Method & iCAP® Results

VAN AND SONS - DS-1 99FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
49	99.28	240.00	21.17	21.79	291.758	85.8	30.49	31.5
50	99.28	240.00	22.19	22.68	295.711	87.0	31.95	31.6
51	99.29	240.00	24.21	24.62	291.036	85.6	34.86	31.7
52	99.29	240.00	24.44	24.95	292.906	86.1	35.19	31.7
53	99.30	240.00	22.27	23.14	289.549	85.2	32.07	31.6
54	99.30	240.00	23.06	23.36	288.015	84.7	33.21	31.5
55	99.30	240.00	23.45	23.76	284.931	83.8	33.76	31.6
56	99.31	240.00	22.71	23.37	288.416	84.8	32.71	31.6
57	99.31	240.00	25.45	26.00	292.034	85.9	36.65	31.7
58	99.32	240.00	24.28	24.74	288.862	85.0	34.97	31.6
59	99.32	240.00	22.40	22.80	284.699	83.7	32.25	31.5
60	99.33	240.00	22.45	22.93	287.294	84.5	32.33	31.6
61	99.33	240.00	24.05	24.70	292.373	86.0	34.63	31.6
63	99.34	336.78	23.37	23.91	295.804	87.0	33.66	31.7
64	99.34	336.78	22.58	23.06	287.431	84.5	32.52	31.5
65	99.34	336.78	21.86	22.24	287.063	84.4	31.47	31.5
66	99.35	336.78	21.30	21.98	285.907	84.1	30.67	31.6
67	99.35	336.78	22.23	22.59	285.746	84.0	32.01	31.6
68	99.35	336.78	23.13	23.44	291.505	85.7	33.31	31.6
69	99.35	336.78	24.52	25.21	288.538	84.9	35.31	31.6
70	99.36	336.78	25.46	26.14	295.682	87.0	36.66	31.5
71	99.36	336.78	25.22	25.73	293.756	86.4	36.32	31.6
72	99.36	336.78	22.71	22.89	291.674	85.8	32.70	31.6
73	99.37	336.78	24.14	24.35	285.451	84.0	34.77	31.5
74	99.37	336.78	23.04	23.53	288.572	84.9	33.18	31.6
75	99.37	336.78	23.60	24.21	292.673	86.1	33.99	31.6
76	99.37	336.78	24.81	25.18	293.191	86.2	35.72	31.6
77	99.38	336.78	24.42	24.90	292.804	86.1	35.16	31.6
78	99.38	336.78	21.53	22.07	288.006	84.7	31.00	31.6
79	99.38	336.78	23.56	23.94	293.256	86.3	33.92	31.3
80	99.39	336.78	25.08	25.40	291.087	85.6	36.12	31.6
81	99.39	336.78	25.15	25.72	292.330	86.0	36.22	31.7
82	99.39	336.78	24.66	25.34	293.883	86.4	35.51	31.9
83	99.40	336.78	21.60	22.16	287.029	84.4	31.10	31.2
84	99.40	336.78	24.51	25.42	286.921	84.4	35.30	31.6
85	99.40	336.78	21.15	21.61	287.849	84.7	30.46	31.6
86	99.40	336.78	21.88	22.44	292.118	85.9	31.51	31.6
87	99.41	336.78	22.11	22.75	287.133	84.5	31.84	31.5
88	99.41	336.78	24.97	25.93	288.224	84.8	35.95	31.8
89	99.41	336.78	21.24	21.62	281.791	82.9	30.59	31.5
90	99.42	336.78	22.25	22.82	292.537	86.0	32.04	31.4
91	99.42	336.78	24.63	25.48	290.910	85.6	35.47	31.7
92	99.42	336.78	24.42	25.27	287.595	84.6	35.17	31.5
93	99.43	336.78	23.71	24.52	286.278	84.2	34.14	31.6
94	99.43	336.78	22.86	23.20	280.130	82.4	32.92	31.5
95	99.43	336.78	21.45	21.96	287.375	84.5	30.89	31.6
96	99.43	336.78	22.26	22.85	293.408	86.3	32.06	31.5
97	99.44	336.78	20.16	20.89	298.446	87.8	29.04	31.6
98	99.44	336.78	21.13	21.72	297.254	87.4	30.43	31.6
99	99.44	336.78	20.59	21.22	302.487	89.0	29.65	31.6
100	99.45	336.78	21.45	22.17	287.192	84.5	30.89	31.5
101	99.45	336.78	20.95	21.42	292.076	85.9	30.17	31.6
102	99.45	336.78	20.90	21.25	292.730	86.1	30.09	31.5

Case Method & iCAP® Results

VAN AND SONS - DS-1 99FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
103	99.46	336.78	20.30	20.93	290.663	85.5	29.23	31.6
104	99.46	336.78	23.69	24.19	292.130	85.9	34.11	31.6
105	99.46	336.78	20.26	20.67	299.719	88.2	29.17	31.5
106	99.46	336.78	20.01	20.79	294.872	86.7	28.81	31.6
107	99.47	336.78	20.49	21.09	291.310	85.7	29.51	31.5
108	99.47	336.78	21.97	22.62	285.936	84.1	31.64	31.5
109	99.47	336.78	21.14	21.92	291.533	85.7	30.44	31.6
110	99.48	336.78	22.28	23.34	280.373	82.5	32.08	31.5
111	99.48	336.78	21.71	22.37	233.018	68.5	31.26	31.7
		Average	23.23	23.73	288.134	84.7	33.45	31.3
		Maximum	25.46	26.14	302.487	89.0	36.66	31.9
		Minimum	20.01	20.67	233.018	68.5	28.81	1.9

Total number of blows analyzed: 109

BL#	Sensors
1-111	F1: [333NWJ-2] 214.5 (1.00); F2: [333NWJ-2] 201.7 (1.00); A1: [K11483] 416.9 (1.00); A2: [K11485] 436.6 (1.00)

Time Summary

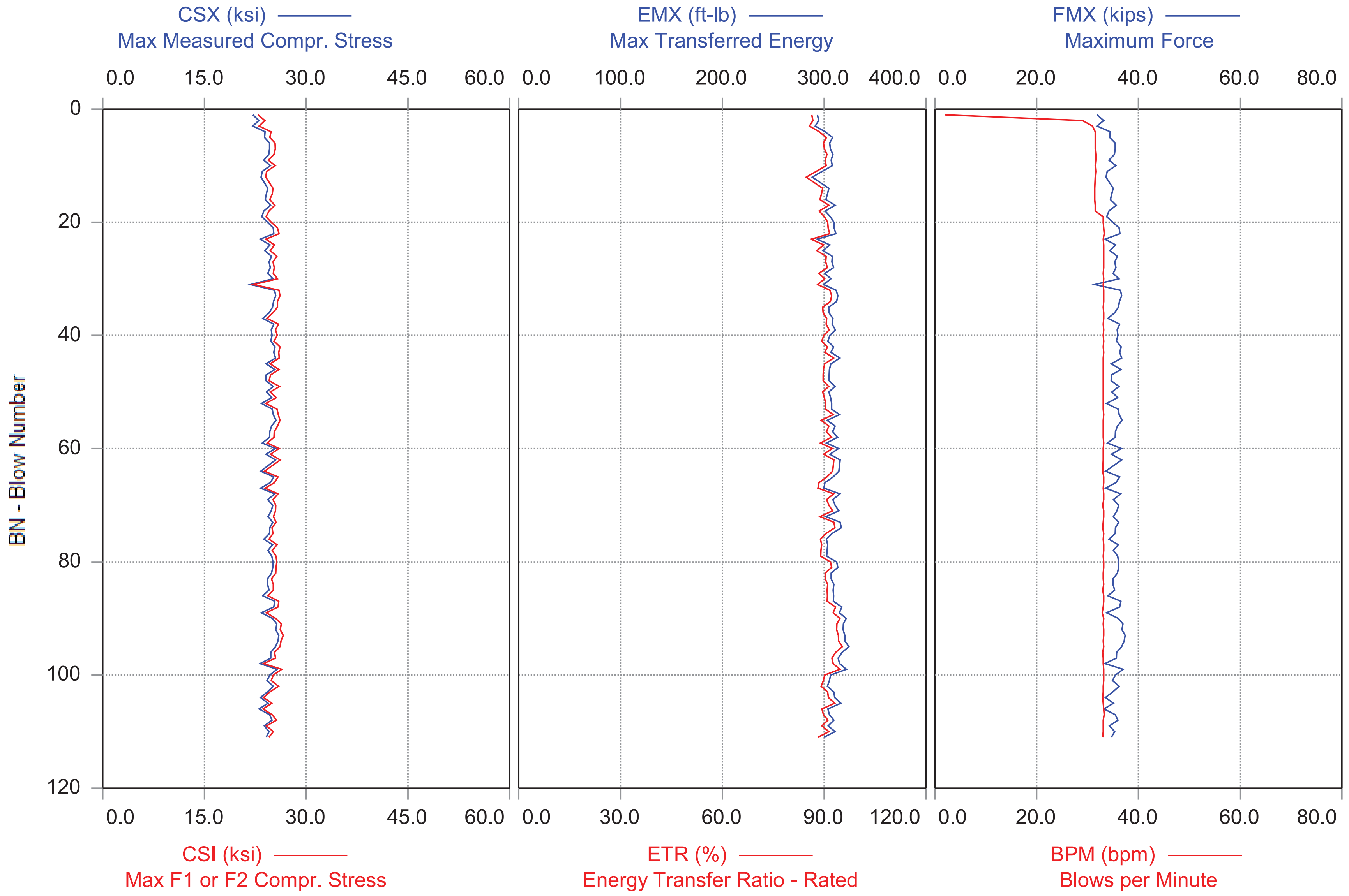
Drive 5 minutes 14 seconds 10:24 AM - 10:30 AM BN 1 - 111

Fugro Consultants, Inc. - PDIPLOT2 Ver 2022.1.62.0 - Case Method & iCAP® Results

Printed: 05-January-2023

Test started: 04-January-2023

VAN AND SONS - DS-2 103.5FT



Case Method & iCAP® Results

VAN AND SONS - DS-2 103.5FT A
OP: SW Date: 04-January-2023

AR: 1.44 in² SP: 0.492 k/ft³
LE: 108.50 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

CSX: Max Measured Compr. Stress ETR: Energy Transfer Ratio - Rated
CSI: Max F1 or F2 Compr. Stress FMX: Maximum Force
EMX: Max Transferred Energy BPM: Blows per Minute

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
1	103.51	115.21	22.14	22.91	293.572	86.3	31.88	1.9
2	103.52	115.21	23.03	23.90	294.709	86.7	33.17	29.0
3	103.53	115.21	22.13	23.04	291.289	85.7	31.86	31.0
4	103.53	115.21	23.93	24.82	301.059	88.5	34.46	31.5
5	103.54	115.21	23.86	24.63	308.378	90.7	34.35	31.5
6	103.55	115.21	24.61	25.40	305.364	89.8	35.43	31.5
7	103.56	115.21	24.61	25.46	306.236	90.1	35.44	31.5
8	103.57	115.21	24.49	25.28	308.729	90.8	35.27	31.6
9	103.58	115.21	23.74	24.40	307.034	90.3	34.19	31.6
10	103.59	115.21	24.69	25.44	308.118	90.6	35.56	31.5
11	103.60	115.21	23.53	24.15	298.049	87.7	33.88	31.6
12	103.60	115.21	23.35	24.04	288.194	84.8	33.62	31.5
14	103.61	299.40	24.33	25.13	304.686	89.6	35.03	31.4
15	103.61	299.40	24.13	24.99	303.320	89.2	34.74	31.4
16	103.62	299.40	23.97	24.63	301.882	88.8	34.52	31.4
17	103.62	299.40	24.73	25.38	310.738	91.4	35.61	31.5
18	103.62	299.40	23.76	24.51	301.141	88.6	34.21	31.5
19	103.63	299.40	23.46	24.09	306.217	90.1	33.79	33.1
20	103.63	299.40	24.29	24.86	309.686	91.1	34.98	33.1
21	103.63	299.40	25.14	25.79	309.972	91.2	36.20	33.2
22	103.64	299.40	25.24	26.00	311.696	91.7	36.35	33.3
23	103.64	299.40	23.20	24.06	293.057	86.2	33.42	33.1
24	103.64	299.40	24.68	25.35	305.603	89.9	35.54	33.2
25	103.65	299.40	23.89	24.73	298.906	87.9	34.40	33.2
26	103.65	299.40	24.91	25.65	308.124	90.6	35.87	33.2
27	103.65	299.40	24.52	25.10	307.612	90.5	35.31	33.2
28	103.66	299.40	24.71	25.28	309.280	91.0	35.58	33.2
29	103.66	299.40	24.33	25.15	300.787	88.5	35.04	33.1
30	103.66	299.40	25.11	25.78	306.688	90.2	36.16	33.2
31	103.67	299.40	21.80	22.23	299.685	88.1	31.40	33.1
32	103.67	299.40	25.33	25.98	311.762	91.7	36.47	33.2
33	103.67	299.40	25.51	26.13	313.602	92.2	36.73	33.2
34	103.68	299.40	25.18	25.76	312.291	91.9	36.26	33.2
35	103.68	299.40	25.05	25.79	304.512	89.6	36.07	33.1
36	103.68	299.40	24.52	25.09	305.006	89.7	35.31	33.2
37	103.69	299.40	23.63	24.24	308.540	90.7	34.02	33.2
38	103.69	299.40	25.21	25.91	307.987	90.6	36.30	33.1
39	103.69	299.40	24.87	25.48	311.036	91.5	35.81	33.2
40	103.70	299.40	24.97	25.72	305.811	89.9	35.95	33.1
41	103.70	299.40	24.78	25.28	303.693	89.3	35.69	33.2
42	103.70	299.40	25.41	26.14	309.441	91.0	36.60	33.1
43	103.71	299.40	25.21	25.95	306.732	90.2	36.30	33.2
44	103.71	299.40	25.49	26.05	315.539	92.8	36.70	33.1
45	103.71	299.40	24.07	24.66	306.604	90.2	34.66	33.1
46	103.72	299.40	25.38	25.99	305.328	89.8	36.54	33.1
47	103.72	299.40	24.09	24.75	305.071	89.7	34.69	33.1
48	103.72	299.40	24.08	24.51	304.983	89.7	34.67	33.1

Case Method & iCAP® Results

VAN AND SONS - DS-2 103.5FT A
OP: SW Date: 04-January-2023

BL#	Depth	BLC	CSX	CSI	EMX	ETR	FMX	BPM
	ft	bl/ft	ksi	ksi	ft-lb	(%)	kips	bpm
49	103.73	299.40	25.13	26.05	310.504	91.3	36.19	33.1
50	103.73	299.40	24.16	24.71	304.687	89.6	34.80	33.1
51	103.73	299.40	24.94	25.58	306.422	90.1	35.92	33.1
52	103.74	299.40	23.41	24.04	307.521	90.4	33.71	33.1
53	103.74	299.40	25.02	25.72	307.518	90.4	36.03	33.1
54	103.74	299.40	25.16	25.88	315.175	92.7	36.24	33.2
55	103.75	299.40	25.55	26.14	303.037	89.1	36.79	33.1
56	103.75	299.40	24.90	25.77	310.872	91.4	35.85	33.1
57	103.75	299.40	24.63	25.29	308.438	90.7	35.47	33.1
58	103.76	299.40	24.62	25.25	313.359	92.2	35.45	33.1
59	103.76	299.40	23.54	24.25	302.294	88.9	33.89	33.2
60	103.76	299.40	25.46	25.99	314.225	92.4	36.67	33.1
61	103.77	299.40	24.08	24.76	305.649	89.9	34.68	33.1
62	103.77	299.40	25.49	26.17	315.791	92.9	36.71	33.1
64	103.78	392.00	23.31	23.86	314.558	92.5	33.57	33.0
65	103.78	392.00	25.23	25.86	308.876	90.8	36.34	33.2
66	103.78	392.00	24.70	25.34	300.729	88.4	35.57	33.2
67	103.78	392.00	23.28	23.95	299.801	88.2	33.53	33.1
68	103.79	392.00	25.37	25.85	315.583	92.8	36.53	33.2
69	103.79	392.00	24.36	25.10	308.588	90.8	35.08	33.2
70	103.79	392.00	25.07	25.53	310.753	91.4	36.10	33.0
71	103.79	392.00	24.85	25.47	314.446	92.5	35.78	33.2
72	103.80	392.00	24.38	25.20	302.106	88.9	35.11	33.2
73	103.80	392.00	25.08	25.55	315.802	92.9	36.12	33.1
74	103.80	392.00	24.63	24.98	316.886	93.2	35.47	33.0
75	103.80	392.00	24.59	25.10	307.877	90.6	35.41	33.2
76	103.81	392.00	23.75	24.54	302.271	88.9	34.20	33.1
77	103.81	392.00	25.03	25.68	303.545	89.3	36.05	33.2
78	103.81	392.00	24.37	25.01	302.909	89.1	35.09	33.2
79	103.81	392.00	24.95	25.55	302.521	89.0	35.92	33.1
80	103.82	392.00	25.10	25.66	312.377	91.9	36.14	33.2
81	103.82	392.00	25.07	25.56	313.472	92.2	36.10	33.2
82	103.82	392.00	24.91	25.50	306.955	90.3	35.87	33.1
83	103.82	392.00	24.29	24.89	306.951	90.3	34.98	33.1
84	103.83	392.00	24.30	25.14	309.630	91.1	34.99	33.2
85	103.83	392.00	24.53	25.14	309.014	90.9	35.32	33.0
86	103.83	392.00	23.61	24.40	309.350	91.0	34.00	33.2
87	103.83	392.00	25.38	25.96	309.166	90.9	36.55	33.2
88	103.84	392.00	25.24	25.83	317.559	93.4	36.34	33.1
89	103.84	392.00	23.40	24.14	315.138	92.7	33.70	32.9
90	103.84	392.00	25.04	25.53	321.743	94.6	36.06	33.2
91	103.85	392.00	25.66	26.31	318.915	93.8	36.95	33.1
92	103.85	392.00	25.51	26.18	318.683	93.7	36.74	33.2
93	103.85	392.00	25.95	26.63	320.348	94.2	37.37	33.2
94	103.85	392.00	25.84	26.28	320.458	94.3	37.21	33.1
95	103.86	392.00	25.49	26.15	324.420	95.4	36.71	33.2
96	103.86	392.00	24.80	25.32	317.691	93.4	35.71	33.0
97	103.86	392.00	24.78	25.47	313.832	92.3	35.68	33.1
98	103.86	392.00	23.21	23.72	315.168	92.7	33.42	33.1
99	103.87	392.00	25.69	26.39	321.885	94.7	37.00	33.2
100	103.87	392.00	24.63	25.16	306.500	90.1	35.46	33.2
101	103.87	392.00	24.25	24.84	305.323	89.8	34.92	33.2
102	103.87	392.00	25.14	25.91	303.200	89.2	36.20	33.1

Case Method & iCAP® Results

VAN AND SONS - DS-2 103.5FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
103	103.88	392.00	24.27	24.62	309.613	91.1	34.94	33.1
104	103.88	392.00	23.27	23.73	310.439	91.3	33.51	33.0
105	103.88	392.00	24.38	24.94	316.491	93.1	35.11	33.1
106	103.88	392.00	23.06	23.63	303.734	89.3	33.21	33.2
107	103.89	392.00	24.61	24.97	305.200	89.8	35.44	33.3
108	103.89	392.00	24.98	25.63	309.698	91.1	35.97	33.1
109	103.89	392.00	23.82	24.16	303.916	89.4	34.31	33.1
110	103.89	392.00	24.53	25.16	310.867	91.4	35.33	33.1
111	103.90	392.00	24.11	24.51	300.028	88.2	34.71	33.0
		Average	24.50	25.15	308.057	90.6	35.29	32.6
		Maximum	25.95	26.63	324.420	95.4	37.37	33.3
		Minimum	21.80	22.23	288.194	84.8	31.40	1.9

Total number of blows analyzed: 109

BL#	Sensors
1-111	F1: [333NWJ-2] 214.5 (1.00); F2: [333NWJ-2] 201.7 (1.00); A1: [K11483] 416.9 (1.00); A2: [K11485] 436.6 (1.00)

Time Summary

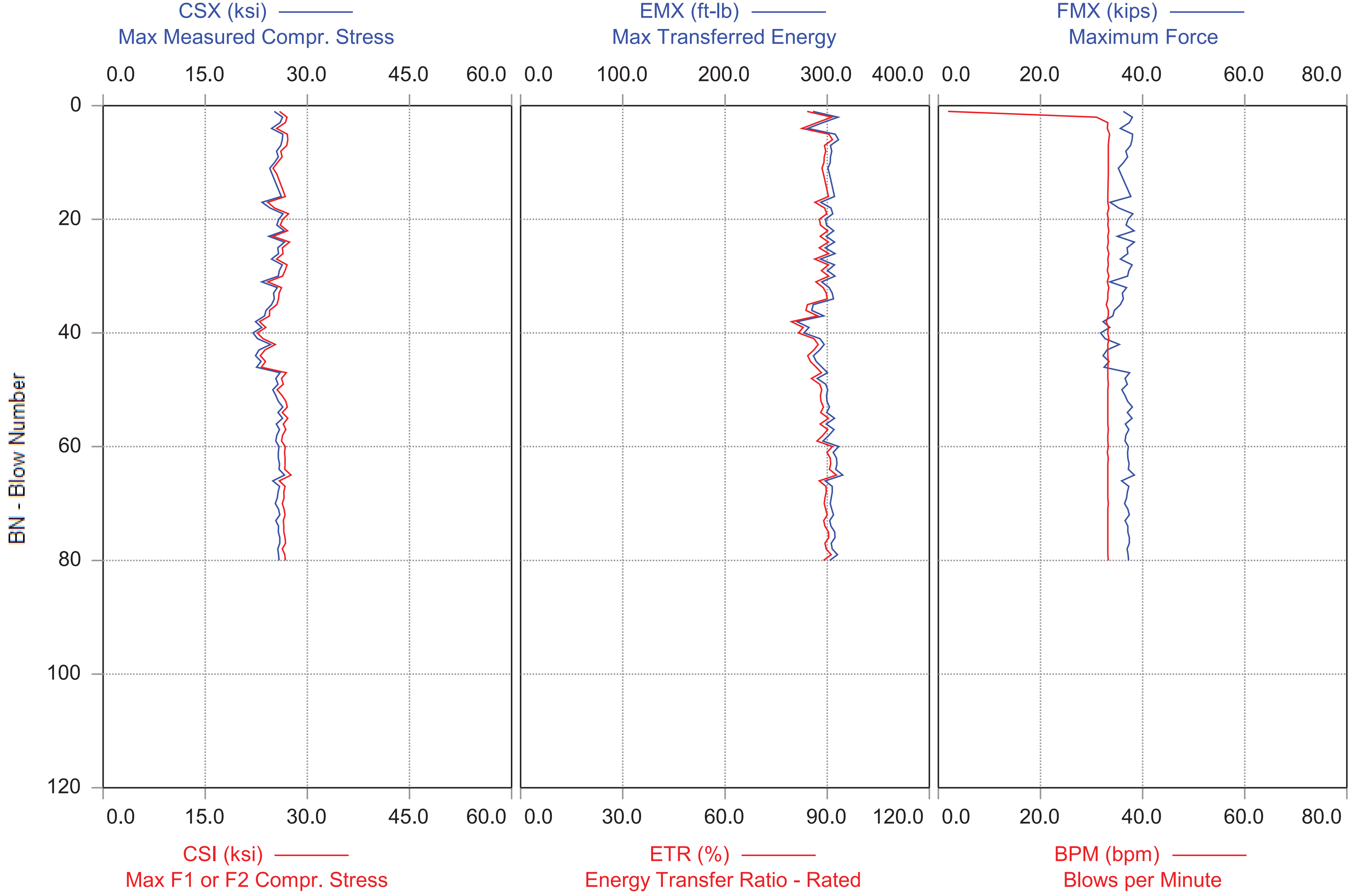
Drive 4 minutes 52 seconds 10:56 AM - 11:01 AM BN 1 - 111

Fugro Consultants, Inc. - PDIPLOT2 Ver 2022.1.62.0 - Case Method & iCAP® Results

Printed: 05-January-2023

Test started: 04-January-2023

VAN AND SONS - DS-3 109FT



Fugro Consultants, Inc. Page 1
PDIPLOT2 2022.1.62.0 Printed 05-January-2023

Case Method & iCAP® Results

VAN AND SONS - DS-3 109FT A
OP: SW Date: 04-January-2023

AR: 1.44 in² SP: 0.492 k/ft³
LE: 113.50 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio - Rated
FMX: Maximum Force
BPM: Blows per Minute

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
1	109.02	57.60	25.16	25.91	286.166	84.2	36.23	1.9
2	109.03	57.60	26.37	27.01	310.523	91.3	37.98	31.0
3	109.05	57.60	25.98	26.75	295.709	87.0	37.42	33.2
4	109.07	57.60	24.75	25.49	280.844	82.6	35.64	33.1
5	109.09	57.60	26.40	27.04	307.877	90.6	38.01	33.5
6	109.10	57.60	26.33	27.15	311.218	91.5	37.91	33.4
7	109.12	57.60	26.12	26.97	303.217	89.2	37.62	33.3
8	109.14	57.60	25.49	26.11	304.511	89.6	36.71	33.3
9	109.16	57.60	25.74	26.31	303.169	89.2	37.06	33.3
10	109.17	57.60	25.20	25.62	302.914	89.1	36.29	33.3
11	109.19	57.60	24.47	24.98	300.731	88.5	35.24	33.3
12	109.21	57.60	24.81	25.52	302.266	88.9	35.72	33.3
16	109.25	94.00	26.17	26.78	307.216	90.4	37.68	33.2
17	109.26	94.00	23.35	24.19	293.769	86.4	33.63	33.2
18	109.27	94.00	24.54	25.18	303.699	89.3	35.33	33.4
19	109.28	94.00	26.45	27.23	305.529	89.9	38.09	33.1
20	109.29	94.00	25.80	26.39	298.254	87.7	37.15	33.3
21	109.30	94.00	25.52	26.02	299.239	88.0	36.75	33.2
22	109.31	94.00	26.62	27.11	306.550	90.2	38.33	33.4
23	109.33	94.00	24.34	24.97	299.048	88.0	35.05	33.2
24	109.34	94.00	26.66	27.40	307.236	90.4	38.39	33.3
25	109.35	94.00	25.66	26.32	298.018	87.7	36.96	33.1
26	109.36	94.00	25.77	26.45	307.635	90.5	37.12	33.4
27	109.37	94.00	24.74	25.47	293.543	86.3	35.63	33.2
28	109.38	94.00	26.33	27.04	307.228	90.4	37.92	33.3
29	109.39	94.00	25.89	26.73	300.229	88.3	37.27	33.1
30	109.40	94.00	25.75	26.36	307.622	90.5	37.07	33.4
31	109.41	94.00	23.33	24.21	294.745	86.7	33.59	33.1
32	109.42	94.00	25.60	26.23	302.069	88.8	36.86	33.4
33	109.43	94.00	25.03	25.86	304.992	89.7	36.04	33.2
34	109.44	94.00	25.15	25.76	306.119	90.0	36.21	33.2
35	109.45	94.00	24.76	25.52	286.298	84.2	35.65	32.9
36	109.46	94.00	23.92	24.47	284.785	83.8	34.45	33.3
37	109.47	94.00	23.73	24.47	296.593	87.2	34.16	33.3
38	109.48	94.00	22.40	23.00	270.335	79.5	32.26	32.9
39	109.50	94.00	23.32	23.90	282.133	83.0	33.58	33.3
40	109.51	94.00	22.04	22.66	277.385	81.6	31.74	33.2
41	109.52	94.00	22.68	23.51	292.853	86.1	32.66	33.4
42	109.53	94.00	24.61	25.28	297.015	87.4	35.44	33.2
43	109.54	94.00	22.92	23.78	292.777	86.1	33.01	33.2
44	109.55	94.00	22.40	23.09	286.692	84.3	32.26	33.2
45	109.56	94.00	23.20	23.86	289.305	85.1	33.40	33.3
46	109.57	94.00	22.55	23.23	294.969	86.8	32.47	33.2
47	109.58	94.00	26.00	26.90	300.274	88.3	37.44	33.2
48	109.59	94.00	25.37	26.20	290.305	85.4	36.54	33.2
49	109.60	94.00	25.69	26.46	298.679	87.8	37.00	33.3
50	109.61	94.00	24.93	25.54	300.659	88.4	35.90	33.2

Case Method & iCAP® Results

VAN AND SONS - DS-3 109FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
51	109.62	94.00	25.39	26.30	299.291	88.0	36.56	33.2
52	109.63	94.00	25.74	26.84	299.786	88.2	37.07	33.2
53	109.64	94.00	26.39	27.07	302.207	88.9	38.00	33.2
54	109.66	94.00	25.69	26.33	299.540	88.1	37.00	33.2
55	109.67	94.00	26.35	27.13	307.200	90.4	37.94	33.2
56	109.68	94.00	25.43	26.48	299.031	88.0	36.62	33.2
57	109.69	94.00	25.91	26.84	306.692	90.2	37.32	33.3
58	109.70	94.00	25.50	26.35	301.781	88.8	36.72	33.2
59	109.71	94.00	25.38	26.23	295.889	87.0	36.54	33.2
60	109.73	42.00	25.84	26.76	311.437	91.6	37.21	33.3
61	109.76	42.00	25.75	26.66	305.839	90.0	37.08	33.1
62	109.78	42.00	25.75	26.72	309.019	90.9	37.08	33.3
63	109.80	42.00	25.93	26.74	309.536	91.0	37.34	33.2
64	109.83	42.00	25.85	26.70	308.444	90.7	37.22	33.2
65	109.85	42.00	26.67	27.60	315.199	92.7	38.40	33.2
66	109.87	42.00	24.93	25.87	298.167	87.7	35.90	33.2
67	109.90	42.00	25.91	26.73	304.888	89.7	37.31	33.2
68	109.92	42.00	25.70	26.54	305.004	89.7	37.00	33.2
69	109.95	42.00	25.58	26.60	303.896	89.4	36.84	33.2
70	109.97	42.00	25.31	26.33	303.064	89.1	36.45	33.3
71	109.99	42.00	25.78	26.58	304.485	89.6	37.12	33.2
72	110.02	42.00	25.98	26.73	306.081	90.0	37.40	33.2
73	110.04	42.00	25.37	26.47	302.579	89.0	36.54	33.2
74	110.07	42.00	25.77	26.55	303.754	89.3	37.11	33.2
75	110.09	42.00	25.76	26.55	307.350	90.4	37.09	33.2
76	110.11	42.00	25.98	26.72	307.690	90.5	37.41	33.2
77	110.14	42.00	25.97	26.80	303.918	89.4	37.40	33.2
78	110.16	42.00	25.68	26.34	305.004	89.7	36.97	33.2
79	110.18	42.00	25.78	26.68	310.177	91.2	37.13	33.2
80	110.21	42.00	25.86	26.78	302.410	88.9	37.24	33.3
		Average	25.23	25.99	300.420	88.4	36.34	32.8
		Maximum	26.67	27.60	315.199	92.7	38.40	33.5
		Minimum	22.04	22.66	270.335	79.5	31.74	1.9

Total number of blows analyzed: 77

BL#	Sensors
1-80	F1: [333NWJ-2] 214.5 (1.00); F2: [333NWJ-2] 201.7 (1.00); A1: [K11483] 416.9 (1.00); A2: [K11485] 436.6 (1.00)

Time Summary

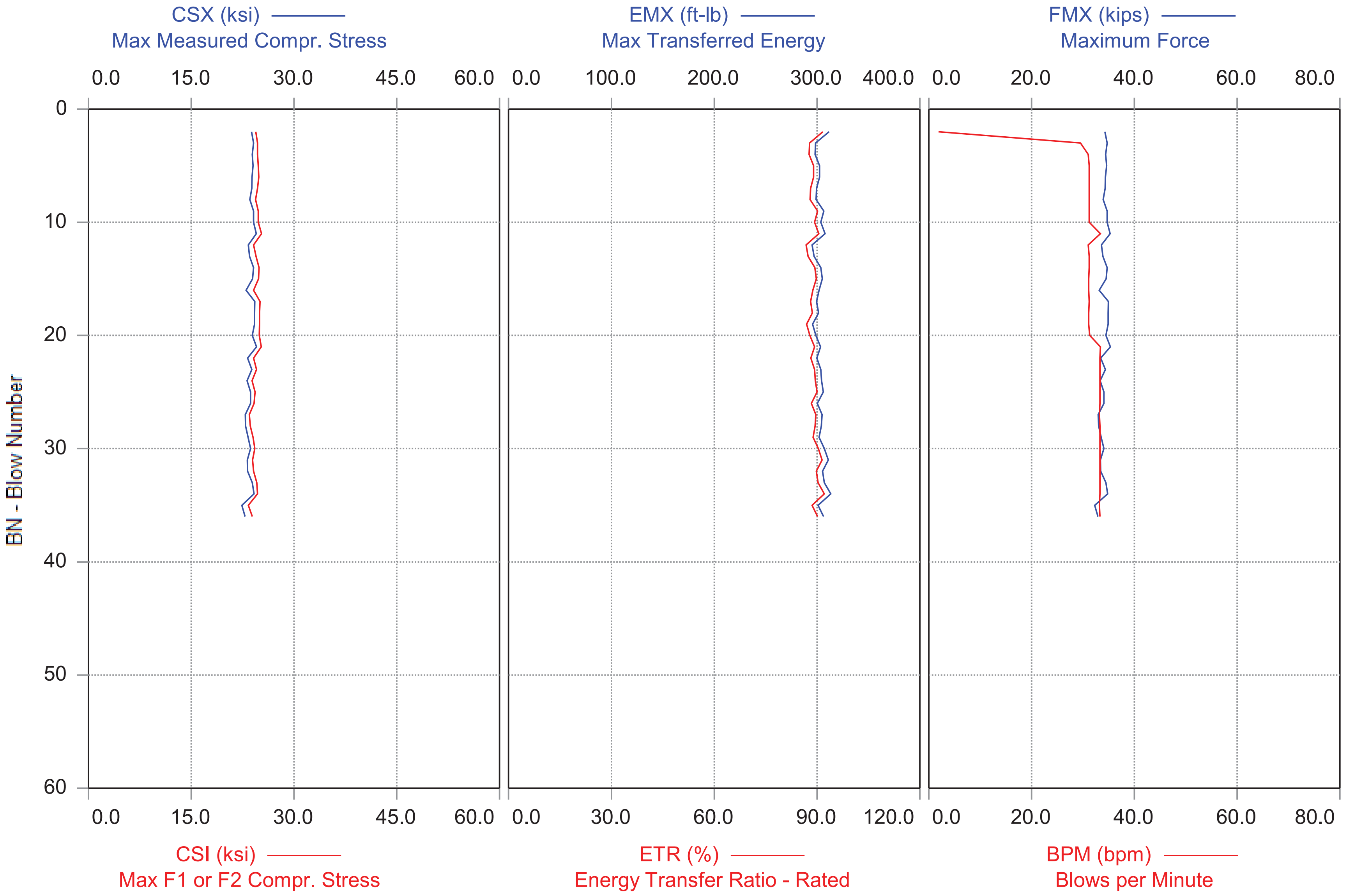
Drive 2 minutes 52 seconds 11:29 AM - 11:32 AM BN 1 - 80

Fugro Consultants, Inc. - PDIPLOT2 Ver 2022.1.62.0 - Case Method & iCAP® Results

Printed: 05-January-2023

Test started: 04-January-2023

VAN AND SONS - DS-4 113.5FT



PDIPLOT2 2022.1.62.0 Printed 05-January-2023

Case Method & iCAP® Results

VAN AND SONS - DS-4 113.5FT A
OP: SW Date: 04-January-2023

AR: 1.44 in² SP: 0.492 k/ft³
LE: 118.50 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

CSX: Max Measured Compr. Stress ETR: Energy Transfer Ratio - Rated
CSI: Max F1 or F2 Compr. Stress FMX: Maximum Force
EMX: Max Transferred Energy BPM: Blows per Minute

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
2	113.61	18.00	23.79	24.44	311.720	91.7	34.26	1.9
3	113.67	18.00	24.10	24.68	298.690	87.8	34.70	29.5
4	113.72	18.00	23.90	24.68	298.087	87.7	34.41	31.0
5	113.78	18.00	24.01	24.80	302.526	89.0	34.58	31.2
6	113.83	18.00	23.88	24.87	302.544	89.0	34.38	31.2
7	113.89	18.00	23.83	24.68	299.662	88.1	34.32	31.2
8	113.94	18.00	23.56	24.41	299.101	88.0	33.92	31.2
9	114.00	18.00	24.10	24.79	306.538	90.2	34.70	31.2
10	114.04	26.00	24.09	24.78	303.568	89.3	34.68	31.2
11	114.08	26.00	24.50	25.27	307.705	90.5	35.29	33.4
12	114.12	26.00	23.32	24.08	295.256	86.8	33.58	31.0
13	114.15	26.00	23.52	24.48	297.142	87.4	33.87	31.2
14	114.19	26.00	24.10	24.90	303.711	89.3	34.70	31.2
15	114.23	26.00	23.96	24.81	305.233	89.8	34.50	31.1
16	114.27	26.00	23.02	24.09	301.956	88.8	33.15	31.1
17	114.31	26.00	24.28	25.06	299.519	88.1	34.96	31.2
18	114.35	26.00	24.23	24.96	301.402	88.6	34.89	31.1
19	114.38	26.00	24.24	24.98	295.694	87.0	34.91	31.1
20	114.42	26.00	23.93	24.94	298.847	87.9	34.45	31.3
21	114.46	26.00	24.54	25.24	303.449	89.2	35.33	33.4
22	114.50	26.00	23.23	24.09	299.778	88.2	33.45	33.3
23	114.54	28.00	23.86	24.55	303.630	89.3	34.35	33.3
24	114.57	28.00	23.14	23.87	304.387	89.5	33.33	33.3
25	114.61	28.00	23.64	24.30	306.083	90.0	34.05	33.3
26	114.64	28.00	23.66	24.17	300.374	88.3	34.07	33.3
27	114.68	28.00	22.89	23.49	304.802	89.6	32.97	33.2
28	114.71	28.00	22.95	23.63	304.084	89.4	33.05	33.3
29	114.75	28.00	23.30	24.03	302.093	88.9	33.55	33.3
30	114.79	28.00	23.67	24.29	307.172	90.3	34.09	33.3
31	114.82	28.00	23.20	23.95	311.017	91.5	33.41	33.3
32	114.86	28.00	23.22	24.11	305.379	89.8	33.43	33.3
33	114.89	28.00	23.92	24.57	307.151	90.3	34.45	33.3
34	114.93	28.00	24.17	24.68	313.310	92.2	34.80	33.3
35	114.96	28.00	22.40	23.33	301.025	88.5	32.26	33.2
36	115.00	28.00	22.85	23.92	306.441	90.1	32.91	33.3
		Average	23.69	24.45	303.116	89.2	34.11	31.3
		Maximum	24.54	25.27	313.310	92.2	35.33	33.4
		Minimum	22.40	23.33	295.256	86.8	32.26	1.9

Total number of blows analyzed: 35

BL# Sensors

2-36 F1: [333NWJ-2] 214.5 (1.00); F2: [333NWJ-2] 201.7 (1.00); A1: [K11483] 416.9 (1.00);
A2: [K11485] 436.6 (1.00)

Time Summary

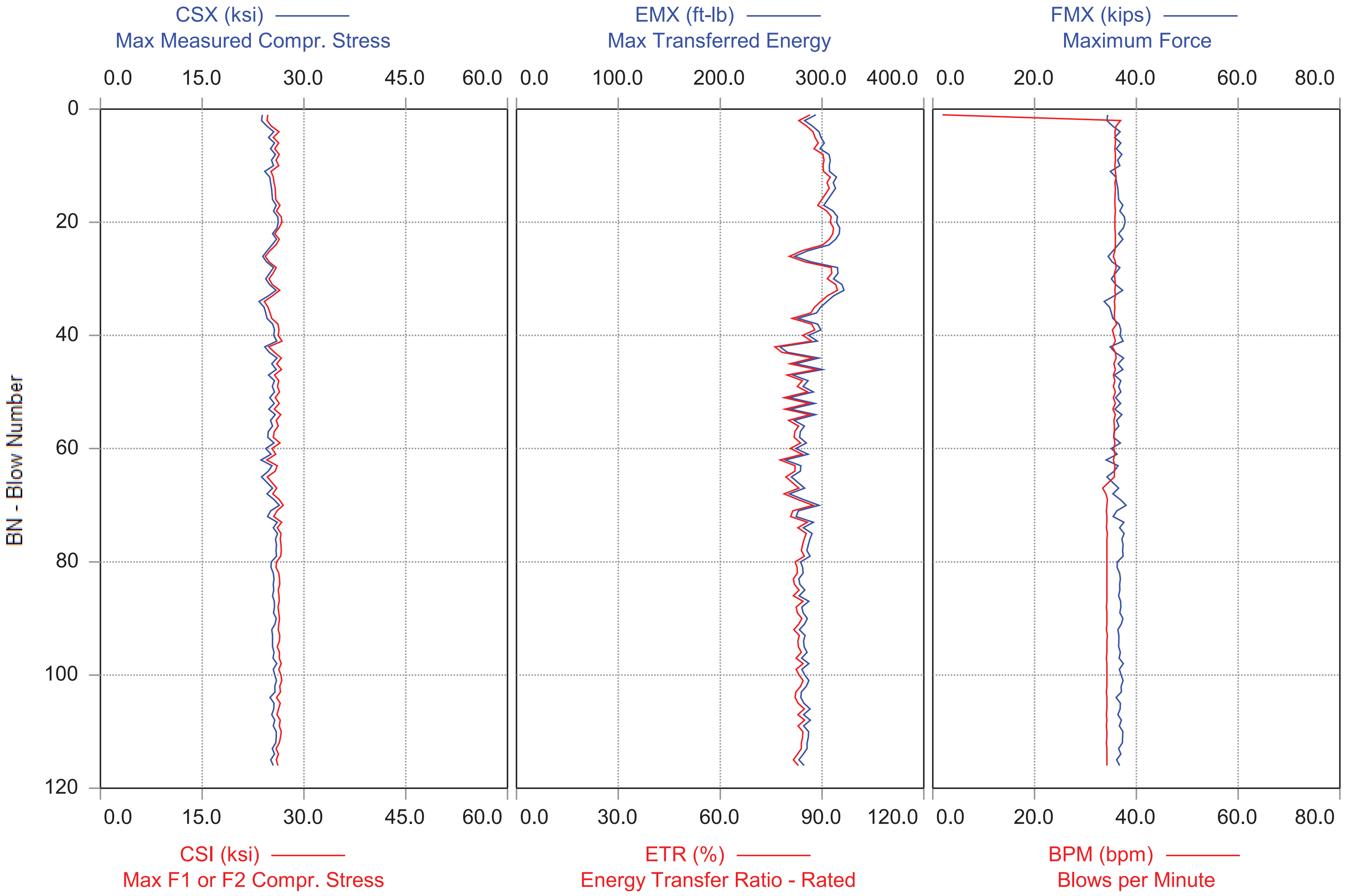
Drive 1 minute 16 seconds 11:58 AM - 12:00 PM BN 2 - 36

Fugro Consultants, Inc. - PDIPLOT2 Ver 2022.1.62.0 - Case Method & iCAP® Results

Printed: 05-January-2023

Test started: 04-January-2023

VAN AND SONS - DS-5 118.83FT



Case Method & iCAP® Results

VAN AND SONS - DS-5 118.83FT A
OP: SW Date: 04-January-2023

AR: 1.44 in² SP: 0.492 k/ft³
LE: 123.50 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio - Rated
FMX: Maximum Force
BPM: Blows per Minute

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
1	118.85	84.00	23.88	24.67	294.057	86.5	34.38	1.9
2	118.86	84.00	23.77	24.57	282.929	83.2	34.23	36.9
3	118.87	84.00	24.52	25.18	290.561	85.5	35.32	36.0
4	118.88	84.00	25.56	26.34	297.050	87.4	36.81	35.8
5	118.89	84.00	24.80	25.47	299.007	87.9	35.71	35.9
6	118.90	84.00	25.68	26.28	302.219	88.9	36.98	35.9
7	118.92	84.00	25.07	25.72	298.145	87.7	36.09	35.8
8	118.93	84.00	25.80	26.36	307.039	90.3	37.15	35.9
9	118.94	84.00	25.22	25.88	308.073	90.6	36.31	35.9
10	118.95	84.00	25.53	26.29	307.081	90.3	36.76	35.8
11	118.96	84.00	24.25	25.14	307.475	90.4	34.92	35.9
12	118.98	84.00	24.98	25.48	314.242	92.4	35.97	36.0
13	118.99	84.00	25.08	25.60	311.185	91.5	36.12	35.8
14	119.00	84.00	25.24	25.76	313.259	92.1	36.35	35.9
16	119.00	1,319.02	25.35	25.84	305.781	89.9	36.51	35.8
17	119.00	1,319.02	25.93	26.46	301.887	88.8	37.35	35.8
18	119.00	1,319.02	25.53	25.95	310.835	91.4	36.76	35.9
19	119.00	1,319.02	26.13	26.64	315.317	92.7	37.63	35.8
20	119.00	1,319.02	26.23	26.77	314.315	92.4	37.77	35.8
21	119.01	1,319.02	26.02	26.32	317.461	93.4	37.47	35.8
22	119.01	1,319.02	25.36	25.70	316.909	93.2	36.52	35.9
23	119.01	1,319.02	25.95	26.38	313.528	92.2	37.36	35.9
24	119.01	1,319.02	25.30	25.88	306.814	90.2	36.44	35.9
25	119.01	1,319.02	24.62	25.00	286.417	84.2	35.45	35.8
26	119.01	1,319.02	23.94	24.30	273.222	80.4	34.47	35.5
27	119.01	1,319.02	24.49	24.87	289.106	85.0	35.26	35.9
28	119.01	1,319.02	25.54	25.92	315.224	92.7	36.77	36.0
29	119.01	1,319.02	24.92	25.45	315.783	92.9	35.88	35.7
30	119.01	1,319.02	24.38	24.85	311.234	91.5	35.10	35.8
31	119.01	1,319.02	24.94	25.34	319.712	94.0	35.92	35.9
32	119.01	1,319.02	25.90	26.45	321.532	94.6	37.30	35.8
33	119.01	1,319.02	24.78	25.38	311.664	91.7	35.69	35.9
34	119.02	1,319.02	23.40	24.17	304.902	89.7	33.69	35.7
35	119.02	1,319.02	24.12	24.72	298.500	87.8	34.73	35.8
36	119.02	1,319.02	24.38	25.00	294.763	86.7	35.11	35.7
37	119.02	1,319.02	24.57	25.27	276.196	81.2	35.38	35.7
38	119.02	1,319.02	25.42	26.18	295.859	87.0	36.60	36.1
39	119.02	1,319.02	25.66	26.33	298.784	87.9	36.95	35.3
40	119.02	1,319.02	25.56	26.24	286.831	84.4	36.80	35.6
41	119.02	1,319.02	25.98	26.78	295.508	86.9	37.41	35.9
42	119.02	1,319.02	24.21	24.84	258.718	76.1	34.87	35.3
43	119.02	1,319.02	24.89	25.67	265.944	78.2	35.84	35.9
44	119.02	1,319.02	26.04	26.67	296.977	87.3	37.49	36.0
45	119.02	1,319.02	25.27	26.01	273.949	80.6	36.39	35.6
46	119.02	1,319.02	25.94	26.71	300.467	88.4	37.35	35.9
47	119.03	1,319.02	24.77	25.65	271.056	79.7	35.67	35.5
48	119.03	1,319.02	25.66	26.29	286.419	84.2	36.95	35.8

Case Method & iCAP® Results

VAN AND SONS - DS-5 118.83FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
49	119.03	1,319.02	25.35	26.11	281.339	82.7	36.51	35.5
50	119.03	1,319.02	25.67	26.42	291.550	85.8	36.96	35.9
51	119.03	1,319.02	24.95	25.78	267.970	78.8	35.93	35.5
52	119.03	1,319.02	25.63	26.38	292.932	86.2	36.91	35.9
53	119.03	1,319.02	24.84	25.63	268.931	79.1	35.76	35.4
54	119.03	1,319.02	25.78	26.59	293.725	86.4	37.13	35.9
55	119.03	1,319.02	25.09	25.93	272.650	80.2	36.13	35.6
56	119.03	1,319.02	25.38	26.21	282.687	83.1	36.55	35.7
57	119.03	1,319.02	24.73	25.64	278.788	82.0	35.62	35.7
58	119.03	1,319.02	24.66	25.47	278.122	81.8	35.51	35.6
59	119.03	1,319.02	25.60	26.47	284.539	83.7	36.87	35.8
60	119.03	1,319.02	24.37	25.30	274.458	80.7	35.09	35.6
61	119.04	1,319.02	25.14	25.86	286.275	84.2	36.20	35.8
62	119.04	1,319.02	23.67	24.52	264.131	77.7	34.08	35.5
63	119.04	1,319.02	25.29	26.07	279.149	82.1	36.42	35.8
64	119.04	1,319.02	24.75	25.76	278.796	82.0	35.65	35.7
65	119.04	1,319.02	23.75	24.60	269.876	79.4	34.21	35.7
67	119.04	4,896.31	25.37	26.00	282.825	83.2	36.54	33.4
68	119.04	4,896.31	24.57	25.43	268.077	78.8	35.38	34.0
69	119.04	4,896.31	25.61	26.36	281.998	82.9	36.87	34.3
70	119.04	4,896.31	26.37	26.96	297.038	87.4	37.97	34.2
71	119.04	4,896.31	25.12	26.03	276.791	81.4	36.18	34.1
72	119.04	4,896.31	24.62	25.57	274.596	80.8	35.45	34.2
73	119.04	4,896.31	26.07	26.71	291.421	85.7	37.54	34.2
74	119.04	4,896.31	25.50	26.08	281.761	82.9	36.71	34.1
75	119.04	4,896.31	26.10	26.61	290.280	85.4	37.58	34.3
76	119.04	4,896.31	25.80	26.51	288.042	84.7	37.15	34.2
77	119.04	4,896.31	25.97	26.61	286.581	84.3	37.40	34.2
78	119.04	4,896.31	25.89	26.65	285.230	83.9	37.28	34.2
79	119.04	4,896.31	25.98	26.58	288.283	84.8	37.41	34.2
80	119.04	4,896.31	25.18	25.95	278.975	82.1	36.26	34.2
81	119.04	4,896.31	25.15	25.89	281.023	82.7	36.22	34.2
82	119.04	4,896.31	25.49	26.27	281.434	82.8	36.71	34.2
83	119.04	4,896.31	25.61	26.40	277.214	81.5	36.87	34.2
84	119.04	4,896.31	25.49	26.42	278.214	81.8	36.70	34.2
85	119.04	4,896.31	25.54	26.22	283.187	83.3	36.78	34.2
86	119.04	4,896.31	25.37	26.34	277.452	81.6	36.53	34.2
87	119.04	4,896.31	25.61	26.36	287.090	84.4	36.88	34.2
88	119.04	4,896.31	25.65	26.19	280.111	82.4	36.94	34.1
89	119.04	4,896.31	25.52	26.27	281.388	82.8	36.74	34.2
90	119.04	4,896.31	25.93	26.40	285.712	84.0	37.34	34.2
91	119.04	4,896.31	25.73	26.30	282.894	83.2	37.05	34.2
92	119.04	4,896.31	25.25	26.23	277.801	81.7	36.36	34.1
93	119.04	4,896.31	25.38	26.42	283.452	83.4	36.55	34.3
94	119.04	4,896.31	25.38	26.40	281.940	82.9	36.55	34.2
95	119.04	4,896.31	25.36	26.08	282.383	83.1	36.52	34.2
96	119.04	4,896.31	25.59	26.41	285.373	83.9	36.85	34.2
97	119.05	4,896.31	25.44	26.36	280.084	82.4	36.63	34.1
98	119.05	4,896.31	25.99	26.67	287.113	84.4	37.43	34.2
99	119.05	4,896.31	25.47	26.29	280.138	82.4	36.68	34.2
100	119.05	4,896.31	25.71	26.64	283.185	83.3	37.03	34.2
101	119.05	4,896.31	25.96	26.74	287.057	84.4	37.38	34.2
102	119.05	4,896.31	25.69	26.42	284.661	83.7	36.99	34.2

Case Method & iCAP® Results

VAN AND SONS - DS-5 118.83FT A
OP: SW Date: 04-January-2023

BL#	Depth ft	BLC bl/ft	CSX ksi	CSI ksi	EMX ft-lb	ETR (%)	FMX kips	BPM bpm
103	119.05	4,896.31	25.74	26.54	279.839	82.3	37.06	34.1
104	119.05	4,896.31	25.01	25.97	279.180	82.1	36.01	34.2
105	119.05	4,896.31	25.61	26.52	282.174	83.0	36.87	34.2
106	119.05	4,896.31	25.57	26.26	288.354	84.8	36.82	34.2
107	119.05	4,896.31	25.27	26.04	281.880	82.9	36.39	34.1
108	119.05	4,896.31	25.72	26.52	288.505	84.9	37.03	34.2
109	119.05	4,896.31	25.47	26.35	281.980	82.9	36.68	34.1
110	119.05	4,896.31	25.93	26.65	286.865	84.4	37.34	34.2
111	119.05	4,896.31	25.91	26.54	286.580	84.3	37.31	34.2
112	119.05	4,896.31	25.86	26.37	285.391	83.9	37.24	34.1
113	119.05	4,896.31	25.35	25.89	285.389	83.9	36.50	34.2
114	119.05	4,896.31	25.66	26.24	281.623	82.8	36.94	34.2
115	119.05	4,896.31	25.10	25.92	277.310	81.6	36.14	34.2
116	119.05	4,896.31	25.50	26.19	282.341	83.0	36.72	34.2
		Average	25.29	25.99	289.281	85.1	36.42	34.8
		Maximum	26.37	26.96	321.532	94.6	37.97	36.9
		Minimum	23.40	24.17	258.718	76.1	33.69	1.9

Total number of blows analyzed: 114

BL#	Sensors
1-116	F1: [333NWJ-2] 214.5 (1.00); F2: [333NWJ-2] 201.7 (1.00); A1: [K11483] 416.9 (1.00); A2: [K11485] 436.6 (1.00)

Time Summary

Drive 4 minutes 44 seconds 12:30 PM - 12:34 PM BN 1 - 116

Appendix B-3 Material calculation package

Confidential and Proprietary Business Information of Black & Veatch

Client Name SJRA
Page 1 of 67
Project Name Spring Creek Watershed (SCW) Flood Control Dams
Project No. 411500
Calculation Title Evaluation of Project Soil Parameters
Verification Method: ☒ Check and Review ☐ Alternate Calculations

Objective: Evaluate the soil parameters that will be used for seepage and stability analysis of the Project

Unverified Assumptions Requiring Subsequent Verification

No.	Assumption	Verified By*	Date

Refer to Page _____ of this calculation for additional assumptions.

This Section Used for Software-Generated Calculations

BV Standard Application
Program Name/Version Microsoft Excel

Review and Approval

Rev	Prepared By*	Date	Checked By*	Date	Approved By*	Date
0	P. Turkson, PhD, P.E.	10/25/2024	David Bentler, PhD, P.E.	12/6/2024		

*Signature required.

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1.0 Objective

Evaluate the design soil parameters that will be used for design of the Walnut Creek Dam and Birch Creek Dam as part of the Spring Creek Watershed Flood Control Dams scope of work for SJRA.

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3.0 Design Basis

The following section provides the design methodology for estimating the subsurface profile and selection of soil parameters for the Spring Creek Flood Control Dams which comprise Walnut Creek Dam and Birch Creek Dam (hereafter referred to as the Project). Based on the provided soil borehole logs and laboratory testing results the design stratigraphy was developed, and the required soil properties were selected for use in seepage and static stability analysis of the Project. Additionally, the stratigraphy and soil properties have been developed with guidance from published literature on geological units with similar characteristics.

A design stratigraphy was developed for the Project that includes the following two dams:

- Walnut Creek Dam (39.1 feet high, bottom of dam elevation 224.5 feet to top of crest elevation 263.6 feet).
- Birch Creek Dam (35.4 feet high, bottom of dam elevation 223.7 feet to top of crest elevation 259.1 feet).

The dams are primarily differentiated by the location of each creek. Each dam section is subdivided into strata based on material classification(s), index properties, and strengths. Dam section differentiation is only applied to **Section 4.0** for design strata elevations or depths.

Design soil parameters were evaluated for each stratum defined in the design stratigraphy based on available field and laboratory data. Where available, laboratory testing was used preferentially, and correlations with field testing and/or estimates from the published literature were used where minimal or no laboratory testing was available.

In situ testing for soil consistency and soil strength properties were performed in the four drilled boreholes. Borings B-1 and B-2 were drilled for the Walnut Creek Dam and borings B-3 and B-4 were drilled for the Birch Creek Dam. The Standard Penetration Test (SPT), which primarily targeted granular soils was used to estimate soil density as well as clayey soil consistency. Hand penetrometer (PP) testing was used to evaluate field cohesive soil strength.

Table 1 provides a summary of borehole locations and associated type of borehole in situ testing performed.

Laboratory testing from the 2024 geotechnical explorations was assigned by Aviles Engineering (Aviles) and reviewed by Black & Veatch. Laboratory testing was completed by Aviles. The laboratory testing results and a final geotechnical investigations report applied in this analysis are provided in **Appendix A** of the Design Basis Memorandum (DBM) (**Reference 17**). If new or revised laboratory data are provided that supersede the data applied in this calculation package, the design soil parameter calculations must be reviewed to confirm that the changes do not invalidate the design soil parameters.

Table 1 **Borehole Location Summary for the Project**

Borehole ID	In Situ Testing Type[2,3,4]		UTM Coordiantes[1] (m)		Surface Elevation (feet)[5]	Borehole Depth (feet)
			Northing	Easting		
	SPT	PP				
B-1	Y	Y	30°11'14.68"N	95°49'49.60"W	250	90
B-2	Y	Y	30°11'20.29"N	95°49'32.18"W	230	120
B-3	Y	Y	30°11'22.52"N	95°49'17.42"W	230	120
B-4	Y	Y	30°11'22.57"N	95°49'6.42"W	245	90

(1) Coordinates are in the UTM Zone 15, WGS 84 coordinate system.
(2) SPT— Standard Penetration Test
(3) PP— Pocket Penetrometer
(4) Y—Yes
(5) Surface elevations are approximate and obtained from Google Earth

3.1 Soil Stratigraphy

The borings drilled around the Project generally encountered soft materials, no rock was encountered. The borings generally encountered alternating layers of silty sands (SM), sandy lean clays (CL), clayey sands (SC), poorly graded sand with silt (SP-SM), sandy fat clay (CH), silty clay with sand (CL-ML) and silty clayey sand (SC-SM).

Based on the results of the geotechnical explorations, the stratigraphic units (from surface to depth) encountered are summarized in **Table 2**. The following subsections in **Section 4.0** provide a general description of the soil stratigraphy encountered.

A longitudinal section of the general stratigraphy of the soil units is shown in **Attachment 1**. Lines designating the interfaces between various strata on the boring logs represent approximate boundaries and the transition between strata. Soil conditions will vary between boring locations.

Table 2 **Stratigraphic Units Encountered During Drilling**

Unit No.	Description (USCS Classification Symbol)
1	Silty Sand (SM)
2	Lean Clay (CL)
3	Clayey Sand (SC)
4	Sand with Silt (SP-SM)
5	Fat Clay (CH)
6	Sand (SP)
7	Silty Clayey Sand (SC-SM)

The stratigraphic soil unit thicknesses encountered in the boreholes are summarized in **Table 3**Error! Reference source not found.. The thicknesses provided are based on interpretations of the boring logs, and in some cases relatively thin interbedded layers have been combined with the predominant layer as one unit. Appearance of the different units in each drilled hole are not necessarily in the order listed in the summary table. The detailed description of soils as recorded on the boring logs can be found in **Appendix A** in the DBM.

Table 3 **Summary of Stratigraphic Units Encountered in Boreholes**

Borehole ID	Subsurface Unit Thickness (feet)						
	Unit 1	Unit 2	Unit 3	Unit 4	Unit 5	Unit 6	Unit 7
B-1	18	18	18	15	15	—	—
B-2	36	46	9	23	5	—	—
B-3	13	15	47	26	10	—	—
B-4	—	—	17	40	5	13	14

3.2 Groundwater

Groundwater was encountered in all the borings. Groundwater was encountered during drilling in borings B-2, B-3 and B-4 at depths ranging from 8 to 28 feet below ground surface (fBGS). Groundwater in B-1 was only encountered after completion of drilling at a depth of 10.2 fBGS. The groundwater level in the borings 15 mins after completion of drilling was noted at depths ranging from 5.5 to 26.5 feet. A summary of the static groundwater depths by borehole is presented in **Table 4**.

Table 4 **Groundwater Levels**

Borehole ID	Water Level (fBGS)	
	During Drilling	15 mins After Drilling Completion
B-1[1]	Not Encountered	Not measured
B-2	12	5.5
B-3	8	5.8
B-4	28	26.5

1. Groundwater in B-1 encountered at 10.2 fbgs after drilling, no time of measurement recorded.

3.3 SPT N Values

To determine design N values for the soil profile, blow count information from the boreholes (B-1 to B-4) were analyzed. The field-measured blow counts (SPT N-value) are corrected to an equivalent N_{60}, by the following equation (**Reference 1**).

$$N_{60} = \frac{E_m C_b C_s C_R N}{0.60}$$

where, E_m = hammer efficiency
C_b = borehole diameter correction factor
C_s = sampler correction
C_R = rod length correction
N = measured N value

The reported hammer efficiency correction factor ($E_m/60\%$) based on energy data from **Reference 2** is 1.46 for the determination of N_{60} values.

For boreholes between 2.5 to 4.5 inches in diameter, the borehole diameter correction factor is 1.0. Based on the geotechnical report, the inside diameter of the hollow stem auger used for drilling is 4 inches; therefore, a correction factor of 1 is used for the correction. Since a standard sampler was used for the penetration tests, a sampler correction factor of 1 is used.

The rod length (C_R) further modifies this calculation based on the sample depth. The following correction factors are used (**Reference 3**):

- For samples less than 13 feet below grade, a C_R of 0.75.
- For samples between 13 and 20 feet below grade, a C_R of 0.85.

- For samples between 20 and 30 feet below grade, a C_R of 0.95.
- For samples over 30 feet below grade, a C_R of 1.0.

N_{60} values for the proposed design soil profile were calculated as the product of SPT N values and the conversion factors. Design values are taken as approximate average values for the layer. Note that raw N values of 50 were not corrected.

3.4 Index Properties

Design values for total unit weights, liquid limits, plasticity index, and moisture contents for each stratum were selected as the statistical mean value within each stratum. Laboratory index testing results were assigned to the corresponding strata based on sample depth and material type recorded on the boring log.

Unit weights were typically assigned using available laboratory data where available. In strata where laboratory testing was unavailable, published literature of typical values for the soil type from **Reference 18** and **Reference 20** were used to estimate unit weight.

3.5 Soil Strength Parameters

The following section describes the design basis of determining shear strength parameters.

3.5.1 Q-Case

For the Q-Case, or undrained case, undrained shear strength (s_u) design values were evaluated for each fine-grained stratum. Values of s_u within each stratum were evaluated based on Unconfined Compression (UC) tests and Unconsolidated-Undrained (UU) tests; s_u estimates from pocket penetrometer (PP) and torvane (TV) shear tests which are typically high were not considered. The PP and TV test methods are best used as a quick field assessment for soft clays, and the results of the tests are not reliable for clays with sand or silt, or for detailed design. Published correlations between SPT N values and s_u were used to derive undrained shear strengths for comparison to s_u from laboratory tests.

After grouping all the data points for each stratum, statistical evaluation of s_u included the following assessments:

1. Average or mean.
2. Sample standard deviation.
3. 33rd percentile (1/3 rule).
4. 95 percent confidence interval of the mean.

Reference 4 provides guidance on methodology to use for the design strength based on the number of samples (*n*) within each stratum:

- For *n* ≤ 10 samples, the lower 95 percent confidence limit (CL) is the design strength.
- For *n* > 10 samples, the 33rd percentile (1/3 rule) is the design strength.

The 95 percent confidence interval of the mean was calculated as follows:

$$< \mu >_{1-\alpha} = \left[\bar{X} + t_{n-1}\left(\frac{\alpha}{2}\right) * \frac{s}{\sqrt{n}} \; ; \; \bar{X} - t_{n-1}\left(\frac{\alpha}{2}\right) * \frac{s}{\sqrt{n}}\right] \quad \text{(Equation 1)}$$

Where:

$< \mu >_{1-\alpha}$ = interval for the mean with α = 5 percent (confidence interval of 95 percent)

$\bar{X}$ = sample mean

$t_{n-1}\left(\frac{\alpha}{2}\right)$ = T-Score evaluated at $\frac{\alpha}{2}$ for n-1 degrees of freedom

s = sample standard deviation

n = sample size

Equation 1 is based on a T-Distribution, which considers the greater uncertainty associated with small samples sizes less than or equal to thirty (30). As n increases (>30), the T-Score approaches an equivalent value to a Z-Score from a normal distribution.

3.5.2 S-Case

For the S-Case, or drained case, design values for effective friction angle (ϕ’) were evaluated for each stratum based on Consolidated-Undrained (CU) triaxial testing, correlations with SPT N-values, and/or based on guidance from published literature where laboratory data estimates are not available.

The value of ϕ’ used to develop shear strength envelopes from CU tests is based on the following relationship, where the value for α is measured from stress paths plotted on the CU lab data sheets:

$$tan\ (\alpha) = \sin\ (\phi') \qquad \text{(Equation 2)}$$

The design envelope is the average of the CU tests strength envelopes within each stratum. Where lab testing is not available, correlations from **Reference 5** based on plasticity index (*PI*) were considered for comparison purposes only.

In addition, design values for the effective cohesion intercept (*c’*) for soil strata which are considered fine-grained were evaluated based on CU triaxial testing and with guidance from published literature. The value of *c’* used to develop shear strength envelopes from CU tests is based on the following relationship, where the value for *d’* is measured from stress paths plotted on the CU lab data sheets:

$$c' = \frac{d\prime}{\cos\ (\phi')} \qquad \text{(Equation 3)}$$

The *c’* for all strata, which are considered free-draining or coarse-grained, is assumed to be 0 pounds per square foot (psf).

For coarse-grained strata, correlations to SPT N-values were used, where SPT testing was available. Correlations from **References 6**, **7**, and **8** were used to select the design value of ϕ’ for coarse-grained strata. When using correlations for SPT N-values, ϕ’ was capped at 38 degrees. In cases where no laboratory or SPT testing were available, ϕ’ was assigned based on typical values from published literature.

The correlation equations are presented as follows:

Fine-grained strata correlations:

Sorensen & Okkels (2013) **(Reference 5)**: $For\ 4 < PI < 50, \phi' = 44 - 14\log_{10} PI$ (Equation 4)
$For\ 50 \leq PI < 150, \phi' = 30 - 6\log_{10} PI$ (Equation 5)

Coarse-grained strata correlations:

Schmertmann (1975) (**Reference 6**): $\phi' = \tan^{-1}\left[\left(\frac{N_{60}}{12.2+\left(20.3\frac{\sigma'_v}{p_a}\right)}\right)^{0.34}\right]$ (Equation 6)

Hatanaka and Uchida (1996) (**Reference 7**): $\phi' = \sqrt{18N_{1,60}} + 20$ (Equation 7)

Peck, Hanson, & Thornburn (1974) (**Reference 8**): $\phi' = 27.1 + 0.3N_{1,60} - 0.00054\left(N_{1,60}\right)^2$ (Equation 8)

Where:

ϕ' = effective stress friction angle

PI = Plasticity index

σ'_v = effective in-situ stress (calculation assumes total unit weight = 120 pcf)

p_a = atmospheric pressure

N_{60} = SPT N-value corrected for field procedures and apparatus to 60% of the theoretical free-fall hammer efficiency

$N_{1,60}$ = SPT N_{60}-value corrected for overburden pressure

3.5.3 R-Case

For the R-Case, which is primarily used for rapid drawdown slope stability analyses, design values for cohesion intercept (c_R) and friction angle (ϕ_R) were developed for fine-grained strata based on the Duncan, Wright, and Wong (1990) procedure (**Reference 9**) detailed in Appendix G of EM-1110-2-1902 (**Reference 10**). These parameters were developed from CU testing. The value of ϕ' used to develop shear strength envelopes in the R-Case is the same ϕ' calculated for the S-Case, based on measurement of α from the CU test data sheets and Equation 2. The design value for c_R is calculated based on results from the CU tests. No more than one CU test was available for the sandy stratum, so single test is the basis of the developed design envelope for the sandy stratum.

3.6 Hydraulic Conductivity

3.6.1 Vertical Hydraulic Conductivity

The design vertical hydraulic conductivities (k_v) were developed for each stratum based on laboratory permeability testing. In strata with more than one permeability test, the geometric mean of the test results is the design value. The geometric mean is appropriate when determining a central value for datasets where the range of values spans multiple orders of magnitude (**Reference 11**). In strata with one permeability test, the test result is the design value. For strata with no testing, either (1) design values from permeability tests within similar materials were used as the design value or (2) typical values within similar materials from published literature were used as the design value. Hydraulic conductivity values generally vary over two orders of magnitude, hence a range of hydraulic conductivity values higher and lower than the design values by one or two orders of magnitude are provided for each stratum.

3.6.2 Anisotropic Ratio

Anisotropic ratio (k_v/k_h) for all strata is assumed based on typical values from values based on typical values from Table 6-6 for natural soils and Table 6-7 for engineered fill (**Reference 12**). Anisotropic ratio of 0.5 was assumed for all foundation stratum. Anisotropic ratio of 0.11 and 0.25 was assumed for clayey stratum and sandy stratum respectively when used as compacted fill. Lower values of anisotropic ratio have been assumed for embankment compacted fill since the embankment will be compacted and placed in horizontal lifts. As such, the vertical hydraulic conductivity will be further reduced than in the horizontal direction. Anisotropic ratio of 1 and 0.25 was assumed for rock riprap and filter sand respectively.

3.7 Consolidation Parameters

The following consolidation parameters were developed as a part of this study and are discussed in detail in the next subsections:

- Initial Void Ratio (e_0).
- Virgin Compression Index (C_c).
- Recompression Index (C_r).
- Overconsolidation Ratio (OCR).
- Coefficient of Consolidation (C_v).

- Coefficient of Secondary Compression (C_α).

3.7.1 Initial Void Ratio (e_0)

The design Initial Void Ratio (e_0) for fine-grained strata were selected based on the average initial void ratios from the available advanced laboratory testing for each stratum.

3.7.2 Virgin Compression Index (C_c), Recompression Index (C_r), and Overconsolidation Ratio (OCR)

The design C_c, C_r, and OCR are calculated for fine-grained strata from laboratory consolidation testing, where available, using the Casagrande Method (**Reference 13**). Laboratory test results were assigned to the corresponding strata based on sample depth and soil type. In strata where laboratory testing was not available, nearby testing results are used if the soil type and index properties are similar.

If consolidation testing was not available or index properties varied significantly from strata with available testing, design values for C_c were evaluated using three correlations with index properties from EM 1110-1-1904 (**Reference 13**), presented below:

1. Void Ratio (e_0): $C_c = 1.15 * (e_0 - 0.35)$ (Equation 8)
2. Moisture Content (MC): $C_c = 0.012 * (MC)$ (Equation 9)
3. Liquid Limit (LL): $C_c = 0.01 * (LL - 13)$ (Equation 10)

The design value (e_0, LL, MC) for the given stratum was inputted into each of the above equations, and the results were compared with laboratory testing, where available. It was noted that the correlation utilizing the void ratio (Equation 8) yielded results that significantly differed from the correlations presented in Equation 9 and 10; therefore, this correlation was not considered for the Project. Once a design value for C_c is selected, the design value of C_r is taken as 1/5 of C_c (**Reference 13**).

3.7.3 Coefficient of Consolidation (c_v)

The design Coefficient of Consolidation (c_v) values were selected for fine-grained strata using the geometric mean of laboratory consolidation testing or the chart from Figure 3-18 of NAVFAC DM 7.1 (**Reference 14**), relating LL to c_v (shown below), where laboratory testing was not available. The LL design value for each stratum and the curve for normally consolidated soils were used to estimate c_v. The selection of the curve for normally consolidated soils is based on the conservative assumption that embankment loading will bring the soils into the virgin compression range of stresses.

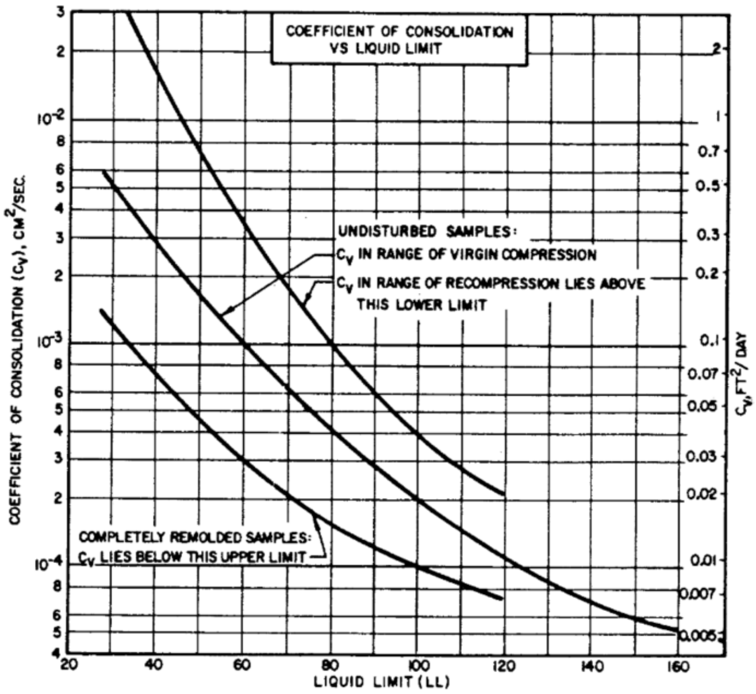

Figure 3-18. Correlations between coefficient of consolidation and liquid limit (NAVFAC DM 7.1)

3.7.4 Coefficient of Secondary Compression (C_α)

The design Coefficient of Secondary Compression (C_α) for the clays were developed from a correlation between C_α/C_C based on soil type from EM 1110-1-1904 (**Reference 13**) and a correlation between moisture content (*MC*) and C_α from the NAVFAC DM 7.1 (**Reference 14**). The C_α design value is based on the midpoint of the C_α/C_C range for corresponding soil types.

Typical ranges of C_α/C_C values are from Table 3-14 in EM 1110-1-1904, shown below (**Reference 13**). A C_α value is developed by multiplying the design C_C value for each stratum with the lower and upper bounds for the corresponding soil type from Table 3-14.

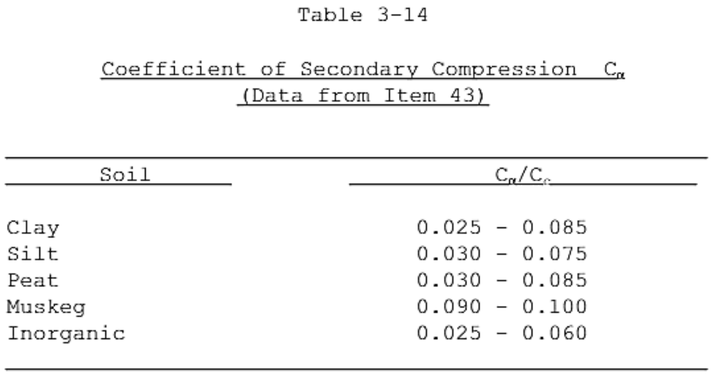

Table 3-14

Coefficient of Secondary Compression C_α
(Data from Item 43)

Soil	C_α/C_c
Clay	0.025 - 0.085
Silt	0.030 - 0.075
Peat	0.030 - 0.085
Muskeg	0.090 - 0.100
Inorganic	0.025 - 0.060

3.7.5 Modulus of Elasticity (E_S)

Design Modulus of Elasticity (E_s) were selected for granular soils using a combination of correlations with SPT N-values, and typical ranges of values based on soil type from Table D-3 in EM 1110-1-1904 (**Reference 13**), depending on the availability of in-situ data. Six(6) correlations with SPT N-Values were selected based on Black & Veatch calculation template (**Reference 15**), and the results averaged. It should be noted that range of strain for estimates of E_s from in-situ tests (CPT and SPT) is on the order of 0.1-1%, resulting in a conservative estimate. Modulus values may need to be scaled to match the appropriate range of strain obtained from deformation analyses (**Reference 16**).

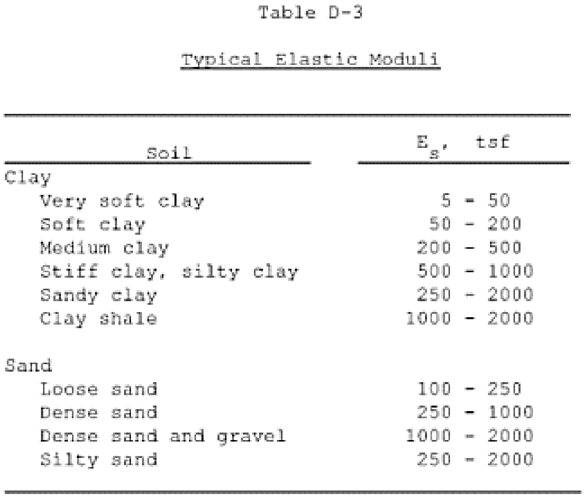

Table D-3

Typical Elastic Moduli

Soil	E_s, tsf
Clay	
Very soft clay	5 - 50
Soft clay	50 - 200
Medium clay	200 - 500
Stiff clay, silty clay	500 - 1000
Sandy clay	250 - 2000
Clay shale	1000 - 2000
Sand	
Loose sand	100 - 250
Dense sand	250 - 1000
Dense sand and gravel	1000 - 2000
Silty sand	250 - 2000

4.0 Analysis

The equations and methods presented in **Section 3.0** of this calculation package were used to estimate the design soil parameters for the Project. The laboratory testing results are provided in **Appendix A** of the DBM. (**Reference 17**)

Subsurface profiles illustrating the subsurface conditions have been developed from the information provided by the geotechnical subsurface exploration (**Appendix A** of the DBM). The stratum boundaries were defined based on material classification(s), index properties, and undrained shear strength. **Section 4.1** describes the design stratigraphy for the Project.

4.1 Subsurface Conditions and Profile

4.1.1 Unit 1 — Silty Sand (SM)

Silty sand soils (Unit 1) ranging in density from loose to dense and consisting of pockets lean clay was encountered at all borehole locations except B-4. Unit 1 varied in color from tan to brown and extended to depths ranging from about 1.25 to 16 fBGS. Unit 1 was also observed at deeper depths below ground surface from 27 to 112 fBGS. The thickness of Unit 1 was observed to range from 3 to 15 feet.

4.1.2 Unit 2 — Lean Clay (CL)

Deposits of very soft to hard sandy lean clay (Unit 2) were encountered in all boreholes except B-4 at depths ranging from 4 to 18 fBGS, and at deeper depths from 32 to 97 feet. The thicknesses of Unit 2 were recorded as ranging from 2 to 24 feet.

4.1.3 Unit 3 — Clayey Sand (SC)

Deposits of the clayey sand (SC) were encountered in all boreholes at depths ranging from 1 to 27 fBGS and at deeper depths ranging from 38 to 112 fBGS, with thickness ranging from 4 to 23 feet. In some of the boreholes, deposits of ferrous nodules and pockets of lean clay were recorded. The density of Unit 3 was recorded as ranging from very loose to medium dense.

4.1.4 Unit 4 — Sand with Silt (SP-SM)

Deposits of sand with silt (Unit 4) were encountered in all the boreholes and were interbedded with pockets of lean clay at various depths. Unit 4 varied in thickness between 5 to 25 feet. Unit 4 was recorded at depths from 22 to 47 fBGS and at deeper depths between 62 to 120 fBGS. The density of Unit 4 was recorded as ranging from loose to very dense.

4.1.5 Unit 5 — Fat Clay (CH)

Deposits of clay (Unit 5) were encountered in all boreholes at depths ranging from 32 to 67 fBGS, with thicknesses ranging from 5 to 15 feet. The clay till unit also consisted of pockets of sand and silt seams, and calcareous and ferrous nodules. The consistency of the clay unit was recorded as firm to hard.

4.1.6 Unit 6 — Sand (SP)

Deposits of sand (Unit 6) were observed in only borehole B-4 at a depth of 77 fBGS and thickness of 13 feet. The sand density was recorded as dense to very dense.

4.1.7 Unit 7 — Silty Clayey Sand (SC-SM)

Similar to Unit 6, deposits of the silty clayey sand (Unit 7) were observed in only borehole B-4 but at a relatively shallow depth of 8 fBGS and thickness of 14 feet. The silty clayey sand density was recorded as medium dense to dense.

4.1.8 Design Stratigraphy

A summary of the design stratigraphic units and their design thicknesses is presented in **Table 5** and **Table 6** for Walnut Creek Dam and Birch Creek Dam respectively. The clay units (Unit 2 and Unit 5) have been modeled as a single unit with similar material properties, hereafter referred to as Silty Clay and Sandy Clay unit. The general appearance of the single clay unit has been modeled at depths from 32 to 70 fBGS and from 77 to 97 fBGS for Walnuat Creek Dam. The single clay unit for Birch Creek Dam has been modeled as sandwiched (from 32 to 62 fBGS) between the sandy units (Unit 1, Unit 3, Unit 4, Unit 6 and Unit 7) which have been modeled as a single unit for seepage analysis and slope stability design with the same material properties. The single sandy unit is hereafter referred to as Silty Sand and Clayey Sand unit. The general foundation design profile for each dam is presented as **Attachment 2**. Red lines designating the interfaces between various strata on the foundation design profile represent approximate boundaries and the transition between strata. Soil conditions will vary between boring locations.

Table 5 **Summary of Design Stratigraphic Units for Walnut Creek Dam Foundation**

Design Elevations/Thicknesses					Basis
Unit No.	**Description**	**Depth (feet), from**	**Depth (feet), to**	**Max Thickness (feet)**	**Justification**
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	0	42	42	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	32	71	39	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	67	87	20	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	77	97	20	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	97	120	23	Differentiated by material change and index properties.

Table 6 **Summary of Design Stratigraphic Units for Birch Creek Dam Foundation**

Design Elevations/Thicknesses					Basis
Unit No.	**Description**	**Depth (feet), from**	**Depth (feet), to**	**Max Thickness (feet)**	**Justification**
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	0	47	47	Differentiated by material change and index properties.
2, 5	Silty Clay and Sandy Clay	32	62	30	Differentiated by material change and index properties.
1, 3, 4, 6, 7	Silty Sand and Clayey Sand	52	120	68	Differentiated by material change and index properties.

4.2 Groundwater Elevation

The after-drilling groundwater elevations based on records from boring logs ranged between 5.5 to 10.2 fBGS for Walnut Creek Dam and from 5.8 to 26.5 fBGS for Birch Creek Dam. The recorded groundwater appears to follow the ground surface topography considering relatively shallow depths to groundwater at the low-lying borings B-2 and B-3. The overall rise in water levels during the 10-minute wait period indicates the intermittent layers of clay may be inducing a confining condition with a piezometric head higher than its spatial elevation. The design groundwater depth for the Project was set at a conservative 5.5 fBGS. Groundwater levels are controlled by topography and the stratigraphic conditions affecting groundwater flow, and level fluctuations may occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the borehole drillings were performed.

4.3 SPT N Value

A summary of the SPT N_{60} statistical analysis and design values for each stratum is included as **Table 7** and **Table 8,** where the basis of the design values is highlighted in bold. Statistical analyses and selection of design values were conducted in accordance with the methods described in **Section 3.3.** Plots of the total unit weight data and design values are included as **Figure 3** and **Figure 2**Error! Reference source not found..

Table 7 **SPT N60 Statistical Analysis, Data Comparison, and Design Values for Walnut Creek Dam**

Stratum	**SPT N60**					**Design Value (bpf)**
	Bottom Depth (feet)	**Sample Size (n)**	**Avg. (bpf)**	**Min. (bpf)**	**Max (bpf)**	
Silty Sand and Clayey Sand	37	16	18	4	34	**18**
Silty Clay and Sandy Clay	69	7	29	18	42	**29**
Silty Sand and Clayey Sand	82	6	40	16	50	**40**
Silty Clay and Sandy Clay	97	2	43	39	48	**43**
Silty Sand and Clayey Sand	120	5	50	50	50	**50**

Table 8 **SPT N60 Statistical Analysis, Data Comparison, and Design Values for Birch Creek Dam**

Stratum	**SPT N60**					**Design Value (bpf)**
	Bottom Depth (feet)	**Sample Size (n)**	**Avg. (bpf)**	**Min. (bpf)**	**Max (bpf)**	
Silty Sand and Clayey Sand	40	12	25	4	39	**25**
Silty Clay and Sandy Clay	58	4	33	18	42	**33**
Silty Sand and Clayey Sand	120	19	43	1	50	**43**

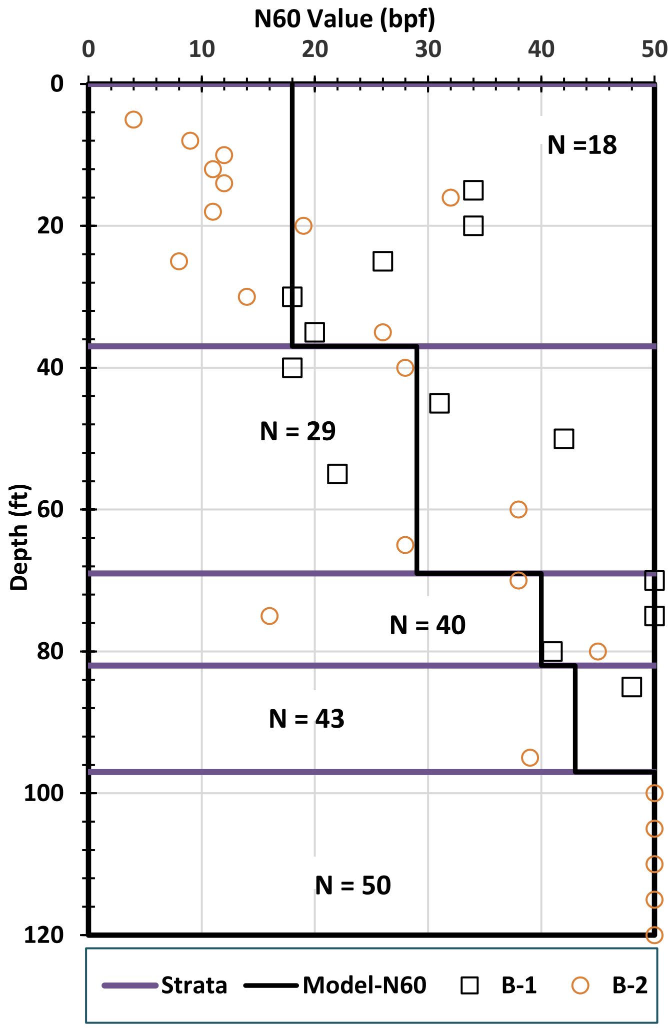

Figure 1. SPT N60 Values for Walnut Creek Dam