VANN ENGINEERING INC

Geotechnical Investigation Report

Pine Strawberry Water Improvement District
SV3 Waterline Improvements
Approximately 2,300 LF (0.44 miles) of Water Transmission Line
Near the Intersection of AZ-87/AZ-260 and FR708
Strawberry, Arizona 85544

Prepared for:

Justin Van De Graaff, P.E.
Sunrise Engineering, Inc.
2045 South Vineyard, Suite 101
Mesa, Arizona 85210

January 8, 2026

Project 33392

GEOTECHNICAL ENGINEERING ▪ ENVIRONMENTAL CONSULTING
CONSTRUCTION TESTING & OBSERVATION

VANN ENGINEERING INC

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Justin Van De Graaff, P.E.
Sunrise Engineering, Inc.
2045 South Vineyard, Suite 101
Mesa, Arizona 85210

RE: Geotechnical Investigation Report
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Justin,

Transmitted herewith is a copy of the final report of the geotechnical investigation on the above-mentioned project. The services performed provide an evaluation at selected locations of the subsurface soil conditions for the purpose of rippability. The materials encountered on the site are believed to be representative of the total area; however, soil and rock materials do vary in character between points of investigation. The recommendations contained in this report assume that the soil conditions do not deviate appreciably from those disclosed by the investigation. Should unusual material or conditions be encountered during construction, the soil engineer must be notified so that they may make supplemental recommendations if they should be required. As an additional service, this firm would be pleased to review the project plans and structural notes for conformance to the intent of this report. We trust that this report will assist you in the design and construction of the proposed project. Vann Engineering, Inc. appreciates the opportunity to provide our services on this project and looks forward to working with you during construction and on future projects. This firm possesses the capability of performing testing and inspection services during construction. Such services include, but are not limited to, compaction testing as related to fill control, foundation inspections and concrete sampling. Please notify this firm if a proposal for these services is desired. Should any questions arise concerning the content of this report, please feel free to contact this office directly

Respectfully submitted,

VANN ENGINEERING, INC.

Registered Professional Engineer (Geological)
CERTIFICATE NO. 15206
JEFFRY D. VANN
Date Signed 1/8/26
ARIZONA, U.S.A.

Mark Smelser, BS
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SECTION I

1.0 INTRODUCTION

Vann Engineering, Inc. understands that approximately 2,300 linear feet (0.44 miles) of water transmission line is being installed at the subject site, and that the approximate depth of the water lines shall be 4.0 to 5.0 feet. This document presents the results of a geotechnical investigation conducted by Vann Engineering, Inc. for the:

Pine Strawberry Water Improvement District
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Approximately 2,300 LF (0.44 miles) of Water Transmission Line
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The services performed provide an evaluation at selected locations of the subsurface soil and rock conditions for the purpose of rippability, trench construction, and safety. The following site plan shows the proposed waterline improvement locations (shaded in green), waterline size, and linear feet to be improved.

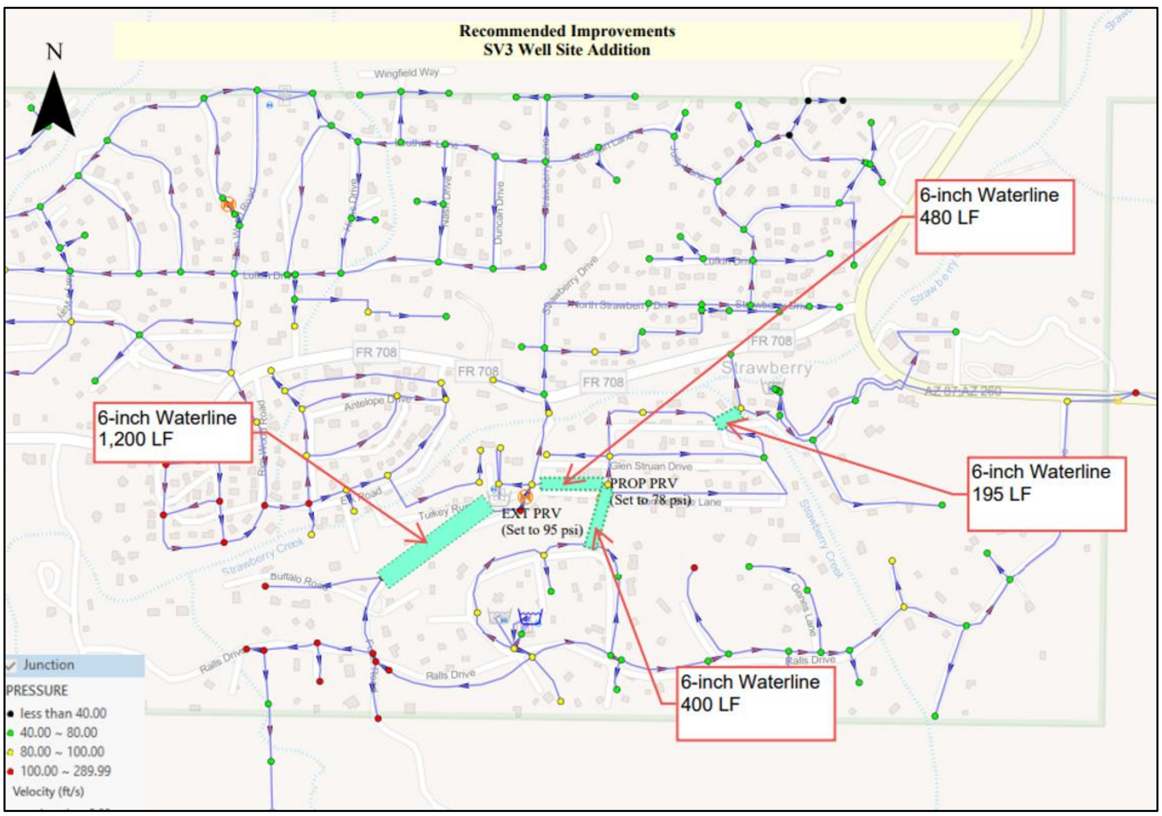


Figure 1: Site plan showing the proposed waterline improvement locations (shaded in green), waterline size, and linear feet to be improved

1.1 Purpose

The purpose of the investigation was two-fold: 1) to determine the physical characteristics of the soil and rock underlying the site, and 2) to provide recommendations pertaining to trench construction, safety, and excavation/rippability.

1.2 Scope of Services

The scope of services for this project includes the following:

- Description of the subject site
- Description of the major soil and rock layers
- Site Plan indicating the locations of all points of exploration
- Laboratory testing results
- Excavation cut slopes for trenches
- Excavation conditions - rippability
- Suitability of on-site soils for use as backfill
- Recommendations pertinent to pipe bedding and trench backfill criteria
- IBC Site Classification

Note: This report does not include, either specifically or by implication, any environmental assessment of the site or identification of contamination or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken. We are available to discuss the scope of work of such studies with you.

1.3 Authorization

The obtaining of data from the site and the preparation of this geotechnical investigation report have been carried out according to this firm's proposal (**VE25GT0905KM8 dated September 5, 2025**), authorized by **Justin De Graaff, P.E. (Master Form of Agreement)** to proceed with the work. Our efforts and report are limited to the scope and limitations as set forth in the proposal.

1.4 Standard of Care

Since our investigation is based upon review of background data, observation of site materials, and engineering analysis, the conclusions and recommendations are professional opinions. Our professional services have been performed using that degree of skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. These opinions have been derived in accordance with current standards of practice and no other warranty, expressed or implied, is made.

The limitations of this report and geotechnical issues which further explain the limitations of the information contained in this report are listed at 7.0.

2.0 PROJECT DESCRIPTION

2.1 Proposed Development

Vann Engineering, Inc. understands that approximately 2,300 linear feet (0.44 miles) of water transmission line is being installed at the subject site, and that the approximate depth of the water lines shall be 4.0 to 5.0 feet.

2.2 Site Description

The site consists of ABC covered and asphalt covered roadways that traverse terrain ranging from undulating to hillside. Visual observations reveal that some natural drainage washes associated with the terrain and some man-made drainage channels paralleling the roadways traverse portions of the site. A couple of soil cut/fill slopes were encountered scattered across portions of the site. Varying amounts of asphalt and ABC were encountered at the site. Greater thicknesses of asphalt and ABC may be encountered at locations not explicitly explored by this firm. Refer to the following table which shows the thickness of asphalt and ABC at each test boring location:

Table 1: Thicknesses of Asphalt and ABC encountered at each Test Boring Location

Test Boring No.	Thickness of Asphalt (in.)	Thickness of ABC (in.)
1	3.0	4.0
2	3.0	3.0
3	2.5	3.0
4	3.0	4.0
5	3.5	5.0

Refer to the following photographs taken during this firm's site investigation.

Figure 2: General site conditions

Figure 3: General site conditions

Figure 4: General site conditions

Figure 5: General site conditions

Figure 6: General site conditions

3.0 SUBSURFACE INVESTIGATION AND LABORATORY TESTING

3.1 Subsurface Investigation

The site's subsurface was explored through the utilization of five (5) exploratory test borings for examination of the subsurface profile. The test borings were advanced to depths of 5.0 feet utilizing a 4.5-inch continuous flight auger.

The locations of the test borings are shown on the Site Plans in Section II of this report and presented as TB-1 through TB-5. The soils encountered were examined, visually classified and wherever applicable, sampled. A field log was prepared for each test boring. The logs contain visual classifications of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples.

The final logs, included in Section II, represent our interpretation of the field logs and may include modifications based on laboratory observation and tests of the field samples. The final logs describe the materials encountered, their thicknesses, and the locations where samples were obtained. The sample locations are noted graphically on the boring logs included in Section II of this report. The Unified Soil Classification System was used to classify soils. Soil classification symbols, denoted on the boring logs, are briefly described in Section II.

In addition, the site subsurface was explored through the utilization of five (5) 24-channel refraction seismic survey lines, denoted on the Site Plans in Section II of this report. Each seismic survey line involved the retrieval of data in two separate directions (*forward and reverse*). As such, ten (10) refraction seismic surveys were conducted at the site. The lengths of the seismic survey lines were 36.0 feet, thereby allowing an examination of the subsurface to depths of 12.0 feet below the existing site grade. Information pertaining to the subsurface profile was obtained through analysis of seismic refraction data and geological observations of the site. Seismic wave velocities, representative of the various strata, are listed in Section II of this report.

Note: Changes in the calculated velocity indicate strata breaks or distinct changes within the same stratum. The important concept to remember with this method is that it is predominantly effective where velocities increase from layer to layer, moving downward from the surface. Analytical methods are used by this firm for determining the depth to the various layers, even in the most complex multi-layer situations. However, when a denser, and hard soil or rock layer overlies a weaker or less dense soil or rock layer, the weaker or less dense layer is masked and not detected by the seismograph. If a weaker layer is encountered during the excavation efforts, this office should be contacted immediately for further recommendations. Generally, the depth of a seismic survey investigation is approximately equal to one-third the length of the survey.

Seismic survey exploration depths, as mentioned above and depicted on the Cross Sections presented herein, are calculated by using a computer program (SeisImager 2D) that generates cross sections of the subsurface geology at each seismic survey location. Further, total exploration depths, as stated above, of the seismic survey study may vary from one survey line to the next. Furthermore, the calculated depths are dependent on the program's ability to interpret the subsurface layering and are based primarily on the penetration and refraction of the seismic wave into and through the subsurface stratum.

The materials encountered on the site are believed to be representative of the total area; however, soil and rock materials do vary in character between points of investigation. The recommendations contained in this report assume that the soil conditions do not deviate appreciably from those disclosed by the investigation. Should unusual materials or conditions be encountered during construction, the soil engineer must be notified so that they may make supplemental recommendations if required.

3.2 Laboratory Testing

Laboratory analyses were performed on a representative soil sample to aid in material classification and to estimate pertinent engineering properties of the on-site soils in preparation of this report. Testing was performed in general accordance with applicable test methods.

Representative samples obtained during the field investigation were subjected to the following laboratory analyses:

Table 2: Laboratory Testing

Test	Sample(s)	Purpose
Expansion	Remolded subgrade Soils (7)	Potential for heave upon wetting
Sieve Analysis, Atterberg Limits, and Moisture Content	Native subgrade soils (5)	Soil classification and in-situ moisture content
Soluble Sulfates and Chlorides	Native subgrade soils (1)	Limited soil-related corrosion potential

Refer to Section III of this report for the complete results of the laboratory testing. The samples will be stored for 30 days from the date of issue of this report, and then disposed of unless otherwise instructed in writing by the client.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The seismic refraction survey data obtained from the site was analyzed and subjected to computer aided analyses relative to engineering applications. The seismic refraction survey data indicates the following physical and mechanical properties of the subsurface soil and rock.

Variations on the order of 1.5 feet may be encountered in the layer depth calculations due to the variability of the materials, degrees of weathering, and orientation of the structures. Refer to the following tomographic cross sections, test boring logs, and the general layered cross sections located in Section II of this report for the subsurface layering determined by analysis of the seismic refraction survey and test boring data.

The locations of the seismic surveys and test borings are depicted on the Site Plans in Section II.

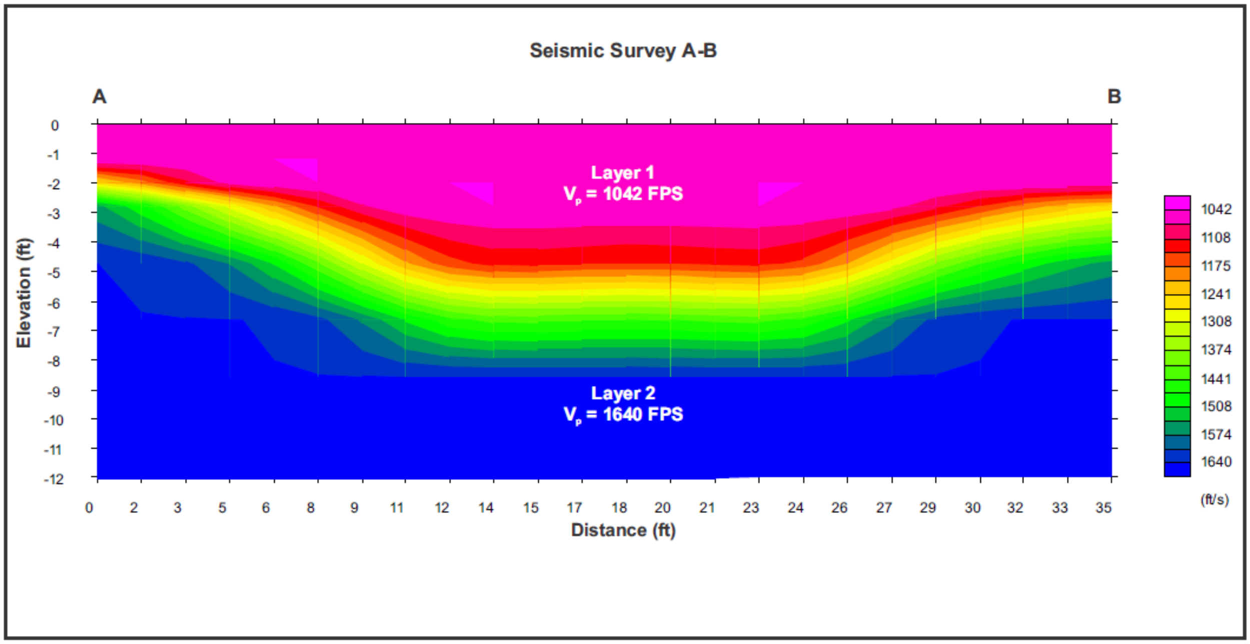


Figure 7: Tomographic Cross Section of Seismic Survey Line A-B

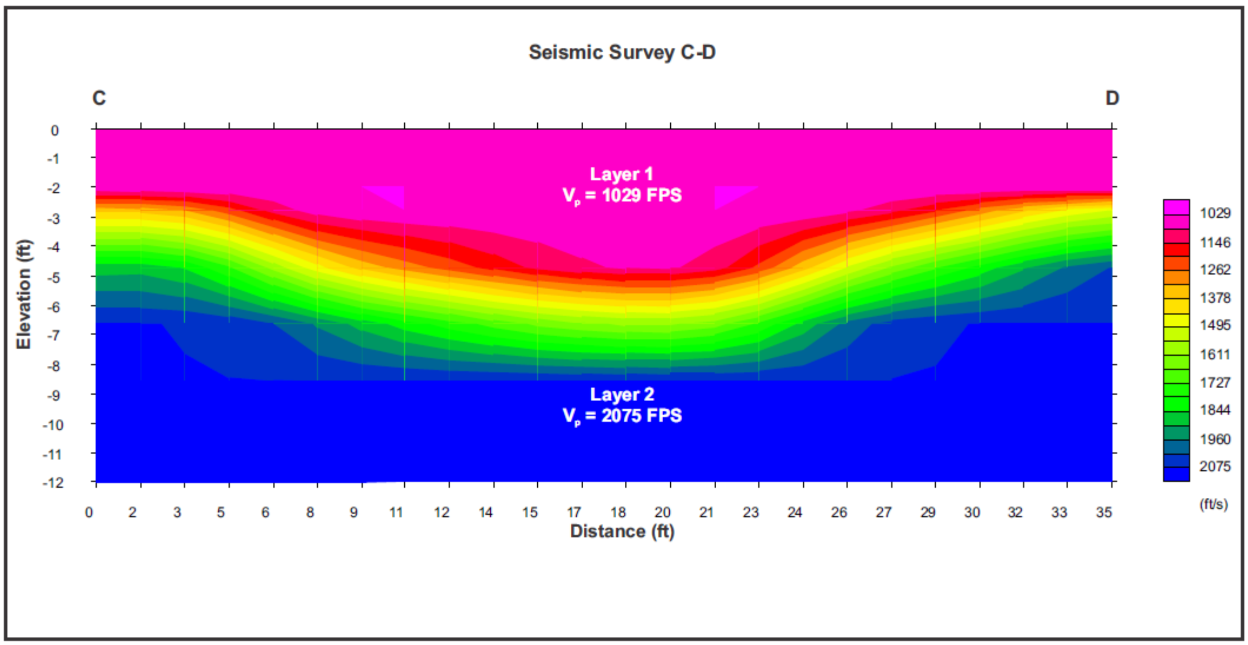


Figure 8: Tomographic Cross Section of Seismic Survey Line C-D

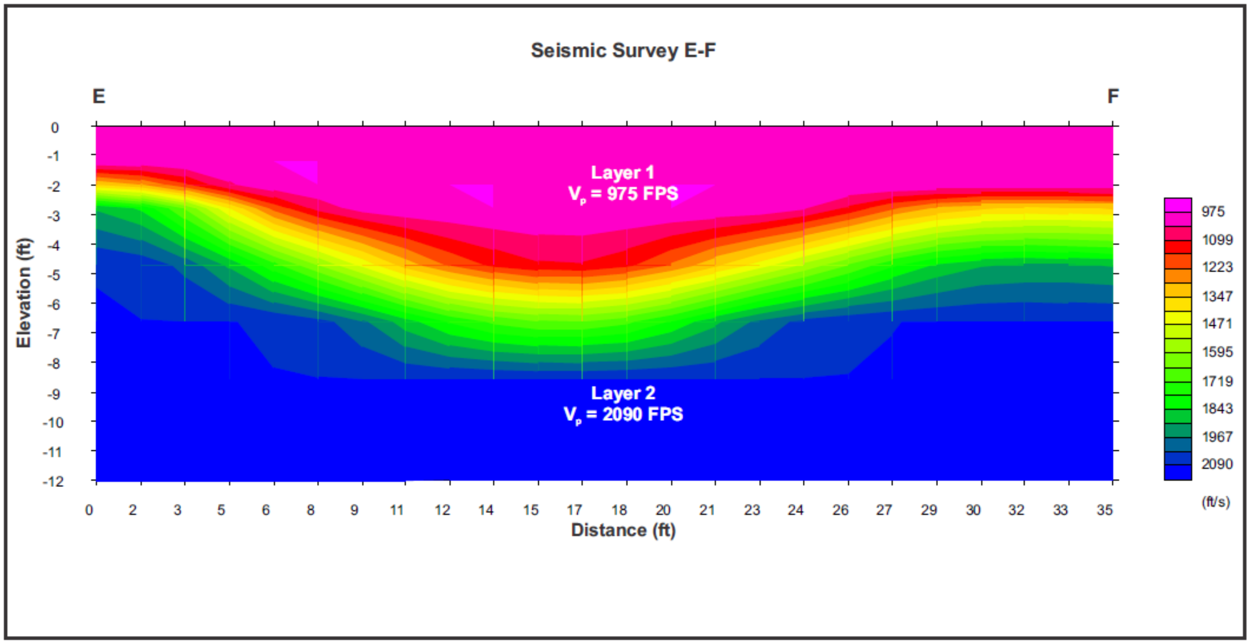


Figure 9: Tomographic Cross Section of Seismic Survey Line E-F

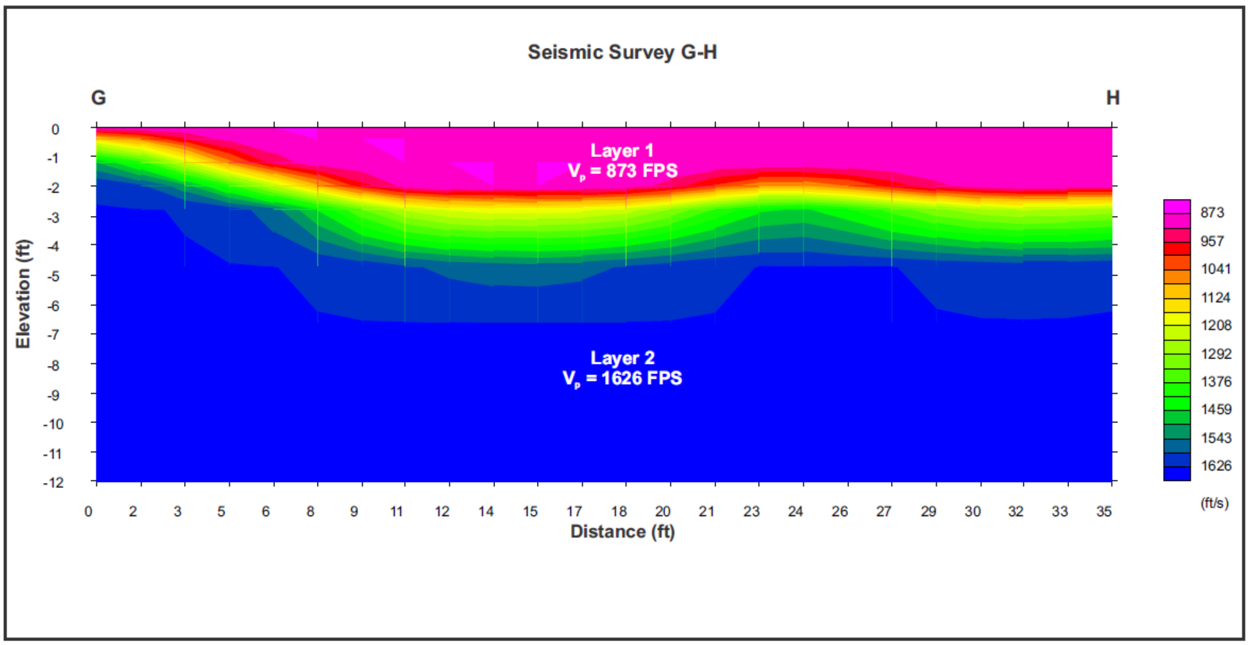


Figure 10: Tomographic Cross Section of Seismic Survey Line G-H

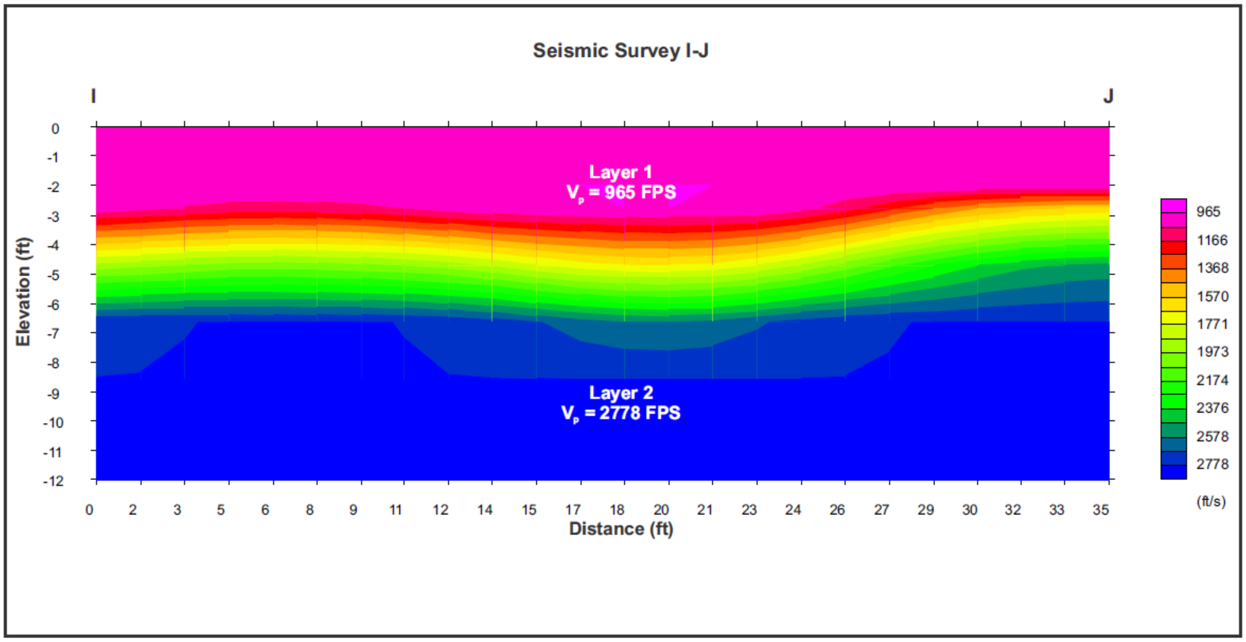


Figure 11: Tomographic Cross Section of Seismic Survey Line I-J

4.2 Groundwater

No groundwater was encountered during the course of this firm's site investigation to a depth of 5.0 feet. Groundwater is expected to be at a depth of approximately 110.20 feet according to well data measurements in the area (GWSI Registry ID: 55-588181).

Refer to the following Arizona Groundwater Site Inventory (GWSI) map for an approximate location of the site in relation to the nearby well:

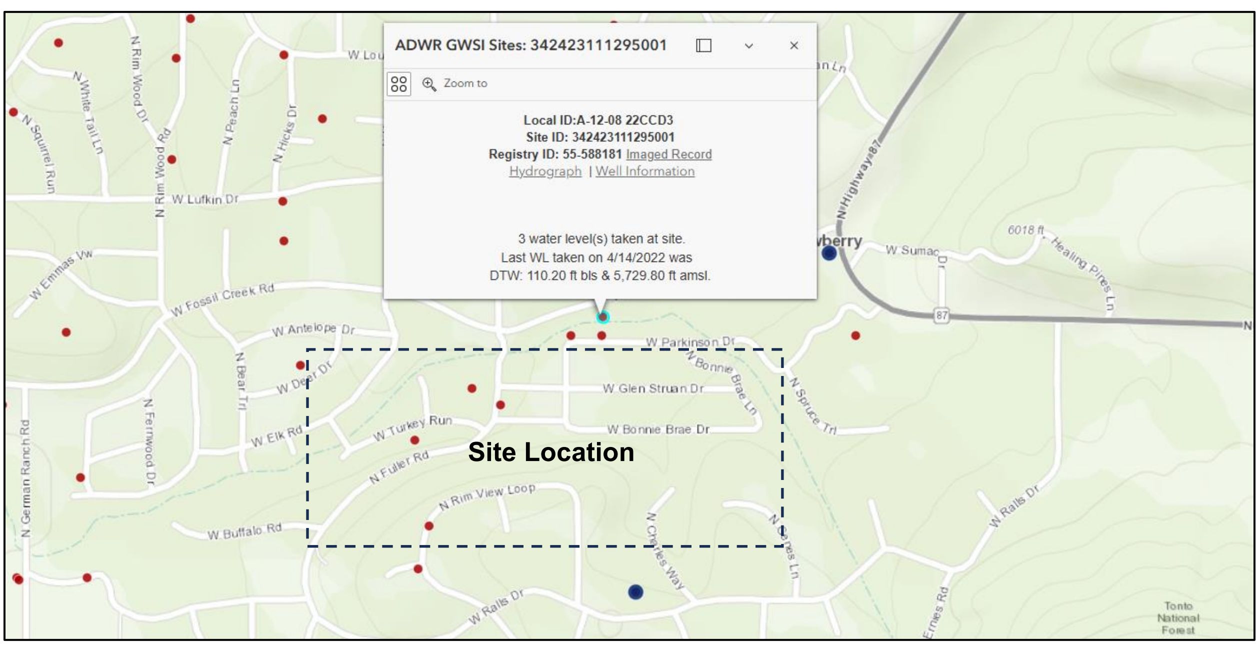


Figure 12: Groundwater Map

4.3 Local Geology

The local geology and our field investigation indicate that a thin layer of firm to stiff soil (defined herein as Layer 1) overlies a very stiff to hard soil (defined herein as Layer 2). Please Note: although no rock was encountered at the locations tested, the listed rock types are known to be present in the region, and the listed rock types only describe the rock types known to exist within the specified geologic units.

Refer to the following Geologic Map for the specific rock types known to exist in the.

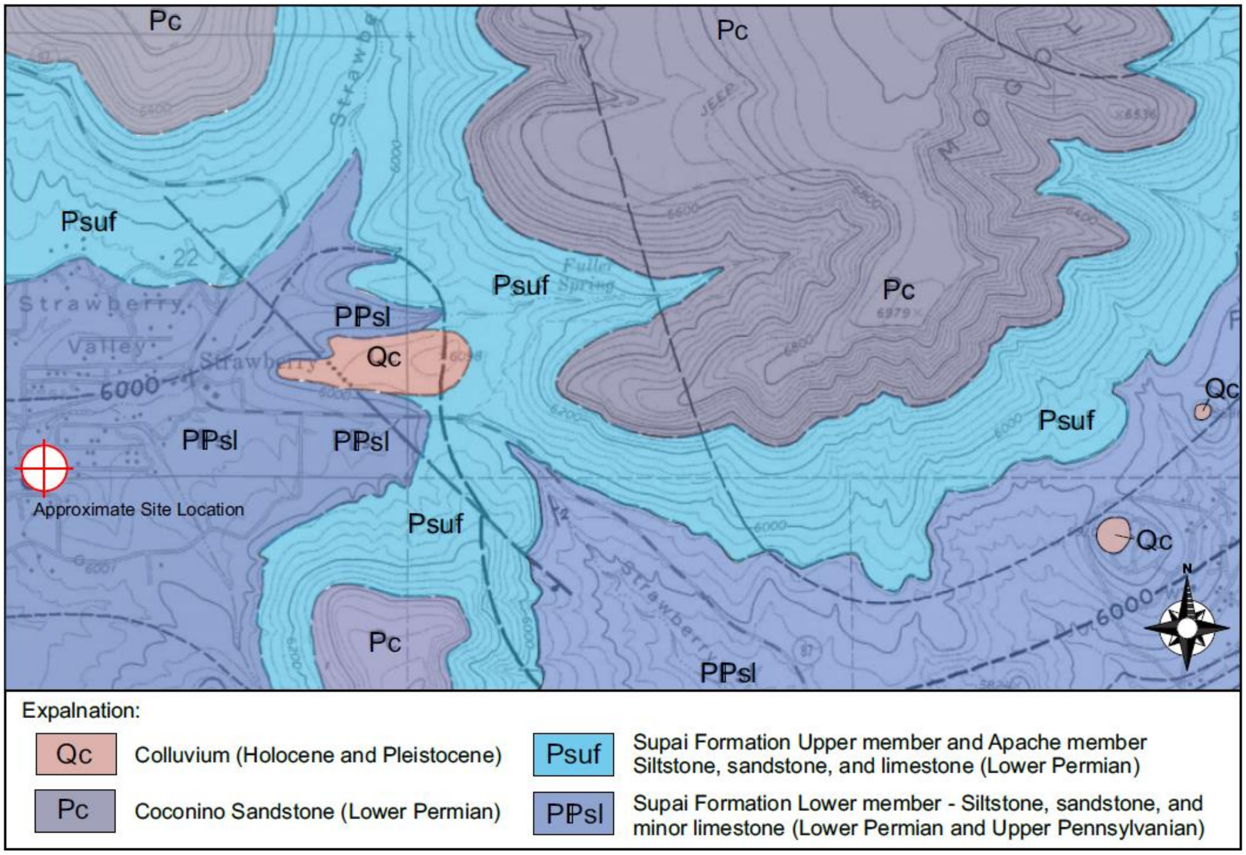


Figure 13: Geologic map of site and surrounding area

Geologic Map referenced from the U.S. Department of the Interior U.S. Geological Survey, Geologic Map of the Pine 7.5 Quadrangle, Coconino and Gila Counties, Arizona, Miscellaneous Field Studies Map MF-2123, by Melanie K. Weisman and Gordon W. Weir, 1990. Color added by Vann Engineering, Inc.

4.4 Limited Soil-Related Corrosion Discussion

The values presented for corrosion-related laboratory testing should be used to determine potentially corrosive characteristics of the on-site soils tested with respect to their contact with the various construction materials that will be used at the subject property. The corrosion-related laboratory testing results are specific to the locations and elevations sampled and no other inference is implied. If the actual on-site soils that will be in contact with structures and construction materials are from different locations and elevations than those presented herein, additional corrosion testing must be performed.

Table 3: Corrosion Test Results Summary

Sample Location	Depth Interval	Chlorides (ppm)	Sulfate (%)
TB-2	4.0’-5.0’	12	0.133

The project structural engineer should cross reference the soluble sulfate and chloride testing results from the locations and depth intervals presented with Table 19.3.1.1 of Section 318 of the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete to determine the appropriate exposure class to utilize for the project.

All corrosion-related laboratory testing presented herein must be provided to the on-site contractors and material specifiers to obtain recommendations on corrosion from the suppliers of the materials that will be used. Corrosion can result from many combinations of environmental conditions, materials, construction design, landscaping, and other factors, and no single guideline addresses all corrosion possibilities. Nevertheless, important corrosion information can be obtained from the American Wood Protection Association (AWPA), the International Building Code (IBC), International Residential Code (IRC), and local building codes.

Vann Engineering is not a corrosion engineering firm, and the scope of our work was limited to performing corrosion-related laboratory testing on selected samples at specific locations and elevations, presenting the results herein, and providing a brief comparison of the corrosion-related laboratory testing results to selected criteria. A registered corrosion engineer must be consulted if the potential corrosion of construction materials, underground utilities, and structures is a concern.

5.0 RECOMMENDATIONS

The recommendations contained herein are based upon the properties of the surface and subsurface soils and rocks as described by the field evaluation, the results of which are presented and discussed in this report. Alternate recommendations may be possible and will be considered upon request.

5.1 Excavating Conditions

Excavations greater than 4.0 feet should be sloped or braced as required to provide personnel safety and satisfy local safety code regulations. For the soil layer, a 1:1 slope must not be exceeded.

Subsurface soils with a plasticity index less than 12 will be susceptible to sloughing. It is recommended that the soil strata details from the boring logs in Section II of this report be utilized in conjunction with on-site observations to determine when appropriate measures be incorporated in the final design and construction to mitigate potential damage and injuries associated with sloughing.

The potentially hard dig layer (measured in degrees of rippability denoted as "rippable," "marginal," and "non-rippable") consists of soil and rock of varied constituencies. The temporary excavation slopes for the potentially hard dig layers are presented on the following table. The following table summarizes the seismic wave velocity, temporary excavation slopes, and possible rippability conditions for the various layers.

The rippability conditions are based on the seismic P-wave velocities and data utilized by Caterpillar Inc. and included in their "Handbook of Ripping."

Table 4: Potentially Hard Dig Equipment Performance Based on Seismic Velocity (Adapted from Caterpillar Performance Handbook)

Seismic Survey Line/Test Boring	Hard Dig Layer Description	Depth of Occurrence (ft)	Velocity (fps)	Temporary Trench Slope Excavation once the Hard Dig Layer is Encountered (H:V)	Excavating Equipment Performance/Requirement			
					D8R/D8T, or equivalent	D9R/D9T, or equivalent (Hard Dig)	D10T2, or equivalent (Very Hard Dig)	D11, or equivalent (Extremely Hard Dig)
TB-1	None Encountered to 5.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
TB-2	None Encountered to 5.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
TB-3	None Encountered to 5.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
TB-4	None Encountered to 5.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
TB-5	None Encountered to 5.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
A-B	None Encountered to 12.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
C-D	None Encountered to 12.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
E-F	None Encountered to 12.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
G-H	None Encountered to 12.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable
I-J	None Encountered to 12.0 feet	-	-	1:1	Rippable	Rippable	Rippable	Rippable

Variations on the order of 1.5 feet may be encountered in the layer depth calculations due to the variability of the materials, degrees of weathering, and orientation of the structures at the locations of the seismic surveys.

Temporary construction slopes should be designed and excavated in strict compliance with the rules and regulations of the Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA), 29 CFR, Part 1926. This document was prepared to better ensure the safety of workers entering trenches or excavations and requires that all excavations conform to new OSHA guidelines. The contractor is solely responsible for protecting excavations by shoring, sloping, benching or other means as required to maintain stability of both the excavation sides and bottom. Vann Engineering, Inc. does not assume any responsibility for construction site safety or the activities of the contractor.

The subsurface soils are considered to be OSHA Type B soil. Temporary excavations into Type B (Layer 1) soils are to be configured no steeper than a 1H:1V incline. The maximum temporary trench depth, without the use of shoring, is 20.0 feet (OSHA maximum). Deviation from these recommendations will necessitate a trench support system or shield.

The above recommendations apply to a maximum cut slope / trench cut height of 5.0 feet.

This firm should be notified during construction in order to verify field conditions and inspect all cut slopes for structural features (e.g., shear zones, foliation, fracture orientations, joint orientations and slabbing) contained within the rock mass that could lead to slope instability and eventual slope failure. If conditions relative to the integrity and stability of the rock mass are observed during the site excavation and are noted during a site inspection, this firm may alter the above-recommended cut slopes to adhere to a more stable condition. Therefore, it is critical that all cut slope excavations be inspected at a point when; if unstable conditions are identified, mitigation measures can be implemented before large scale cuts have been performed or slope failure occurs (i.e., timely inspecting and potentially modifying the cut slope recommendations, or possibly recommending the use of rock anchors, rock netting, or retaining walls for slope stability, while the cut is not yet greater than 10 feet in height).

Unforeseen conditions may develop during cutting operations. If conditions arise which were not addressed by this design, it is imperative that this firm be notified such that the situation can be addressed properly. In all construction activities related to site grading, the concept of toe removal should become well understood. All slopes, whether they are natural or fill, have a toe (the lowest portion of the slope). When the toe is removed, the slope may become unstable. For purposes of construction, the entire site should be considered to exist on a slope. Any cut into the natural slope will result in the removal of the toe for the up-slope portion, resulting in the potential movement of up-slope boulders riding on the surface.

Trenches extending below 5.0 feet below construction grade will require the contractor to develop a trench safety plan to protect personnel entering the trench or trench vicinity.

The preceding table presents this firm's analysis of safe cut slopes for the anticipated subsurface conditions. **However, it should be noted that the subsurface layers), once exposed, could reveal hidden characteristics that may indicate the potential for slope instability during and after cutting operations.**

5.2 Site Classification

This project is not located over any known active faults or fault-associated disturbed zones. An IBC Site Class of D may be utilized in the earthquake considerations for the proposed site.

5.3 Maximum Contact Stresses to Maintain Zero Settlement

The anticipated trench has an anticipated depth of 4.0 to 5.0 feet. With that, the maximum contact stress of the proposed system cannot exceed that which would cause any appreciable settlement.

The following table is a guide to the maximum allowable contact stress (or allowable soil bearing capacity) to maintain a relatively zero settlement condition:

Table 5: Maximum Contact Stress

Depth below the Surface	Maximum Contact Stress (Allowable Bearing Capacity) to Maintain Zero Settlement
4.0 feet	480 psf
5.0 feet	600 psf
6.0 feet	720 psf
7.0 feet	840 psf
8.0 feet	960 psf

5.4 General Site Preparation

The following recommendations are presented as a guide in the compilation of construction specifications. The recommendations are not comprehensive contract documents and should not be utilized as such.

Although underground facilities such as septic tanks, cesspools, basements, and dry wells were not encountered, such features might be encountered during construction. These features should be demolished or abandoned in accordance with the recommendations of the geotechnical engineer. Such measures may include use of an approved CLSM.

The bottoms of the trench excavations should expose competent soils and should be relatively dry and free of loose, soft, or disturbed soils. If questionable soils are encountered at the base of the trench excavation, their competency should be verified through probing and density testing. Following completion of trench excavation, a minimum of 8.0 inches of the cut native surface soils should be scarified, moisture processed and compacted as specified herein. The scarification and compaction requirement applies to cut situations as well as fill situations. Soft, wet, or weak soils and deleterious materials should be subexcavated and/or overexcavated to expose competent soils.

Any excavated material may be reused as backfill provided the maximum particle size is 3.0 inches, and a suitable percentage of fines will be generated to ensure a stable mixture. Complete removal and cleaning of any undesirable materials and proper backfilling of trenches will be necessary to ensure minimal settlement.

5.5 Trench Bedding, Haunching Material, and Initial Backfilling Material (Shading)

Bedding shall consist of Select Material Type B or Aggregate Base as per MAG Table 702 (presented below for reference), or equivalent local government specifications, or granular material (Section 601.4.8 in MAG – see below), or equivalent local government specifications, containing no pieces larger than 1-1/2 inches and free of broken concrete, broken pavement, wood or other deleterious material.

Open graded rock will not be used without written approval from this office. Recycled asphalt cannot be utilized as bedding.

Table 702-1			
Sieve Analysis Test Methods AASHTO T-27, T-11			
Sieve Size	**Accumulative Percentage Passing Sieve, by Weight**		
	Select Material		**Aggregate Base Course**
	Type A	**Type B**	
3 in.	100	- -	- -
1-1/2 in.	- -	100	100
1 in.	- -	- -	90 – 100
No. 4	30 - 75	30 - 70	38 - 65
No. 8	20 - 60	20 - 60	25 – 60
No. 30	10 - 40	10 - 40	10 – 40
No. 200	0 - 12	0 - 12	3 – 12
Plasticity Index Test Methods AASHTO T-89 Method A, T-90, T146 Method A			
Maximum allowable value	5	5	5
Fractured Face, One Face Test Method ARIZ 212, Percent by Weight of the Material Retained on a #4 Sieve			
Minimum required value	50	50	50
Resistance to Degradation and Abrasion by the Los Angeles Abrasion Machine Test Method AASHTO T-96, Percent Loss by Weight			
Maximum allowable value at 100 revolutions	10	10	10
Maximum allowable value at 500 revolutions	40	40	40

Figure 14: Bedding Material Specifications (reference: MAG Table 702-1), or equivalent local government specifications

For purposes of this specification (601.4.8 Granular Material and Native Backfill Material), granular material is material for which the sum of the plasticity index and the percent of the material passing a No. 200 sieve does not exceed 23. The Plasticity index shall be tested in accordance with, T-89 and T-90. Haunching material is placed between the bedding and the springline. It should be comprised of ABC material according to the above table. Initial backfilling material (shading) must also be comprised of ABC. Shading extends from the springline upward to 1.0 feet above the top of the pipe.

5.6 Final Backfill Compaction

Final backfill compaction applies to backfill placed between 1.0 feet above the top of the pipe upward to the final surface/grade. All backfill compaction densities shall be placed as specified as follows, or by local government specifications, or equivalent, the minimum density requirements for trench backfill listed. Bedding shall consist of granular material containing no pieces larger than 1.5 inches and free of broken concrete, broken pavement, wood or other deleterious material. Open graded rock will not be used without the written approval of the Engineer.

Table 6: Minimum Trench Compaction Densities

Backfill Type	Location	From Surface to 2 feet Below Surface	From 2 feet Below Surface to 1 foot Above Top of Pipe	From 1 foot Above Top of Pipe to Bottom of Trench
I	Under any existing or proposed pavement, curb, gutter, sidewalk, or such construction included in the contract, or when any part of the trench excavation is within 2 feet of the above.	100% for granular 95% for non-granular	95%	95%
II	On any utility easement street, road, or alley right-of-way outside limits of backfill Type 1.	85%	85%	90%
III	Around any structures or exposed utilities outside limits of backfill Type 1.	95% in all cases	III	Around any structures or exposed utilities outside limits of backfill Type 1.

All backfill should be placed in horizontal lifts with compacted thicknesses not to exceed one foot. The moisture content of all final backfill soil should be within the range of optimum -2 to optimum +4 percent. Water settling shall not be used to settle such materials in any part of the trench.

5.7 Utility Trench Backfill Settlement

If backfills are not compacted as recommended, excessive settlement may result. Excessive settlement of loose trench backfill may cause damage to overlying and nearby pavements, slabs, pedestrian walkways, planters, and other structures. Flooding has also been experienced in below grade areas due to breakage of utility lines embedded in loose retaining wall backfills, and from infiltration of surface water (irrigation and/or rainfall) through loose retaining wall backfills. Backfills may consist of compacted native soils. Backfill compaction should be accomplished by mechanical methods. Water jetting or flooding of loose, dumped backfills to increase moisture contents should be prohibited.

Even with proper backfill compaction (well compacted – 95 percent minimum), the backfill will have the potential for about 1.2 inches of settlement (for 10.0 feet of total backfill) in the event of wetting by irrigation or broken conduits. With moderately compacted backfill (90 percent minimum), the magnitude of backfill settlement may approach 3.0 inches (for 10.0 feet of total backfill).

Further, with poorly compacted backfill (85 percent minimum), the approximate magnitude of backfill settlement may reach as much as 6.0 inches (for 10.0 feet of total backfill). The preceding estimates for backfill settlement are those which may occur through settlement of the backfill alone, without any surcharge or other structural loading condition. Refer to the following table which reflects the anticipated settlement without any structural loads.

Table 7: Backfill Settlement

Backfill Types			Anticipated Settlement without any Structural Loads (in.)		
% Compaction	Description	% Estimated Strain	2.5 feet of backfill	5.0 feet of backfill	7.5 feet of backfill
95-98	Very Well Compacted	**0.5**	0.15	0.3	0.45
95	Well Compacted	**1**	0.3	0.6	0.9
90	Moderately Compacted	**2.5**	0.75	1.5	2.25
85	Poorly Compacted	**5**	1.5	3.0	4.5
80	Very Poorly Compacted	**7.5**	2.25	4.5	6.75

6.0 ADDITIONAL SERVICES

Vann Engineering, Inc. should be retained to provide documentation that the recommendations presented herein are met. This firm possesses the capability of performing testing and inspection services during the course of construction. Such services include, but are not limited to, compaction testing and concrete sampling and testing. Please notify this firm if a proposal for these services is desired.

The recommendations contained in this report are contingent on Vann Engineering, Inc. observing and/or monitoring:

A. Suitability of borrow materials
B. Backfilling and compaction of excavations (e.g., Utility trench backfill)
C. Concrete sampling and testing for footings, stem walls and floor slabs
D. Compliance with the geotechnical recommendations

7.0 LIMITATIONS

This report is not intended as a bidding document, and any contractor reviewing this report must draw their own conclusions regarding specific construction techniques to be used on this project. The scope of services carried out by this firm does not include an evaluation pertaining to environmental issues. If these services are required by the lender, we would be most pleased to discuss the varying degrees of environmental site assessments. This report is issued with the understanding that it is the responsibility of the owner to see that its provisions are carried out or brought to the attention of those concerned. In the event that any changes of the proposed project are planned the conclusions and recommendations contained in this report shall be reviewed and the report shall be modified or supplemented, as necessary.

7.1 Site Soil and Rock Changes

The materials encountered on the site are believed to be representative of the total area; however, soil and rock materials do vary in character between points of investigation. The recommendations contained in this report assume that the soil conditions do not deviate appreciably from those disclosed by the investigation. Should unusual material or conditions be encountered during construction, the soil engineer must be notified so that supplemental recommendations may be considered if they are required.

7.2 Recommendations Prior to and During Construction

Prior to construction, we recommend the following:

- Consultation with the design team in all areas that concern soils and rocks to ensure a clear understanding of all key elements contained within this report.
- Review of the General Structural Notes to confirm compliance to this report and determination of which allowable soil bearing capacity has been selected by the project structural engineer (this directly affects the extent of earthwork and foundation preparation at the site).
- This firm be notified of all specific areas to be treated as special inspection items (designated by the architect, structural engineer or governmental agency).

Relative to this firm's involvement with the project during the course of construction, we offer the following recommendations:

- The site or development owner should be directly responsible for the selection of the geotechnical consultant to provide testing and observation services during the course of construction.
- This firm should be contracted by the owner to provide the course of construction testing and observation services for this project, as we are most familiar with the interpretation of the methodology followed herein.
- All parties concerned should understand that there exists a priority surrounding the testing and observation services completed at the site.

7.3 Time Length Report Limitations

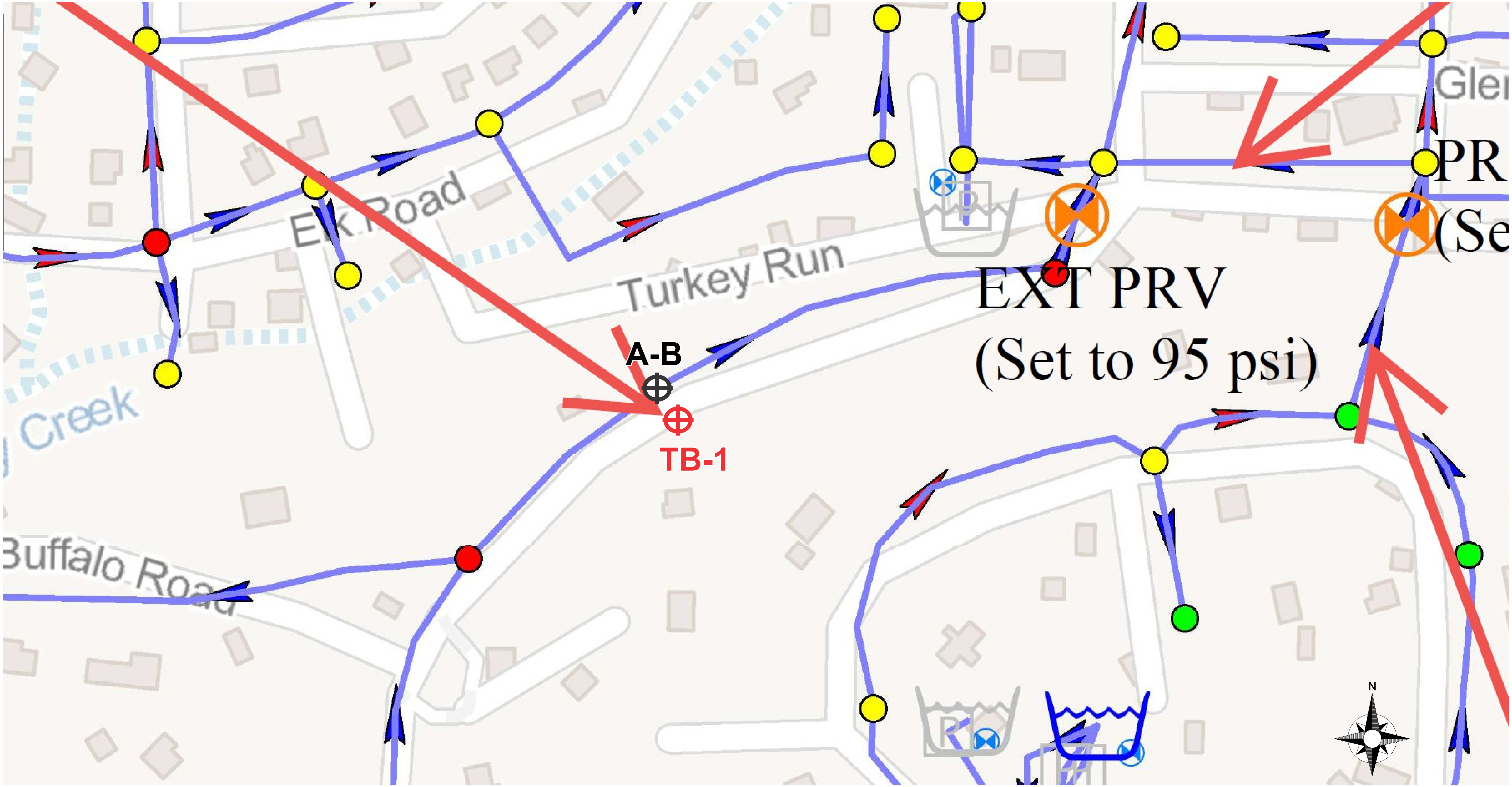
Regarding the use of this report, the following limitations, restrictions, and actions are imposed on the use of this report, depending on the timeframe between the initial report issue date and its ability to be used for additional design and construction:

- Report prepared within 1 year – preparation of a simple update / reliance letter.
- Report prepared between 1 and 3 years – preparation of a letter preceded by a site visit from an engineer from the firm to verify consistent field conditions (report to include new code changes, standard-of-care changes, and internal recommendation-protocol changes).
- Report prepared between 3 and 7 years – preparation of an update following potentially limited new field exploration, laboratory testing, code changes, standard-of-care changes, and internal recommendation-protocol changes.
- Report prepared beyond 7 years – a new report is mandated.

DEFINITION OF TERMINOLOGY

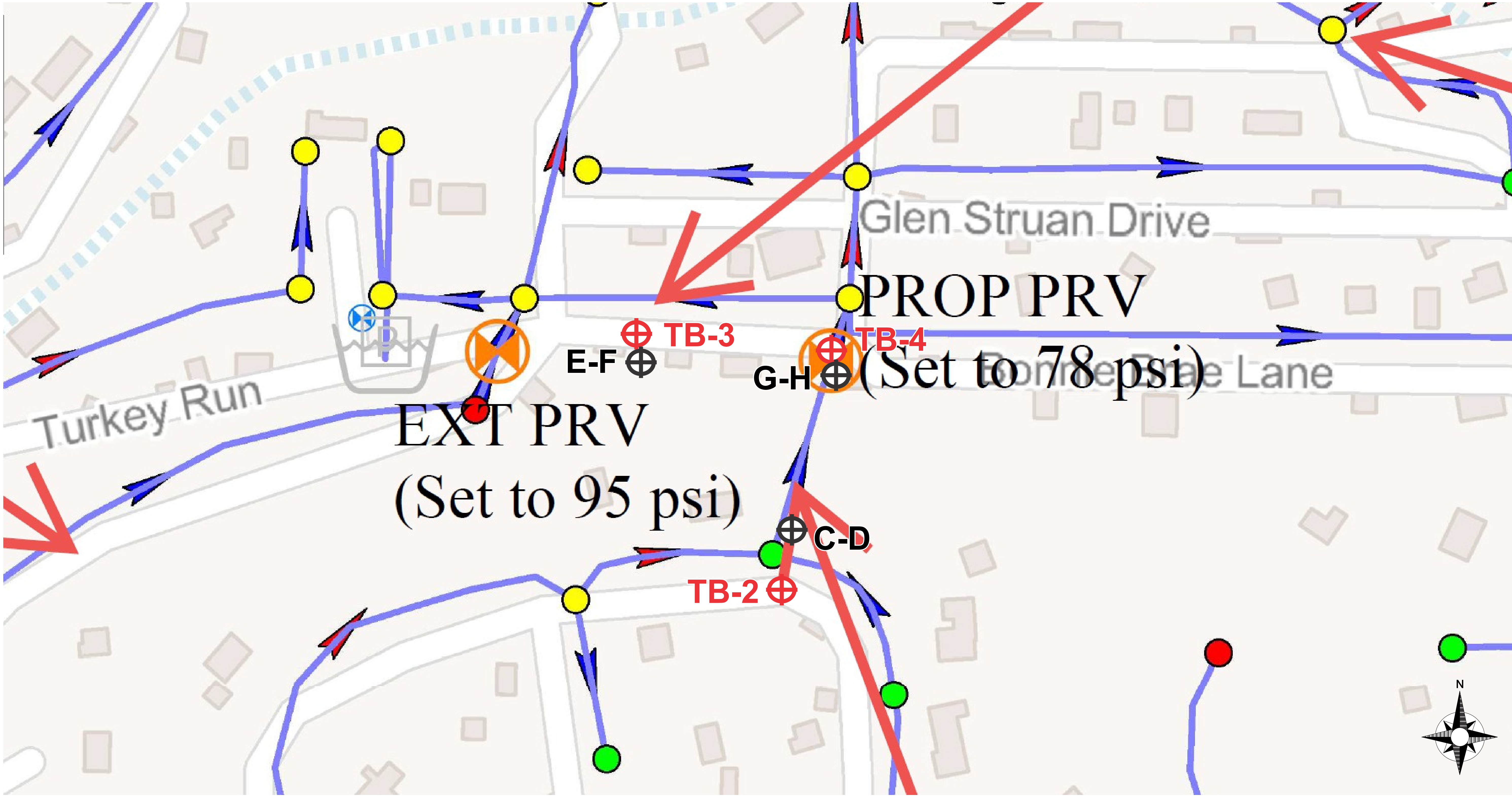
Allowable Soil Bearing Capacity Allowable Foundation Pressure	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
Aggregate Base Course (ABC)	A sand and gravel mixture of specified gradation, used for slab and pavement support.
Backfill	A specified material placed and compacted in a confined area.
Base Course	A layer of specified material placed on a subgrade or subbase.
Base Course Grade	Top of base course.
Bench	A horizontal surface in a sloped deposit.
Caisson	A concrete foundation element cased in a circular excavation, which may have an enlarged base. Sometimes referred to as a cast-in-place pier.
Concrete Slabs-on-Grade	A concrete surface layer cast directly upon a base, subbase, or subgrade.
Controlled Compacted Fill	Engineered Fill. Specific material placed and compacted to specified density and/or moisture conditions under observation of a representative of a soil engineer.
Differential Settlement	Unequal settlement between or within foundation elements of a structure.
Existing Fill	Materials deposited through the action of man prior to exploration of the site.
Expansive Potential	The potential of a soil to increase in volume due to the absorption of moisture.
Fill	Materials deposited by the action of man.
Finish Grade	The final grade created as a part of the project.
Heave	Upward movement due to expansion or frost action.
Native Grade	The naturally occurring ground surface.
Native Soil	Naturally occurring on-site soil.
Overexcavate	Lateral extent of subexcavation.
Rock	A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting, or other methods of extraordinary force for excavation.
Scarify	To mechanically loosen soil or break down the existing soil structure.
Settlement	Downward movement of the soil mass and structure due to vertical loading.
Soil	Any unconsolidated material composed of disintegrated vegetable or mineral matter, which can be separated by gentle mechanical means, such as agitation in water.
Strip	To remove from present location.
Subbase	A layer of specified material between the subgrade and base course.
Subexcavate	Vertical zone of soil removal and recompaction required for adequate foundation or slab support
Subgrade	Prepared native soil surface.

SECTION II



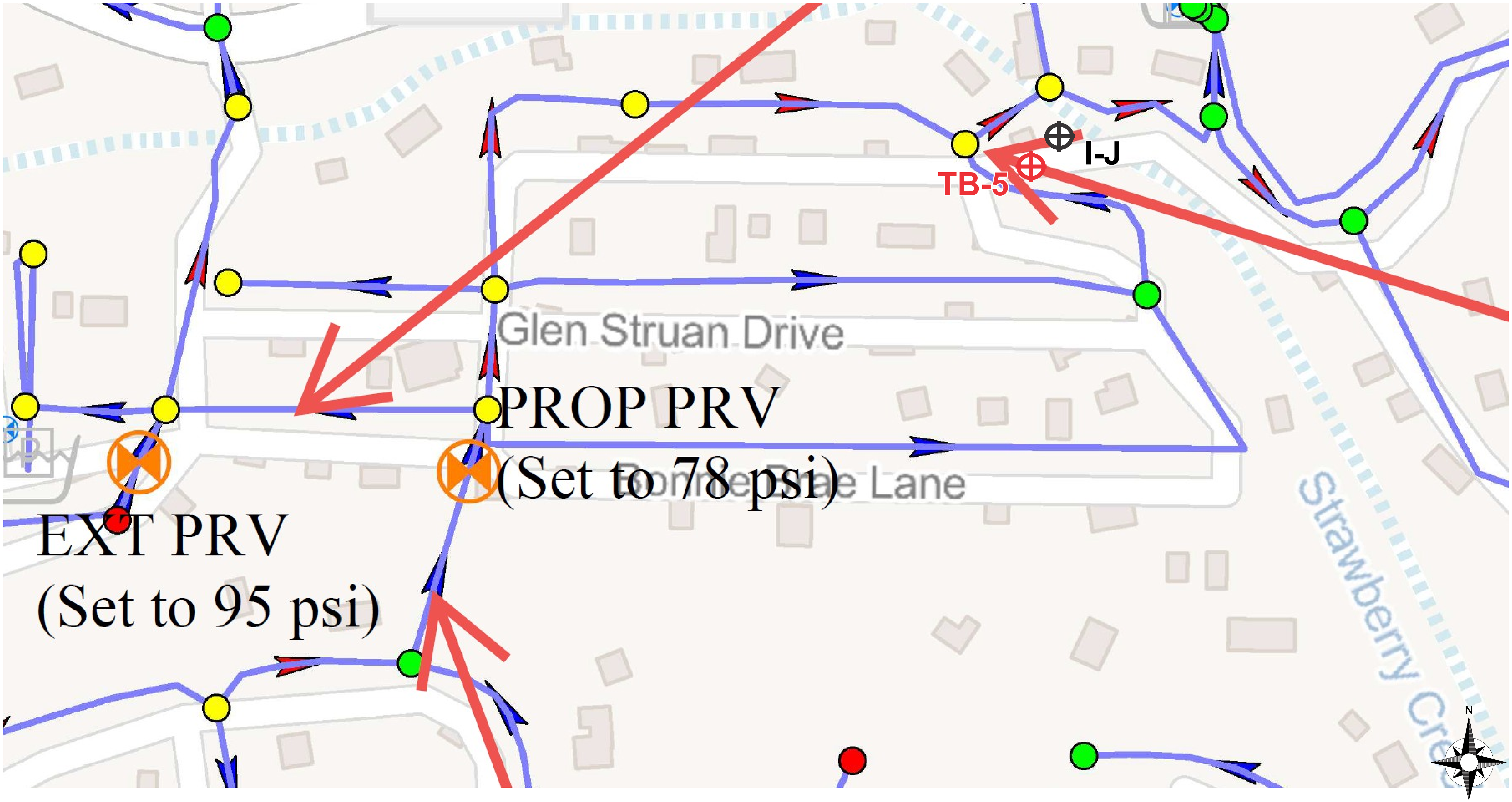
SITE PLAN 1 | PROJECT 33392
PINE STRAWBERRY WATER IMPROVEMENT DISTRICT - SV3 WATERLINE IMPROVEMENTS
APPROXIMATELY 2,300 LF (0.44 MILES) OF WATER TRANSMISSION LINE
NEAR THE INTERSECTION OF AZ-87/AZ-260 AND FR708
STRAWBERRY, ARIZONA 85544

TEST BORING LOCATION
SEISMIC SURVEY LOCATION



SITE PLAN 2 | PROJECT 33392
PINE STRAWBERRY WATER IMPROVEMENT DISTRICT - SV3 WATERLINE IMPROVEMENTS
APPROXIMATELY 2,300 LF (0.44 MILES) OF WATER TRANSMISSION LINE
NEAR THE INTERSECTION OF AZ-87/AZ-260 AND FR708
STRAWBERRY, ARIZONA 85544

TEST BORING LOCATION
SEISMIC SURVEY LOCATION



SITE PLAN 3 | PROJECT 33392
PINE STRAWBERRY WATER IMPROVEMENT DISTRICT - SV3 WATERLINE IMPROVEMENTS
APPROXIMATELY 2,300 LF (0.44 MILES) OF WATER TRANSMISSION LINE
NEAR THE INTERSECTION OF AZ-87/AZ-260 AND FR708
STRAWBERRY, ARIZONA 85544

TEST BORING LOCATION
SEISMIC SURVEY LOCATION

Depth	Graphic	Lithologic Description	Notes	Groundwater	Samples	Blow Counts	Standard Penetration Test (SPT) 10 20 30 40 50
		Existing site grade					
0		ASPHALT (3 inches) 0.25 ft					
		ABC (4 inches) 0.58 ft					
1		Spread Fill (5 inches), damp, 10% gravel, 25% sand, 65% fines, poorly graded, subangular coarse-grained particles, stiff, PI of 16-18, non-cemented 1 ft					
2		Sandy Clay, some gravel (CL), damp, 10% gravel, 25% sand, 65% fines, poorly graded, subangular coarse-grained particles, very stiff, PI of 17, non-cemented			▽	10 14 15	29
4					■		
5		5 ft					
		Test Boring discontinued at 5 ft					
6							
7							
8							
9							
10							

Drill Date: **12-17-2025** | Drilled By: **VEI** | Logged By: **JD** | Drill: **CME55** | Client: **Sunrise Engineering**

Depth	Graphic	Lithologic Description	Notes	Groundwater	Samples	Blow Counts	Standard Penetration Test (SPT) 10 20 30 40 50
		Existing site grade					
0		ASPHALT (3 inches) — 0.25 ft					
		ABC (3 inches) — 0.5 ft					
		Spread Fill (5 inches), damp, 10% gravel, 50% sand, 40% fines, poorly graded, subangular coarse-grained particles, stiff, PI of 12-14, non-cemented — 0.92 ft					
1		Clayey Sand, some gravel (SC), damp, 10% gravel, 50% sand, 40% fines, poorly graded, subangular coarse-grained particles, medium dense, PI of 12, non-cemented				7 15 15	
2					▽		30
3							
4					■		
5		5 ft Test Boring discontinued at 5 ft					
6							
7							
8							
9							
10							

Drill Date: **12-17-2025** | Drilled By: **VEI** | Logged By: **JD** | Drill: **CME55** | Client: **Sunrise Engineering**

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Depth	Graphic	Lithologic Description	Notes	Groundwater	Samples	Blow Counts	Standard Penetration Test (SPT) 10 20 30 40 50
		Existing site grade					
0		ASPHALT (2.5 inches) — 0.21 ft					
		ABC (3 inches) — 0.46 ft					
		Spread Fill (5 inches), damp, 10% gravel, 25% sand, 65% fines, poorly graded, subangular coarse-grained particles, stiff, PI of 10-12, non-cemented — 0.88 ft					
1		Sandy Clay, some gravel (CL), damp, 10% gravel, 20% sand, 70% fines, poorly graded, subangular coarse-grained particles, very stiff, PI of 11, non-cemented				3 3 3	
2					▽		6
4					■		
5		5 ft — Test Boring discontinued at 5 ft					
6							
7							
8							
9							
10							

Drill Date: **12-17-2025** | Drilled By: **VEI** | Logged By: **JD** | Drill: **CME55** | Client: **Sunrise Engineering**

RSLog / VEI Boring Log 10' / vann / admin / December 29, 2025 03:54 PM

Depth	Graphic	Lithologic Description	Notes	Groundwater	Samples	Blow Counts	Standard Penetration Test (SPT) 10 20 30 40 50
		Existing site grade					
0		ASPHALT (3 inches) — 0.25 ft					
		ABC (4 inches) — 0.46 ft					
		Spread Fill (5 inches), damp, 10% gravel, 25% sand, 65% fines, poorly graded, subangular coarse-grained particles, stiff, PI of 18-20, non-cemented — 0.83 ft					
1		Sandy Clay, some gravel (CL), damp, 10% gravel, 30% sand, 60% fines, poorly graded, subangular coarse-grained particles, very stiff, PI of 19, non-cemented				12 20 21	
2					▽		● 41
3							
4					■		
5		5 ft — Test Boring discontinued at 5 ft					
6							
7							
8							
9							
10							

Drill Date: **12-17-2025** | Drilled By: **VEI** | Logged By: **JD** | Drill: **CME55** | Client: **Sunrise Engineering**

RSLog / VEI Boring Log 10' / vann / admin / December 29, 2025 03:54 PM

Depth	Graphic	Lithologic Description	Notes	Groundwater	Samples	Blow Counts	Standard Penetration Test (SPT) 10 20 30 40 50
		Existing site grade					
0		ASPHALT (3.5 inches) 0.29 ft					
		ABC (5 inches) 0.71 ft					
1		Spread Fill (4 inches), damp, 25% sand, 75% fines, poorly graded, subangular coarse-grained particles, stiff, PI of 4-7, non-cemented 1.04 ft					
		Silty Sandy Clay (CL-ML), damp, 30% sand, 70% fines, poorly graded, subangular coarse-grained particles, very stiff, PI of 4, non-cemented				5 12 21	
2					▽		33
3							
4					■		
5		5 ft					
		Test Boring discontinued at 5 ft					
6							
7							
8							
9							
10							

Drill Date: **12-17-2025**	Drilled By: **VEI**	Logged By: **JD**	Drill: **CME55**	Client: **Sunrise Engineering**

RSLog / VEI Boring Log 10' / vann / admin / December 29, 2025 03:54 PM

VELOCITY CLASSIFICATION DATA

Pine Strawberry Water Improvement District – SV3 Waterline Improvements
Approximately 2,300 LF (0.44 miles) of Water Transmission Line
Near the Intersection of AZ-87/AZ-260 and FR708 (Near N34º24’15.03”, W111º29’58.43”)
Strawberry, Arizona 85544

Average Velocity of Layer 1: 980 fps (873 to 1042)

Average Velocity of Layer 2: 1858 fps (1626 to 2090)

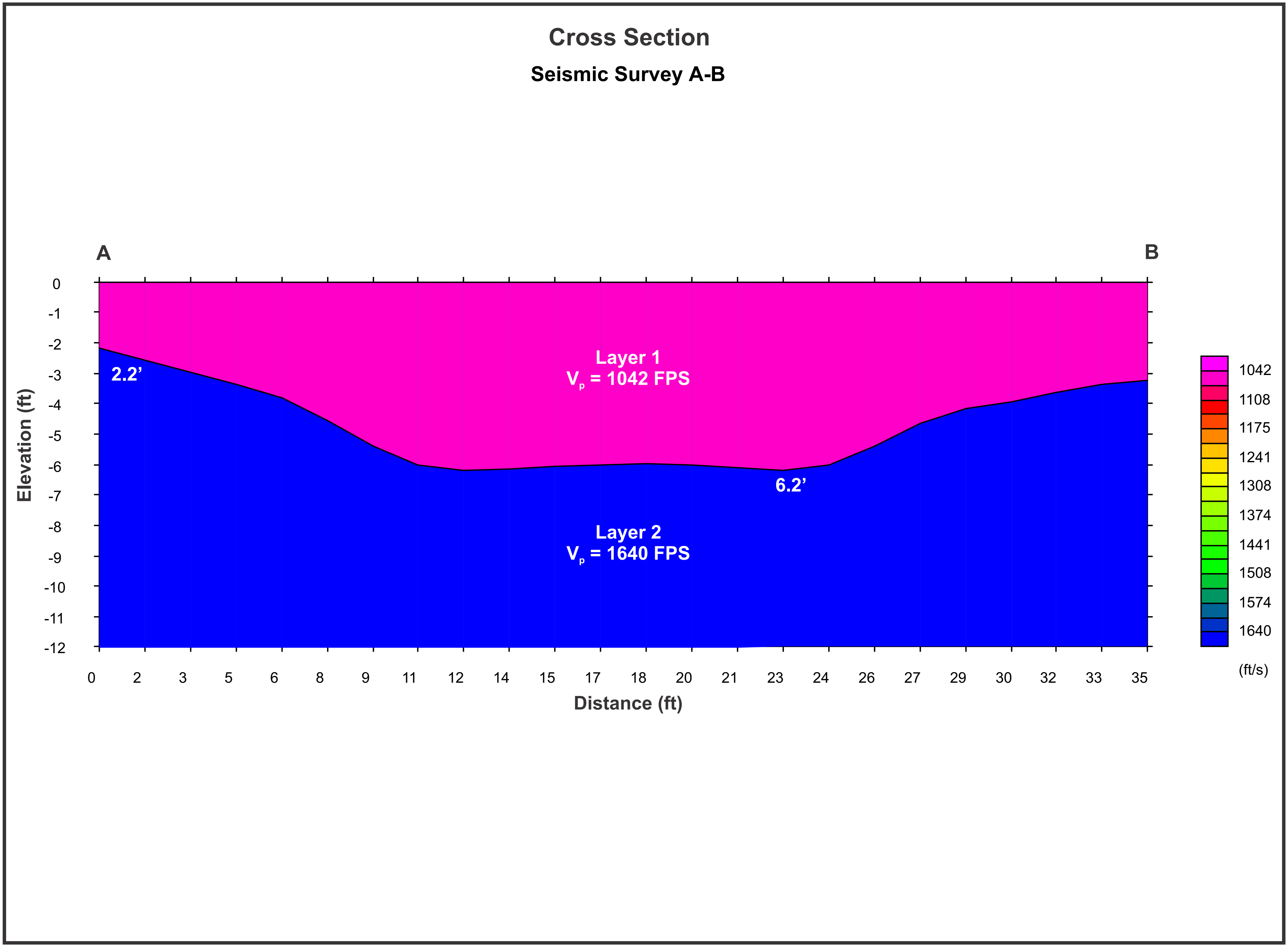
Average Depth to Layer 2: 3.6 feet

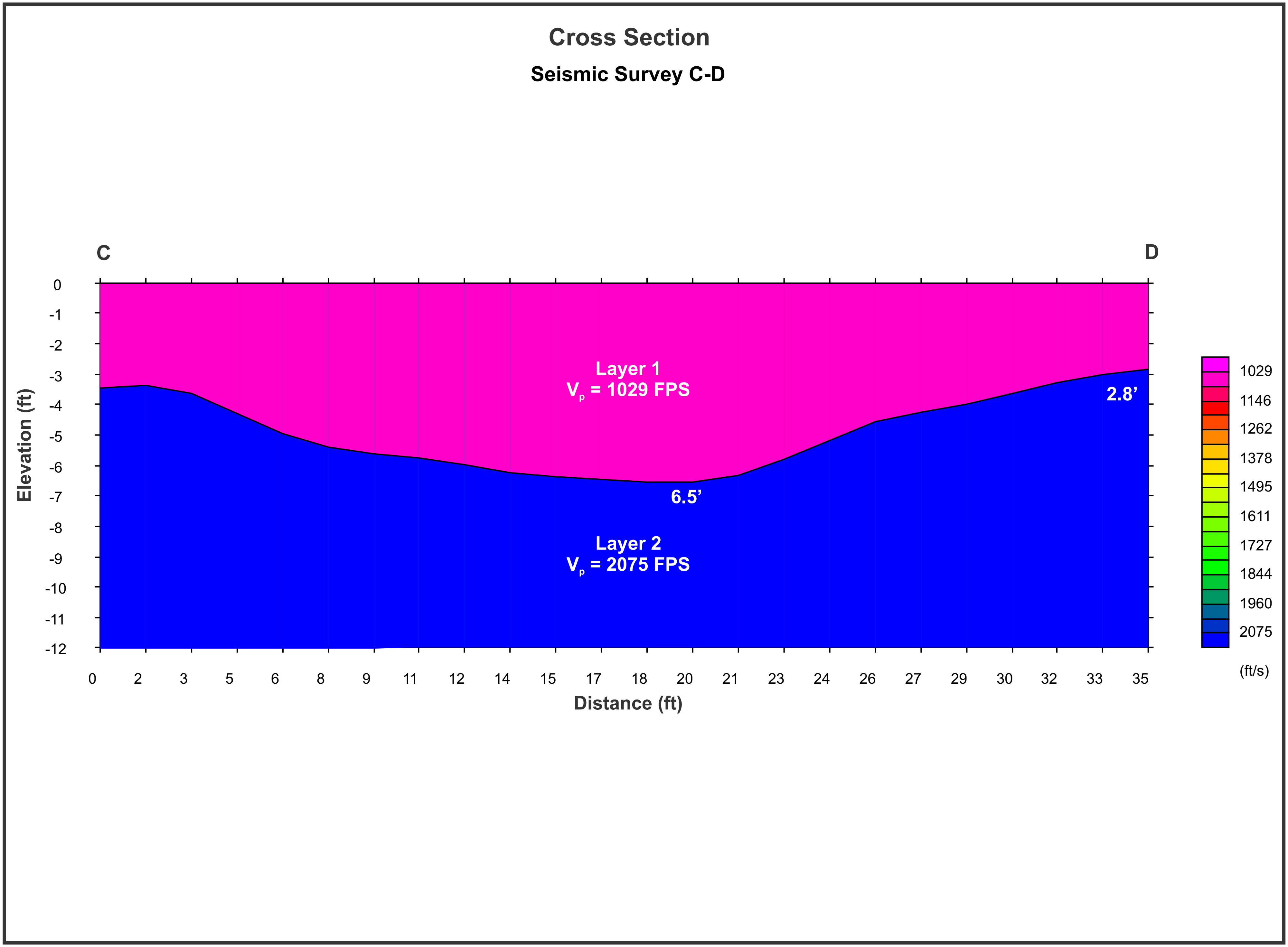
Range: 1.5 to 6.5 feet

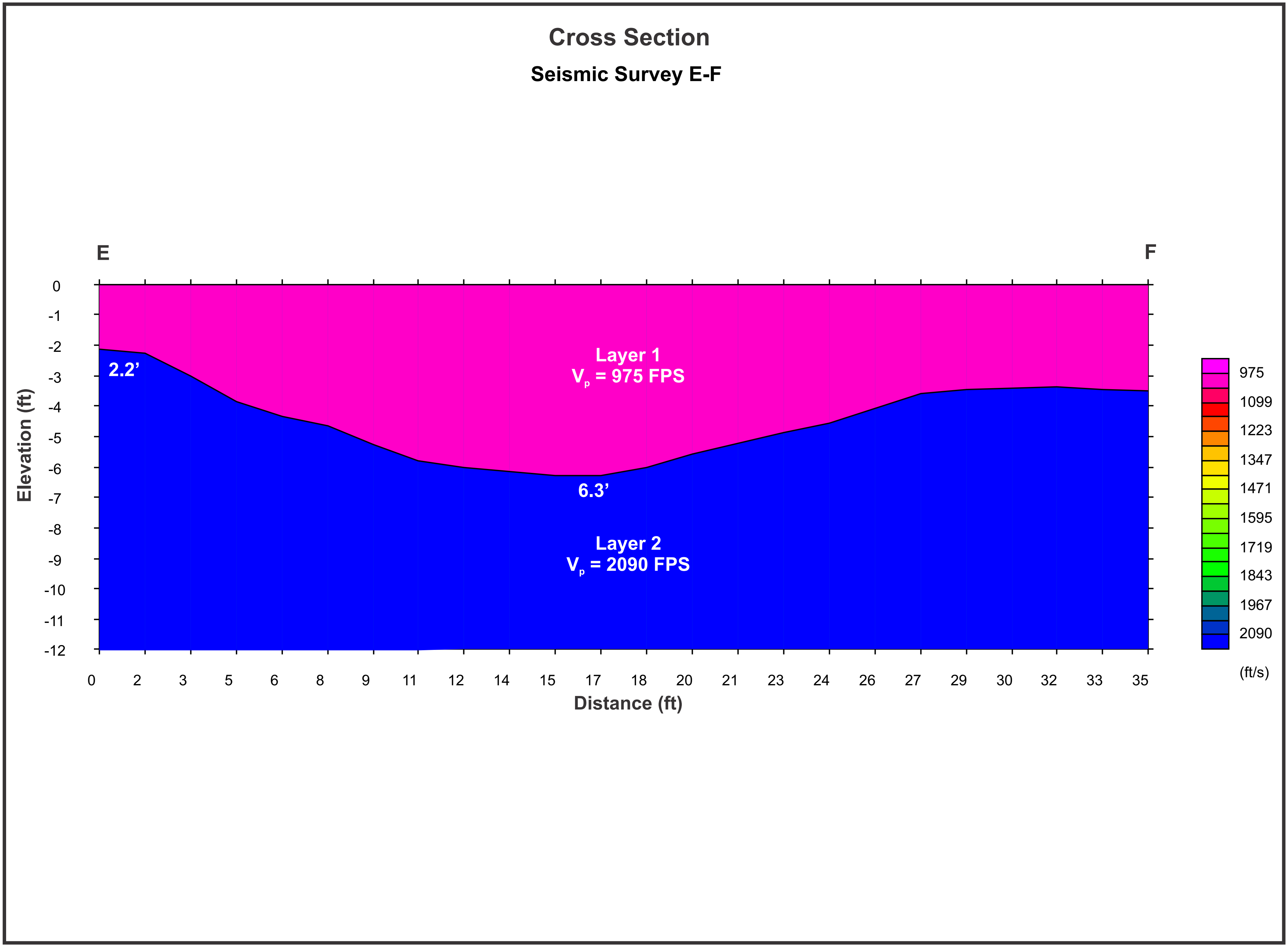
Layer 1: Firm clay

Layer 2: Very stiff to hard clay

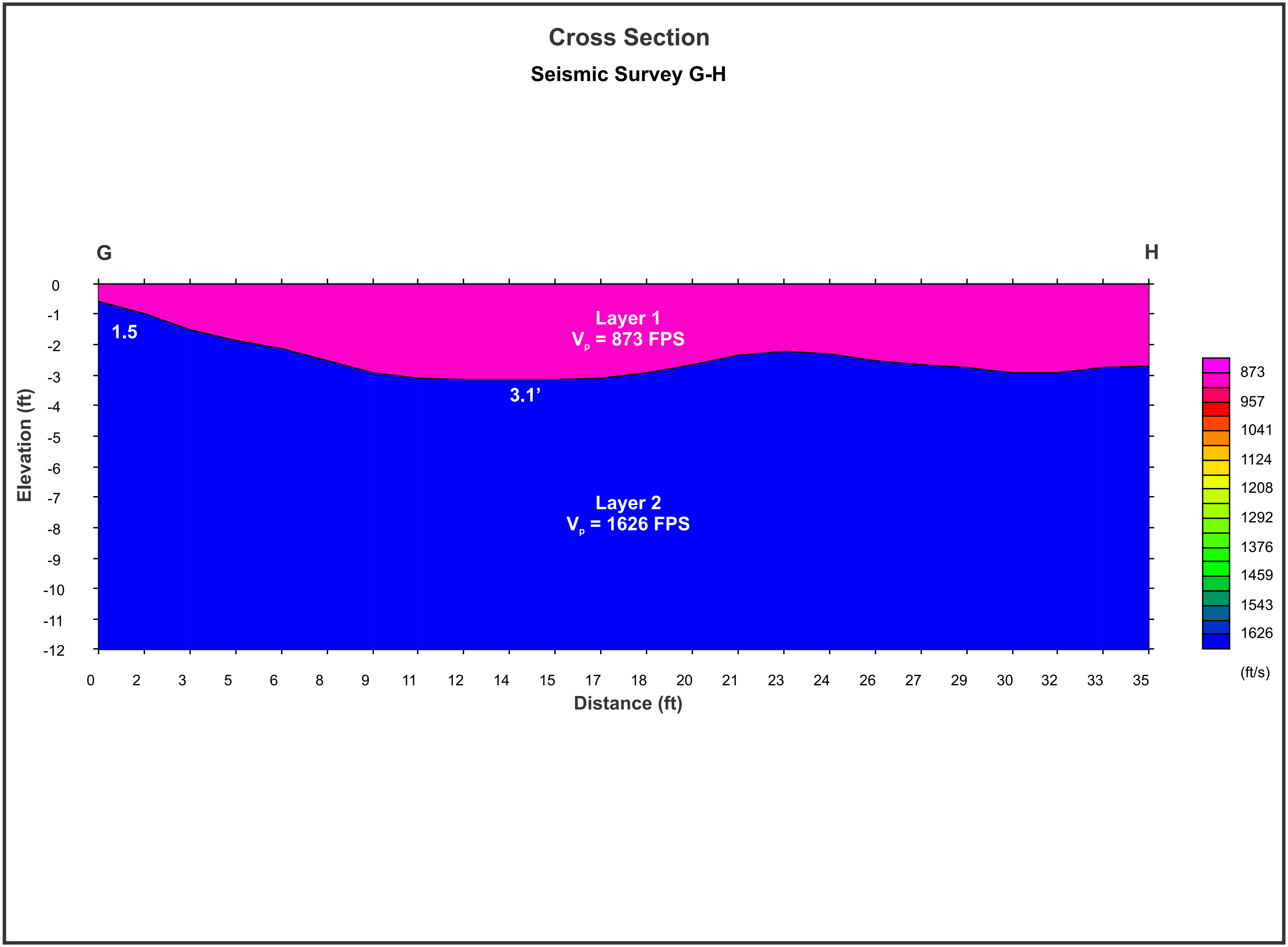
Line	Layer 1			Layer 2		
	Velocity	Depth (ft)		Velocity	Depth (ft)	
A - B	1042	-	-	1640	2.2	6.2
C - D	1029	-	-	2075	2.9	6.5
E - F	975	-	-	2090	2.2	6.3
G - H	873	-	-	1626	1.5	3.1
TB-1	-	-	-	-	-	5.0
TB-2	-	-	-	-	-	5.0
TB-3	-	-	-	-	-	3.0
TB-4	-	-	-	-	-	1.5
TB-5	-	-	-	-	-	1.5
Averages	980	-		1858	3.6	


Cross Section
Seismic Survey A-B
A
B
Layer 1
V_p = 1042 FPS
2.2'
6.2'
Layer 2
V_p = 1640 FPS
Elevation (ft)
Distance (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
0 2 3 5 6 8 9 11 12 14 15 17 18 20 21 23 24 26 27 29 30 32 33 35
1042
1108
1175
1241
1308
1374
1441
1508
1574
1640
(ft/s)


Cross Section
Seismic Survey C-D
C
D
Layer 1
Vp = 1029 FPS
2.8'
6.5'
Layer 2
Vp = 2075 FPS
Elevation (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
0 2 3 5 6 8 9 11 12 14 15 17 18 20 21 23 24 26 27 29 30 32 33 35
Distance (ft)
1029
1146
1262
1378
1495
1611
1727
1844
1960
2075
(ft/s)



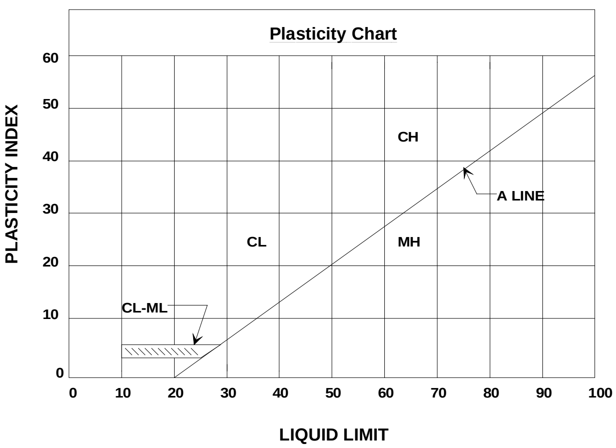
Cross Section
Seismic Survey E-F
E
F
Layer 1
V_p = 975 FPS
2.2'
6.3'
Layer 2
V_p = 2090 FPS
Elevation (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
0 2 3 5 6 8 9 11 12 14 15 17 18 20 21 23 24 26 27 29 30 32 33 35
Distance (ft)
975
1099
1223
1347
1471
1595
1719
1843
1967
2090
(ft/s)


Cross Section
Seismic Survey G-H
G
H
Layer 1
Vp = 873 FPS
Layer 2
Vp = 1626 FPS
1.5
3.1’
Elevation (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
0 2 3 5 6 8 9 11 12 14 15 17 18 20 21 23 24 26 27 29 30 32 33 35
Distance (ft)
873
957
1041
1124
1208
1292
1376
1459
1543
1626
(ft/s)

LEGEND

Major Divisions				Group Symbol	Typical Names
Coarse-Grained Soils (Less than 50% passes No. 200 sieve)	Gravels (50% or less or coarse fraction passes No. 4 sieve)	Clean Gravels (Less than 5% passes No. 200 sieve)		GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
				GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
		Gravels with Fines (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on Plasticity Chart.	GM	Silty gravels, gravel-sand-silt mixtures.
			Limits plots above "A" line & hatched zone on Plasticity Chart.	GC	Clayey gravels, gravel-sand-clay mixtures.
	Sands (More than 50% of coarse fraction passes No. 4 sieve)	Clean Sands (Less than 5% passes No. 200 sieve)		SW	Well graded sands, gravelly sands.
				SP	Poorly graded sands, gravelly sands.
		Sands with Fines (More than 12% passes No. 200 sieve)	Limits plots below "A" line & hatched zone on Plasticity Chart.	SM	Silty sands, sand-silt mixtures.
			Limits plots above "A" line & hatched zone on Plasticity Chart.	SC	Clayey sands, sand-clay mixtures.
Fine-Grained Soils (50% or more passes No. 200 sieve)	Silts-Plot below "A" line & hatched zone on Plasticity Chart	Silts of Low Plasticity (Liquid Limit Less Than 50)		ML	Inorganic silts, clayey silts with slight plasticity.
		Silts of High Plasticity (Liquid Limit More Than 50)		MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
	Clays-Plot above "A" line & hatched zone on Plasticity Chart	Clays of Low Plasticity (Liquid Limit Less Than 50)		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		Clays of High Plasticity (Liquid Limit More Than 50)		CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity.

Note: Coarse grained soils with between 5% & 12% passing the No. 200 sieve and fine grained soils with limits plotting in the hatched zone on the Plasticity Chart to have double symbol.



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Cobbles	Above 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to 3/4 in.
Fine gravel	3/4 in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

TEST DRILLING EQUIPMENT & PROCEDURES

Drilling Equipment

VANN ENGINEERING INC uses a CME-55 drill-rig capable of auger drilling to depths of 150 feet in southwestern soils. The drill is truck-mounted for rapid, low cost mobilization to the jobsite and on the jobsite. The CME-55 owned by this firm is powered by a 300 cubic inch, 6-cylinder Ford industrial engine that produces 124 horsepower. This energy is transmitted through a rugged mechanical drive that provides 7,000 foot-lbs of torque on the drillstring. Two 72-inch hydraulic cylinders develop 16,000 lbs of downward thrust and 24,000 lbs of retractive force. Two hydraulic cable hoists and a mechanical cathead allow downhole sampling and testing at any depth to be accomplished with great speed and accuracy. For drilling operations, the truck is stabilized with platform mounted vertical hydraulic jacks with a 48-inch stroke. Drilling through soil or softer rock is performed with 6¾ inch O.D. hollow-stem, or 4½-inch continuous flight auger. Carbide insert teeth are normally used on the auger bits so they can often penetrate rock or very strongly cemented soils that require blasting or very heavy equipment for excavation. The operation of well-maintained equipment by an experienced crew allows VANN ENGINEERING INC to complete any type of drilling job with minimum downtime and maximum efficiency.

Sampling Procedures

Dynamically driven tube samples are usually obtained at selected intervals in the borings by the ASTM D1586 procedure. In many cases, 2 inch O.D., $1^3/_8$-inch I.D. samplers are used to obtain the standard penetration resistance. "Undisturbed" samples of firmer soils are often obtained with 3-inch O.D. samplers lined with 2.42 inch I.D. brass rings. The driving energy is generally recorded as a number of blows of a 140-pound hammer, utilizing a 30-inch free fall drop, per foot of penetration. However, in stratified soils, driving resistance is sometimes recorded in 2 or 3-inch increments so that soil changes and the presence of scattered gravel or cemented layers can be readily detected and the realistic penetration values obtained for consideration in design. These values are expressed in blows per foot on the logs. Undisturbed sampling of softer soils is sometimes performed with thin-walled Shelby tubes (ASTM D1587). Tube samples are labeled and placed in watertight containers to maintain field moisture contents for testing from auger cuttings.

Continuous Penetration Tests

Continuous penetration tests are performed by driving a 2-inch O.D. blunt nosed penetrometer adjacent to or in the bottom of test borings. The penetrometer is attached to $1^5/_8$-inch O.D. drill rods to provide clearance and thus minimize side friction so that penetration values are as nearly as possible a measure of end resistance. Penetration values are recorded as the number of blows of a 140 pound hammer, utilizing a 30-inch drop required to advance the penetrometer in one foot increments or less.

As an alternate, Cone Penetration Testing may be utilized in an effort to determine the point capacity of the cone tip, and skin friction measured on the cone sleeve.

Boring Records

Drilling operations are directed by our field engineer or geologist who examines soil recovery and prepares boring logs. Soils are visually classified in accordance with the Unified Soil Classification System (ASTM D2487) with appropriate group symbols being shown on the logs.

INTRODUCTION TO SEISMIC REFRACTION PRINCIPLES

Any disturbance to a soil or rock mass creates seismic waves which are merely the propagation of energy into that mass, manifested by distinct waveforms. There are two basic types of seismic waves; body waves and surface waves.

Body waves are either compressional or shear in nature, they penetrate deep into the substrata, and reflect from or refract through the various geologic layers. Any emission of an energy source into a medium exhibits both a compression wave (P Wave) and a shear wave (S Wave). P-Waves propagate in the form of oscillating pulses, traveling forward and backward, parallel to the direction of the wave front. S-Waves propagate in the form of distortional pulses, oscillating perpendicular to the wave front.

P-Waves travel at the highest velocities. Recording instruments that detect an energy transmission will generally observe the arrival of the P-Wave, followed by the S-Wave and surface waves.

All geologic materials exhibit P-Wave velocities in certain ranges, which relate to the density, specific gravity, elastic modulus, and moisture content of the specific material. As a material density and specific gravity increase so does its P-Wave velocity. Similarly, an increase in moisture content will cause an increase in P-Wave velocity. Generally, materials exhibiting higher P-Wave velocities will display higher elastic moduli.

In keeping with this relationship, determining the P-Wave velocities for the various subsurface layers, may yield very important and useful data relative to the engineering properties of the individual layers. In order to accomplish this task, methods of investigation, or surveys, were developed to establish the P-Wave velocity for subsurface layers. The method adopted by the VANN ENGINEERING INC Geophysical team examines the layer velocities, through refraction theory. Assuming that a P-Wave will refract through the various layers, according to the angle of incidence of the propagating wave form and the medium it is traveling through, it is then possible to detect a contrasting subsurface stratum by changes in the velocity of an induced seismic wave.

The procedure is outlined as follows:

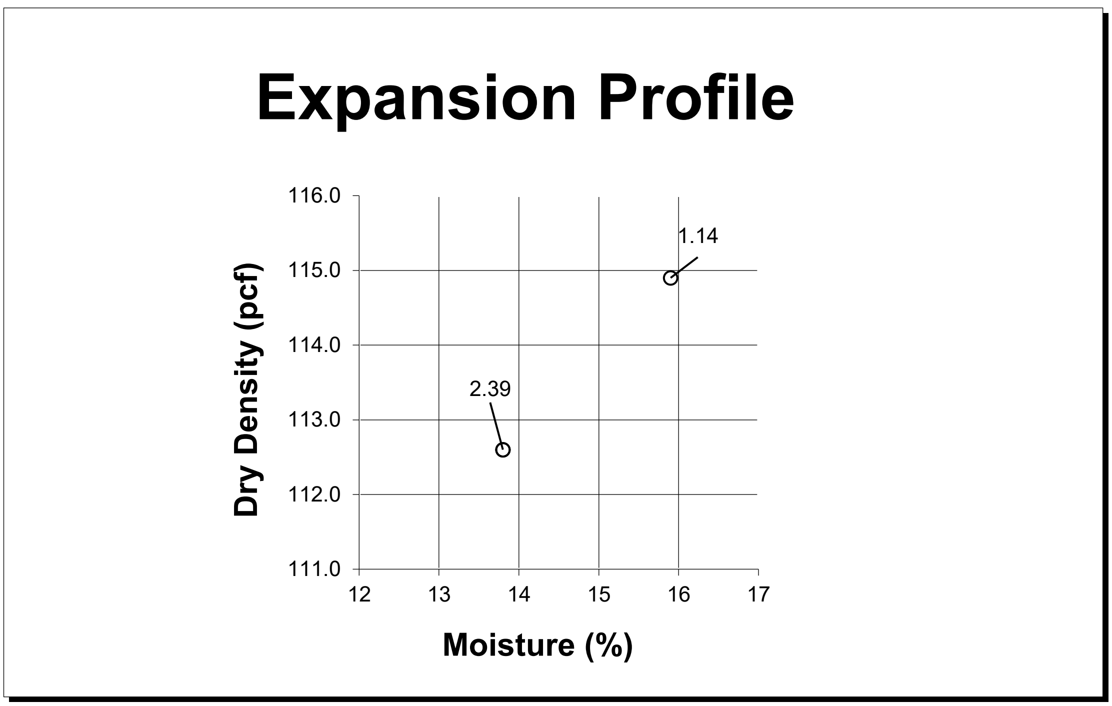
A geophone is inserted into the ground or on a rock surface. Attached to it is a recording device. At predetermined intervals away from the geophone, in a linear array, a heavy sledgehammer strikes a stable plate or rock surface. Typically, the intervals of successive hammer impacts range from five to twenty feet. A timing device attached to the hammer, trips a measured recording sweep time, at the moment of impact. The arrival time of the induced P-Wave is measured and recorded at each interval. The length of a survey is closely related to the depth of investigation. Generally, the depth of investigation is approximately equal to one-third the length of the survey. For example, if it is desired to examine the substrata to a depth of twenty feet, the survey should extend a distance of at least sixty feet. Changes in the calculated velocity indicate strata breaks or distinct changes within the same stratum. The important concept to remember with this method is that it is predominantly effective where velocities increase from layer to layer, moving downward from the surface. Analytical methods are also available for determining the depth to the various layers, even in the most complex multi-layer situations

SECTION III

VANN ENGINEERING, INC.
GEOTECHNICAL ENGINEERING | ENVIRONMENTAL CONSULTING | CONSTRUCTION TESTING & OBSERVATION
PO Box 35487 | Phoenix, Arizona 85069-5487

Pine Strawberry Water Improvement District-SV3 Waterline Improvements
Approximately 2,300 LF (0.44 miles) of Water Transmission Line
Near the Intersection of AZ-87/AZ-260 and FR708
Strawberry, Arizona 85544

Sample Location	Remolded Moisture Content (%)	Dry Density (PCF)	Volume Change After Saturation (%)	Adjusted Volume Change After Saturation (%)
TB-1 (4.0'-5.0')	15.9	114.9	1.27	1.14
TB-1 (4.0'-5.0')	13.8	112.6	2.65	2.39
Percent #4 Retained		10		



Project 33392

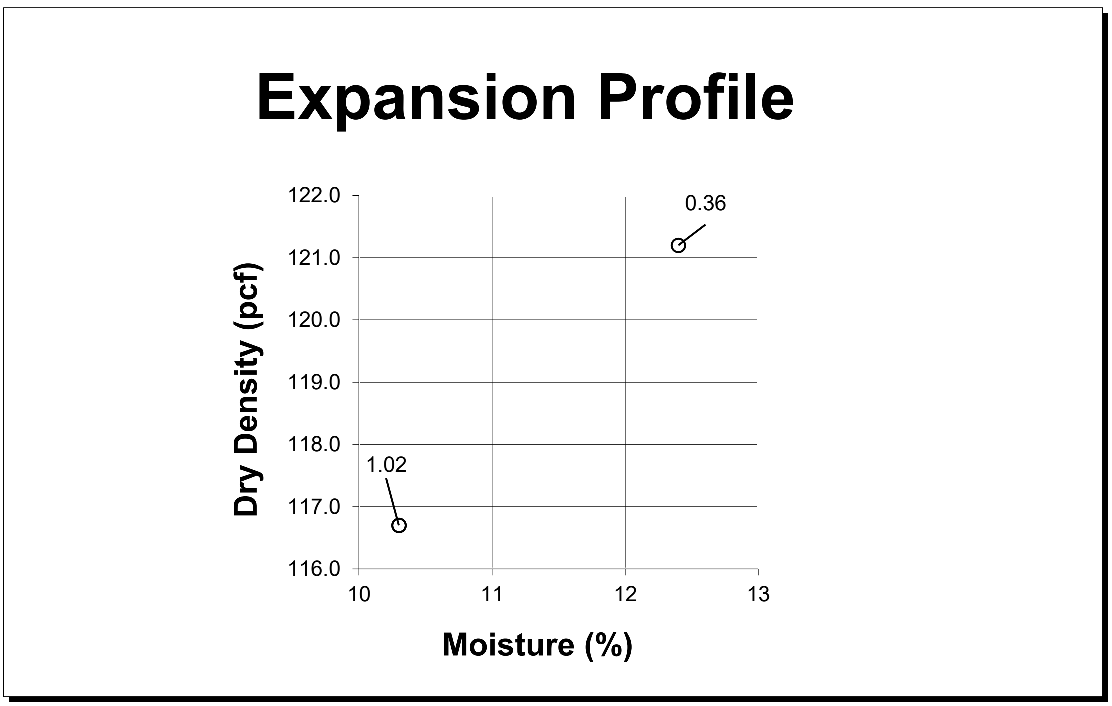
VANN ENGINEERING, INC.

GEOTECHNICAL ENGINEERING | ENVIRONMENTAL CONSULTING | CONSTRUCTION TESTING & OBSERVATION

PO Box 35487 | Phoenix, Arizona 85069-5487

Pine Strawberry Water Improvement District-SV3 Waterline Improvements
Approximately 2,300 LF (0.44 miles) of Water Transmission Line
Near the Intersection of AZ-87/AZ-260 and FR708
Strawberry, Arizona 85544

Sample Location	*Remolded Moisture Content (%)*	*Dry Density (PCF)*	*Volume Change After Saturation (%)*	*Adjusted Volume Change After Saturation (%)*
TB-2 (4.0'-5.0')	12.4	121.2	0.39	0.36
TB-2 (4.0'-5.0')	10.3	116.7	1.11	1.02
Percent #4 Retained		8		



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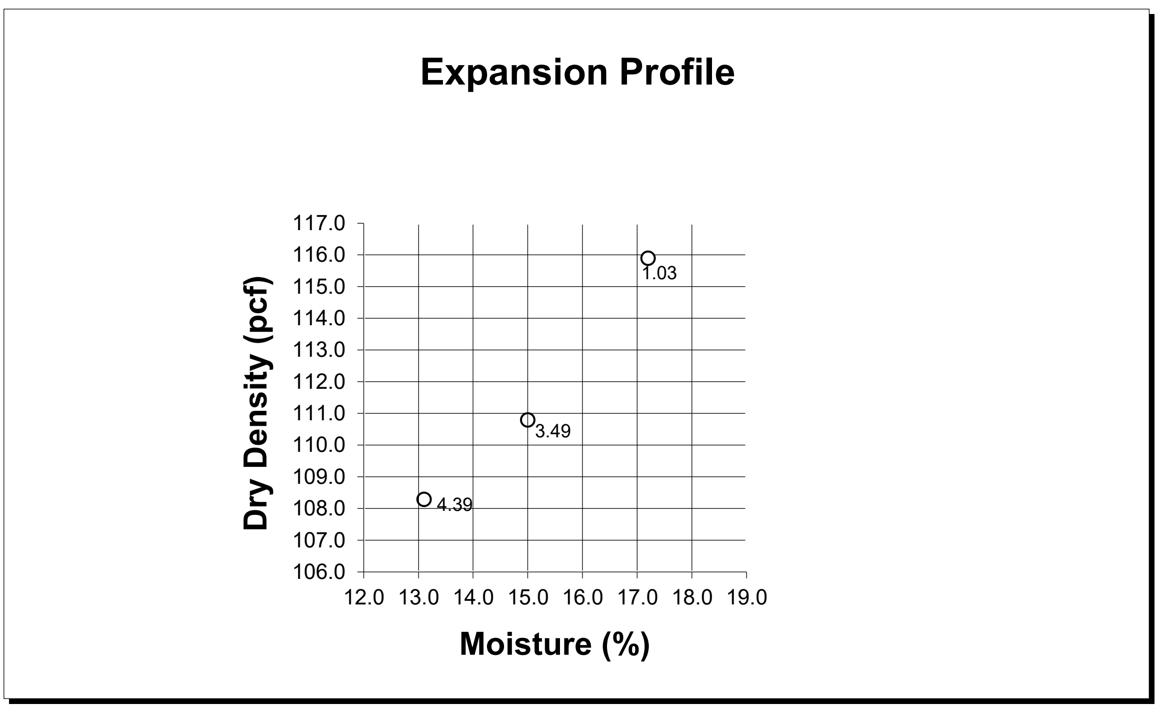
Sample Location	Remolded Moisture Content (%)	Dry Density (PCF)	Volume Change After Saturation (%)	Adjusted Volume Change After Saturation (%)
TB-4 (4.0'-5.0')	17.2	115.9	1.13	1.03
TB-4 (4.0'-5.0')	15.0	110.8	3.83	3.49
TB-4 (4.0'-5.0')	13.1	108.3	4.82	4.39
Percent #4 Retained		9		



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CLASSIFICATION TEST DATA

Pine Strawberry Water Improvement District-SV3 Waterline Improvements
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Sample Location	*Sieve Analysis (% Passing Sieve Size)* 3”	2”	1”	#4	#10	#40	#100	#200	*Atterberg Limits* LL	PI	USCS	*Moisture Content %*
TB-1 (4.0’-5.0’)	-	100	100	90	83	75	-	64	33	17	CL	8.3
TB-2 (4.0’-5.0’)	-	100	100	92	85	79	-	39	31	12	SC	6.0
TB-3 (4.0’-5.0’)	-	100	100	91	85	78	-	68	30	11	CL	9.2
TB-4 (4.0’-5.0’)	-	100	100	91	84	77	-	61	34	19	CL	6.4
TB-5 (4.0’-5.0’)	-	100	100	99	97	94	-	72	22	4	CL-ML	5.3

SULFATES AND CHLORIDES TEST RESULTS

Pine Strawberry Water Improvement District-SV3 Waterline Improvements
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Sample Location	*Test Interval (feet)*	*Sulfate (%)*	*Chloride (ppm)*
TB-2	4.0-5.0	0.133	12

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