VANN
ENGINEERING INC

Geotechnical Investigation Report

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY, SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Prepared for:

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May 4, 2020

Project 27266

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Vann Engineering Inc.

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May 4, 2020 Project 27266

Andrew Baird
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201 North Montezuma, Suite 206
Prescott, Arizona 86301

RE: GEOTECHNICAL INVESTIGATION REPORT
RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY, SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Andrew:

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 throughout the zone of significant foundation influence and cut slope / retaining wall recommendations. 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 he 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 the course of 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 5/4/2020
ARIZONA, U. S. A.

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TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION I

1.0 INTRODUCTION

Vann Engineering, Inc. understands that an extension to Forest Road as it will connect to Highway 89A, northwest of the intersection of Highway 89A and Highway 179 is proposed for construction. It is this firm's understanding there is a potential for cuts up to 30.0 feet and fills up to 15.0 feet. This document presents the results of a geotechnical investigation conducted by Vann Engineering, Inc.

**RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY, SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA**

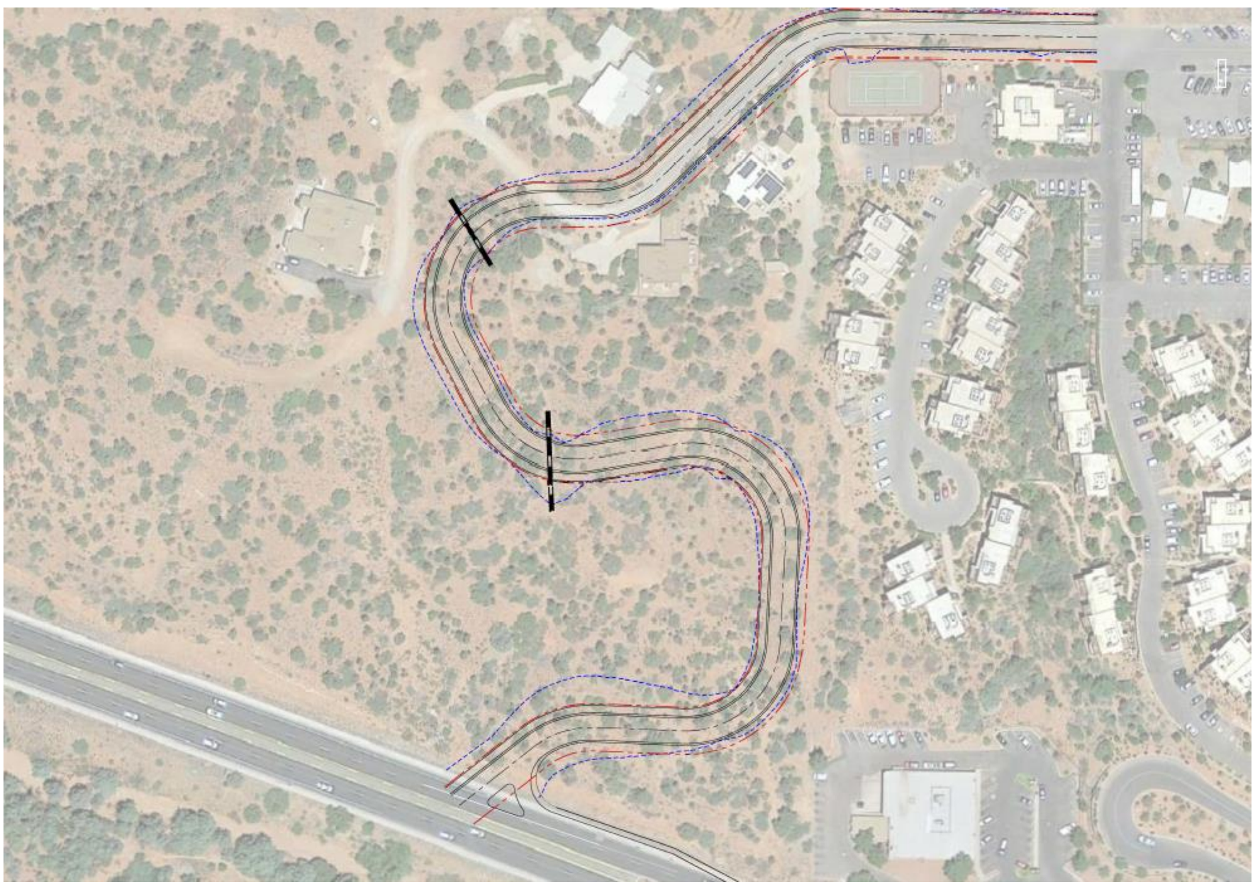


Figure 1: Aerial photograph and over lay of the proposed construction

The services performed provide an evaluation at selected locations of the subsurface soil conditions throughout the zone of significant foundation influence.

1.1 Purpose

The purpose of the investigation was two-fold: 1) to determine the physical characteristics of the soil underlying the location of the proposed retaining walls, and 2) to provide final recommendations for safe and economical foundation design.

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 layers, i.e. depth to weathered rock and more competent rock
- Site Plan indicating the locations of all points of exploration
- General excavation conditions (rippability); equipment required for each layer encountered
- IBC Site Classification for years up through 2018
- Recommendations for retaining wall design, including bearing capacity, passive resistance, base friction, active stress and potential at-rest stress – sloping backslopes will also be considered in our recommendations
- Recommendations for rockery walls
- Recommendations for safe cut slopes
- Other potential design options as the soil and rock conditions dictate
- Recommendation for pavement design (off-site)

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. Recommendations for basement-level facilities or scour have not been included in our scope of services.

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 revised proposal **(VE19GT1009AC dated 10/9/19, and the Standard Agreement for Professional Services Between Kimley-Horn and Associates, Inc. and Vann Engineering, Inc.)**, to proceed with the work. Our efforts and report are limited to the scope and limitations 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 and 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, express 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 an extension to Forest Road as it will connect to Highway 89A, northwest of the intersection of Highway 89A and Highway 179 is proposed for construction. It is this firm's understanding there is a potential for cuts up to 30.0 feet and fills up to 15.0 feet.

2.2 Site Description

The subject site is a vacant hillside area, sloping down to the south, located north of Highway 89A. The subject site is moderately to heavily vegetated with large trees and bushes. At the location on TB-1, 2.0 inches overlying 9.0 inches of compacted subgrade were encountered. At the location on TB-2, 6.0 inches of compacted subgrade were encountered. Over-sized aggregate (greater than 3.0 inches), small-sized boulders, and rock outcrops were encountered across the site surface. Please note that significant over-sized aggregate (greater than 3.0 inches) will exist within the subsurface soil layer and must not be used as structural fill. Refer to the following photographs which depict the site conditions at the time of the field effort.

Figure 2: General site conditions

Figure 3: General site conditions showing the presence of over-sized aggregate (greater than 3.0 inches)

Figure 4: General site conditions showing the presence of over-sized aggregate (greater than 3.0 inches)

Figure 5: General site conditions showing small boulders and outcrops

Figure 6: General site conditions at the location of TB-2

Figure 7: Cut slope at the base of the site – North side of Hwy 89A (Psm – Permian Supai Group)

3.0 SUBSURFACE INVESTIGATION AND LABORATORY TESTING

3.1 Subsurface Investigation

The sites subsurface was explored through the utilization of three (3) test borings to depths ranging from 3.5 to 4.0 feet. A depth shallower than 15.0 feet corresponds to the depth to shallow auger refusal on moderately weathered and fractured sandstone rock. The locations of the test borings are shown on the Site Plan in Section II of this report and are presented as TB-1 through TB-3.

The site's subsurface was also explored through the utilization of six (6) 24-channel refraction seismic survey lines, denoted on the Site Plan in Section II of this report (along the proposed bridge location). The seismic survey lines involved the retrieval of data in two separate directions (*forward and reverse*). As such, twelve (12) refraction seismic surveys were conducted at the site. The seismic survey lengths were 72.0 feet, thereby allowing an examination of the subsurface to a depth of 28.0 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 I 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 harder 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. Thus, the Cross Sections presented herein may not reveal a possible weaker underlying layer, within or below the depicted layers. 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. For example, if it is desired to examine the substrata to a depth of 20.0 feet, the survey should extend a distance of 60.0 feet. However, 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. Thus, the actual seismic survey exploration depths were 28.0 feet below the existing grade, regardless of the length of the survey lines.

The materials encountered on the subject 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 are based on the assumption 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.

3.2 Laboratory Testing

Laboratory analyses were performed on representative soil samples 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 1: Laboratory Testing

Test	Sample(s)	Purpose
Response to Wetting, Moisture Content, and Dry Density	Undisturbed native Soils (3)	Foundation bearing capacity and settlement analysis
Expansion	Native subgrade soils (8)	Potential for heave upon wetting
Sieve Analysis, Atterberg Limits, & Moisture Content	Native subgrade soils (4)	Soil classification

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 Local / Regional Geology

The rock in the area is sandstone. The following snip of a geologic map shows the pervasive occurrence of the sandstone.

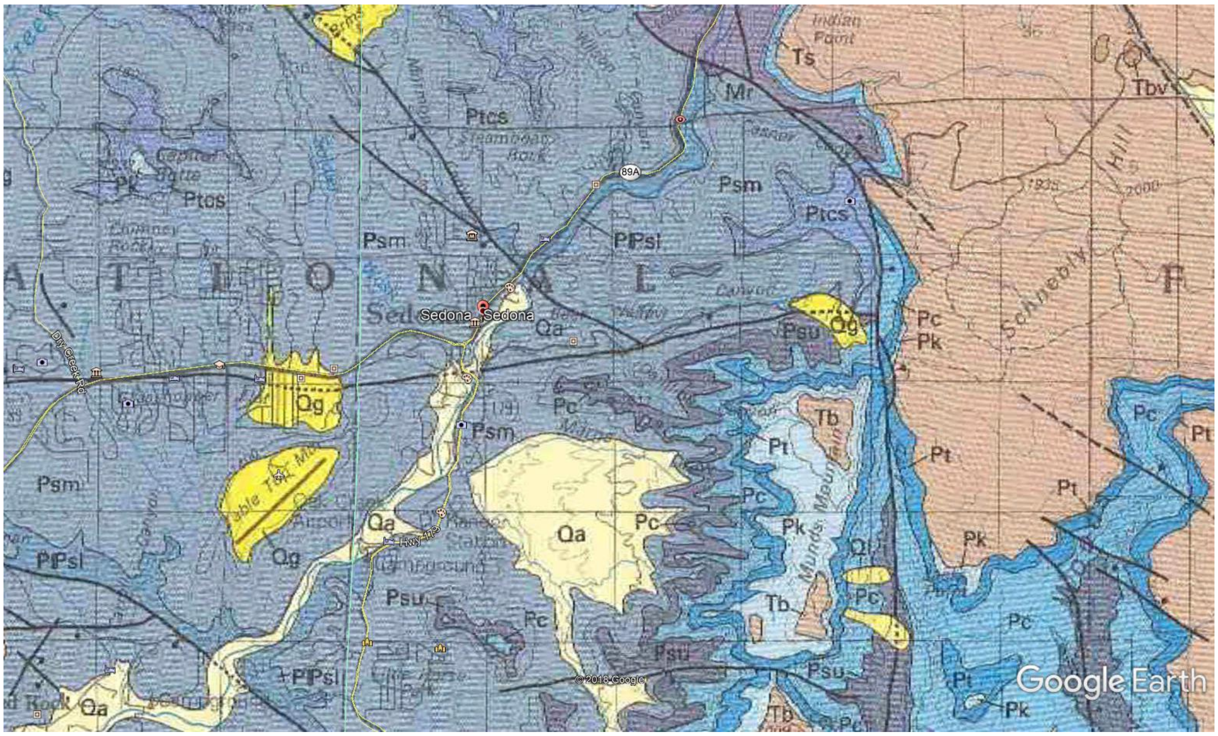


Figure 8: Local geology

The rock in close proximity to the site (particularly to the northwest of 89A) is denoted by the classification of Psm. An additional snip provides a useful classification of the rocks associated with the Psm designation.

Psm

Middle part (Lower Permian)—Siltstone, sandstone, conglomerate, and minor dolomitic limestone. Siltstone, sandy to clayey siltstone and sandstone, silty, very fine to fine grained, both grayish red and reddish brown, quartzose, in part micaceous, commonly calcitic; mostly in obscure sets of laminae and thin to medium horizontal beds. Conglomerate, light gray to grayish red, composed of pebbles and cobbles of limestone and calcitic mudstone in a matrix of calcitic siltstone and sandstone; in lenses, commonly 30–300 m (100–1,000 ft) long and 1.5–8 m (5–25 ft) thick, interstratified with siltstone and sandstone in basal one-third of map unit. Limestone and dolomitic limestone, light-gray to medium-gray, fine-grained; in rare thin beds in upper part of unit. Forms moderate to steep slope, studded irregularly with discontinuous ledges; equivalent to middle member and lower part of upper member of the Supai Formation of Huddle and Dobrovolny (1945). About 210–300 m (700–1,000 ft) thick

Figure 9: Geologic classification

4.2 Site Stratigraphy (Soil and Rock Layering)

The following is a general summary of the on-site soil and rock characteristics based on information obtained during this firm's subsurface investigation. The soil sample, test boring data, and seismic refraction data obtained from the site were analyzed and subjected to laboratory testing and computer aided analyses relative to engineering applications. The laboratory test results, test boring data, and seismic refraction data indicate the following physical and mechanical properties of the subsurface soil and rock:

Table 2: Site Stratigraphy

Layer	Velocity Range (FPS)	Depth of Occurrence[1]	Classification
1	1323 to 2809	Extends from the surface to highly variable depths ranging from 0.4 to 8.8 feet[3]	Moderately dense coarse-grained alluvial deposits consisting of silty clayey sandy gravel (GC-GM)[2]
2	4552 to 5426	Below highly variable depths ranging from 0.4 to 5.9 feet[4]	Highly to moderately weathered and fractured, poor, weak sandstone rock (Psm)
3	6058 to 7419	Below highly variable depths ranging from 0.9 to 8.8 feet[5]	Moderately weathered and fractured, fair, moderately strong sandstone rock (Psm)

[1]Average calculated depth below the existing site surface at the locations of the seismic surveys and test borings. Variations on the order of 2.0 feet may be encountered in the layer depth calculations due to the variability of the materials, degrees of weathering, and orientation of the structures.

[2]As stated herein, over-sized particles are those greater than 3 inches. It must be noted that over-sized particles (greater than 3 inches) will occur within Layer 1 and should be anticipated during the excavation process.

[3]Range applicable to all test locations.

[4]Range applicable to seismic surveys A-B, E-F, G-H, K-L, TB-1, TB-2, & TB-3.

[5]Range applicable to seismic surveys C-D & I-J.

Refer to the Test Boring logs and Seismic Cross Sections located in Section II of this report for the subsurface layering determined by our analyses. The locations of the test borings and seismic surveys are depicted on the Site Plan in Section II.

4.3 Engineering Properties of the Site Soils

Expansive soils are soils that expand or swell and are typically known to have a shrink/swell potential. Cohesive soils, or clay soils, tend to shrink as they are dried, and swell as they become wetted. The clay content of the soil determines the extent of the shrink/swell potential. The soils encountered at the site are considered <u>cohesionless</u> based on the laboratory testing (i.e.

measured plasticity indices of 4 to 10). Based on the field and laboratory test data, this firm has determined that the potential for soil expansion is low.

Collapsible soils are typically comprised of silt and sand size grains with small amounts of clay. The collapse potential of a soil depends on the in-situ density, depth of the deposit and the extent of a porous structure. When loading is applied to collapsible soils, originating from the weight of the structure, along with wetting, settlement occurs. Wetting sources are most commonly associated with landscape irrigation, inadequate surface drainage, utility line leakage, proximity of retention basins and water features to a structure, and long-term ponding next to the structure. Based on seismic survey data the soils encountered at the site are considered to have a low potential for collapse and excessive differential soil movement. However, due to the extensive vegetation removal and disturbance to the surface soil during the earthwork phase of construction, the soils encountered at the site are considered to have a moderate potential for excessive differential soil movement (mitigated by the foundations recommendations contained herein).

4.4 Groundwater

No groundwater was encountered during the course of this firm’s site investigation.

4.5 Frost Depth

The minimum foundation embedment depth is 1.5 feet (site elevation is approximately 4300 feet).

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. The following table summarizes the seismic wave velocity 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. included in their "Handbook of Ripping."

Table 3: Excavating Conditions

Layer	Depth of Occurrence[1]	Seismic Wave Velocity (feet per second)	Remarks Relative to Rippability
1	Extends from the surface to highly variable depths ranging from 0.4 to 8.8 feet[4]	1323 to 2809	D8N, Case 580 Trencher[2]
2	Below highly variable depths ranging from 0.4 to 5.9 feet[5]	4552 to 5426	D10N, Caterpillar 235, with hoe ram attachment for backhoes

Layer	Depth of Occurrence[1]	Seismic Wave Velocity (feet per second)	Remarks Relative to Rippability
3	Below highly variable depths ranging from 0.9 to 8.8 feet[6]	6058 to 7419	Blasting techniques may be required to accomplish effect material removal[3]

[1]Average calculated depth below the existing site surface at the locations of the seismic surveys. Variations on the order of 2.0 feet may be encountered in the layer depth calculations due to the variability of the materials, degrees of weathering, and orientation of the structures.

[2]As stated herein, over-sized particles are those greater than 3.0 inches. It must be noted that over-sized particles will occur within Layer 1 and should be anticipated during the excavation process.

[3]This is not a recommendation to blast, it is simply an indication of the effort that may be involved in removing the material.

[4]Range applicable to all test locations.

[5]Range applicable to seismic surveys A-B, E-F, G-H, K-L, TB-1, TB-2, & TB-3.

[6]Range applicable to seismic surveys C-D & I-J.

As inferred herein, over-sized particles are those greater than 3 inches. It must be noted that over-sized particles (greater than 3 inches) will be present within the subsurface and should be anticipated during the excavation process.

Excavations greater than 4.0 feet should be sloped or braced as required to provide personnel safety and satisfy local safety code regulations. 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 insure 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.

For all trenching, the subsurface soils (Layer 1), extending to depths ranging from 0.4 to 8.8 feet, at the subject site are considered to be OSHA Type C soils. Excavations into Type C soils are to be configured no steeper than a 1.5H:1V incline. Excavations into Layer 2 (encountered below depths ranging from 0.4 to 5.9 feet) are to be configured no steeper than a 0.75H:1V incline. Excavations into Layer 3 (encountered below depths ranging from 0.9 to 8.8 feet) are to be configured no steeper than a 1H:2.5V incline. Deviation from these recommendations will necessitate a trench support system or shielding.

5.2 Cut Slope Stability

The following table presents this firm's analysis of safe cut slopes for the anticipated subsurface conditions. However, it should be noted that the subsurface rock material (Layers 2 and 3), once exposed, could reveal hidden characteristics that may indicate the potential for slope instability during and after cutting operations. Therefore, this firm

recommends that the following safe cut slope criteria and associated slope stability analyses be implemented during construction.

Therefore, this firm recommends that the following safe cut slope criteria and associated slope stability analyses be implemented during construction.

Table 4: Cut Slope Recommendations Not Exceeding 30 Feet in Height

Portion of Cut Slope	Temporary Cut Slope Ratio (Horizontal to Vertical) "During the life of construction"[a]	Permanent Cut Slope Ratio (Horizontal to Vertical)
Layer 1	1.5:1	2.5:1
Layer 2	0.75:1.5	1:1
Layer 3	1:2.5	1:2

30.0 feet is recommended as the maximum cut slope height, using the appropriate cut slope ratios for the corresponding height limitation. Should the above presented cut slope recommendations not work with the site's geometry, a series of retaining walls would need to be designed and constructed, or stabilization of a steeper cut slope that is bolted.

This firm should be notified during construction to verify field conditions and inspect all cut slopes for structural features (e.g. shear zones, foliation/parting, fractures, 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 where; if unstable conditions are identified, that mitigation measures can be implemented before large scale cuts have been performed or slope failure occurs (i.e. inspecting and potentially modifying the cut slope recommendations, or possibly recommending the use of rock anchors, rock netting, or retaining walls for slope stability, when the cut is no greater than 10.0 feet in height). Note: Altered recommendations or mitigation measures shall be based on the results obtained from a Markland stability analysis, which is not part of the scope of work for this report.

These slope designs were completed under the assumption that surcharge loads will not be applied at the crest of any existing cut slope. All slopes should be cleared of loose materials. After construction, traffic on the crest of any cut slope should be limited to pedestrian foot traffic only, within 10.0 feet of the crest.

Very small flows of surface water may erode portions of the faces of the existing cut slopes and lead to localized slope movements. For this reason, all surface drainage should be controlled and directed away from any cut slopes. This firm recommends that a V-shaped trench be constructed 5.0 feet up-slope, adjacent and parallel to the crest of any cut-slope and graded to drain. The drainage trench design shall provide adequate protection for keeping water away from any exposed cut-slope and building area.

There exists the possibility of rock falls associated with possible weathered upper portions of any exposed rock stratum. In other words, some localized rock movements should be anticipated. Any such occurrence will be accommodated by the utilization of buffer zones. Buildings should not be constructed in, <u>and pedestrian traffic should be directed away from</u>, buffer zones. At the base of any cut-slope (beyond the toe of the cut-slope), buffer zones should be maintained according to the following schedule:

Table 5: Buffer Zones

Vertical Rock Cut-Slope Height (feet)	Horizontal Rock-Fall Impact Zone Distance (feet)
5	2.5
10	5
15	7.5
20	10
25	12.5
30	15

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. In addition to cut operations, vibrations from heavy equipment can induce a seismic-like component to a cut or natural slope which may reduce the overall slope stability and decrease the factor of safety against sliding below 1. Such vibrations can also dislodge boulders from a normally stable slope. It should also be noted that it is beyond this firm's ability to predict the time and place such an event (*rock fall or slope movement*) will occur. It is well known that erosional processes and gravity work continuously to move rock and soil down-slope, and therefore, <u>future slope movements should be anticipated</u> whether small or large.

To protect the structure from rock falls and rollouts, the following Rock Fall Catchment Geometry diagram must be adhered to. The diagram describes the geometry of the slope protection measures at the base of the slope.

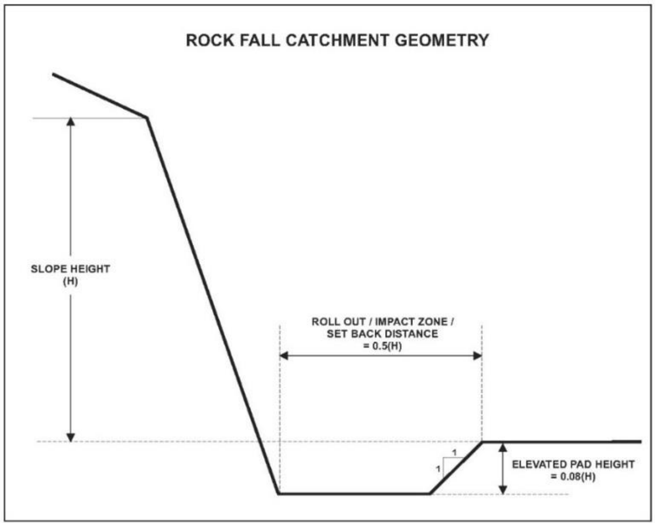


Figure 10: Rock Fall Catchment Geometry

5.3 Backfill Settlement

Retaining wall and utility trench backfill in building and pavement areas should be compacted to the density criteria previously presented herein. If backfills are not compacted as recommended, excessive settlement may result in areas adjoining backfilled retaining walls, or over utilities. Excessive settlement of loose backfills has caused damage to pavements, floor slabs, pedestrian walkways, planters, etc., which adjoin backfilled retaining walls.

Deep compacted backfills will also tend to settle differently relative to retaining walls and should not be used for support of adjoining facilities prone to damage from differential settlements, or facilities attached to the main structure.

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 in all wall backfills and in utility trench backfills. Because of the critical factor of minimizing settlements of approach slabs, particularly careful quality control should be exercised over backfill operations.

Heavily loaded structures that require minimal settlement (i.e. less than ¼-inch differential settlement), such has infinity edge swimming pools, should not bear on backfill soil. Such structures should be designed with deeper foundations (such has drilled shafts, micro piles, helical piers, etc.) which penetrate through the backfill soil and into a stronger stratum below. If recommendations for a deep foundation system are required, please contact this firm so that a subsequent analysis can be performed. 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.

Table 6: 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	10.0 feet of backfill	12.5 feet of backfill	15.0 feet of backfill	17.5 feet of backfill	20.0 feet of backfill	22.5 feet of backfill	25.0 feet of backfill	27.5 feet of backfill	30.0 feet of backfill
95-98	Very Well Compacted	0.5	0.15	0.3	0.45	0.6	0.75	0.9	1.05	1.2	1.35	1.5	1.65	1.8
95	Well Compacted	1	0.3	0.6	0.9	1.2	1.5	1.8	2.1	2.4	2.7	3.0	3.3	3.6
90	Moderately Compacted	2.5	0.75	1.5	2.25	3.0	3.75	4.5	5.25	6.0	6.75	7.5	8.25	9.0
85	Poorly Compacted	5	1.5	3.0	4.5	6.0	7.5	9.0	10.5	12.0	13.5	15.0	16.5	18.0
80	Very Poorly Compacted	7.5	2.25	4.5	6.75	9.0	11.25	13.5	15.75	18.0	20.25	22.5	24.75	27.0

Accordingly, it is recommended that where slabs are supported on grade over fill but are also tied to or connected to elements supported at retaining level, special construction details should be utilized. Concrete slabs should be hinged or keyed at the base where they join the rigid structure in order to allow slight rotation of the slab. These measures will reduce the likelihood that such slabs will crack or suffer noticeable deformations. Also refer to Slab Support presented herein.

5.4 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.

It is recommended that vegetation, asphalt, and all other deleterious materials be removed at the commencement of site grading activities. Although underground facilities such as septic tanks, cesspools, basements, and dry wells were not encountered, such features most likely will be encountered during construction. These features should be demolished or abandoned in accordance with the recommendations of the geotechnical engineer. Such measures may include backfill with 2-sack ABC/cement slurry.

All existing compacted subgrade soils must be removed in the proposed hardscape and pavement areas. According to the field investigation, this will require the removal of approximately 6.0 to 9.0 inches of compacted subgrade at the locations of the TB-1 & TB-2. Greater thicknesses of compacted subgrade may be encountered at locations not drilled by this firm, specifically at the existing portion of Forest Road. The presence of native soils at the base and sides of the subgrade removal excavation must be verified by the project geotechnical engineer.

Following the removal of the above-mentioned, a minimum of 8.0 inches of the native soils should be scarified, moisture processed and compacted as specified below. The scarification and compaction requirements apply to cut situations as well as fill situations.

Any site cut material may be reused as structural supporting fill provided the maximum particle size is 3.0 inches and a suitable percentage of fines will be generated to ensure a stable mixture. **Please note that significant over-sized aggregate (greater than 3.0 inches) will exist within the subsurface and must be accommodated.**

Complete removal and cleaning of any undesirable materials and proper backfilling of depressions will be necessary to develop support for the proposed facilities. Widen all depressions as necessary to accommodate compaction equipment and provide a level base for placing any fill.

Any tree removal efforts made to accommodate the new structure must include removal of the root systems, followed by backfilling of the volume occupied by the root ball. Typically, to remove all significant roots such that the maximum diameter of any root is no greater than ½ inch, it is required to excavate to a depth of 4.0 feet to capture all applicable roots. Further, the lateral extent of each tree root excavation is generally 8.0 feet (twice the depth).

All fills shall be properly moistened and compacted as listed below. All subbase fill required to bring the structure areas up to subgrade elevation should be placed in horizontal lifts not exceeding 6 inches compacted thickness or in horizontal lifts with thicknesses compatible with the compaction equipment utilized.

Fill placement in wash areas or sloped topography should involve horizontal layers placed in 6-inch lifts, such that each successive lift is benched into the native site soils a minimum lateral distance of 5.0 feet.

It is the understanding of this firm that various utility trenches may traverse the completed pads. The backfill of all utility trenches, if not in conformance with this report, may adversely impact the integrity of the completed pads. This firm recommends that all utility trench backfill crossing the pad be inspected and tested to ensure full conformance with this report. Untested utility trench backfill will nullify any as-built grading report regarding the existence of controlled compacted fill beneath the proposed building foundations and place the owner at greater risk in terms of potential unwanted foundation and floor slab movement.

Compaction of backfill, subgrade soil, subbase fill, and base course materials should be accomplished to the following density criteria:

Table 7: Compaction Requirements

Material	Building Area	Percent Compaction (ASTM D698)	Compaction Moisture Content Range (%)
On-site native and fill soils with PI < 12	Below Foundation Level	95 min	optimum -2 to optimum +2
	Above Foundation Level[1]	95 min	optimum -2 to optimum +2
	Below Pavements	95 min	optimum -3 to optimum +1

Material	Building Area	Percent Compaction (ASTM D698)	Compaction Moisture Content Range (%)
Imported fill material	Below Foundation Level	95 min	optimum -2 to optimum +2
	Above Foundation Level[1]	90 min	optimum -2 to optimum +2
	Below Pavements	95 min	optimum -3 to optimum +1
Base course	Below Interior Concrete Slabs	95 min	-
	Below Pavements	100 min	-

[1]Also applies to the subgrade in exterior slab, sidewalk, curb, and gutter areas.

Any soil disturbed during construction shall be compacted to the applicable percent compaction as specified herein. Increase the required degree of compaction to a minimum of 98 percent for fill materials greater than 5 feet below final grade. Natural undisturbed soils or compacted soils subsequently disturbed or removed by construction operations should be replaced with materials compacted as specified above.

All imported fill material to be used as structural supporting fill should be free of vegetation, debris and other deleterious material and meet the following requirements:

Table 8: Imported Fill Soil Parameters

Soil Parameter	Requirement
Plasticity Index:	14 (Maximum)
Particle Size:	3 inches (Maximum)
Passing #200 Sieve:	60 % (Maximum)
Expansion Potential*:	1.5 % (Maximum)
Sulfates:	0.19 % (Maximum)

*Performed on a sample remolded to 95 percent of the maximum ASTM D698 density at 2 percent below the optimum moisture content, under a 100 PSF Surcharge.

*Performed on a sample remolded to 95 percent of the maximum ASTM D698 density at 2 percent below optimum moisture content, under a 100 PSF surcharge. Water settling and/or slurry shall not, in any case, be used to compact or settle surface soils, fill material, or trench backfill within 10 feet of a structure area or within an area which is to be paved.

5.5 Fill Slope Stability

Maximum fill slopes may conform to a 2.5:1 (horizontal: vertical) ratio if fill is placed in accordance with the recommendations contained herein.

5.6 Shrinkage

For balancing grading plans, the estimated shrink of on-site soils has been provided below. The calculated shrink assumes oversized material will be processed and used on the project (i.e. oversized material is crushed and used in engineered fill). Based on the soil properties, the anticipated shrinkage is:

Table 9: Shrinkage

Material	Estimated Shrinkage (Based on ASTM D698A)
On-site soils	13% ± 3

The above value does not take into account losses due to erosion, waste, variance of on-site soils, over-excavation, re-compaction of zones disturbed by demolition, previous site usage or the screening of oversized particles and/or debris. In other words, additional factors can and will create situations where seemingly balanced grading and drainage plans do not balance during construction.

5.7 Site Classification

This project is not located over any known active faults or fault associated disturbed zones. An IBC Seismic Site Classification **B** may be used in the earthquake design of the proposed structure for publication years up through 2018.

5.8 Conventional Surface-Level Spread Foundations for Retaining Wall Structures

Due to the moderate collapse potential, it is recommended that retaining wall foundations bearing on native undisturbed soil (Layer 1) or engineered fill be embedded a minimum of 2.5 feet below the lowest adjacent finish grade within 5.0 feet of the proposed retaining walls.

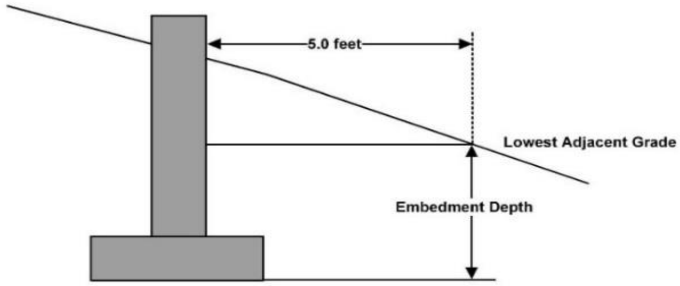
For all construction, 2.0 feet and 1.33 feet are recommended as the minimum width of spread and continuous footings, respectively. The following tables may be used in the design of shallow spread (*column*) and continuous (*wall*) foundations for the proposed retaining walls.

Table 10: Conventional Surface Level Foundations for Retaining Walls

Foundation Embedment Depth (ft)[1]	Bearing Stratum[2]	Allowable Soil Bearing Capacity[3]
2.5	Native undisturbed soil (Layer 1)[4]	1500 PSF
3.0	Native undisturbed soil (Layer 1)[4]	1750 PSF
3.5	Native undisturbed soil (Layer 1)[4]	2000 PSF
Bearing on Layer 2 with a minimum foundation embedment depth of 1.5 feet	Layer 2	4500 PSF
Bearing on or into Layer 3 with a minimum foundation embedment depth of 1.0 feet	Layer 3	6000 PSF

[1]Conditions for foundation embedment depth:

a) The depth below the lowest adjacent exterior grade within 5.0 feet of proposed retaining walls;



[2]Refers to the soil layer that the footing pad rests on, and does not mean to imply that the foundation be fully embedded into that particular stratum

[3]**The allowable soil bearing capacity values are based on a total settlement of ½-inch and a differential settlement of ¼-inch.** The maximum estimated footing settlements (in situ) should be within tolerable limits if constructed in accordance with the recommendations contained in this report and a reasonable effort is made to balance loads on the footings

[4]A mixture of 2-sack ABC/cement slurry may be utilized in the lower portions of the foundation excavations for footings bearing on native undisturbed soil. For example, if ABC/cement slurry is used, 1.0 feet of the mixture should underlie a conventional foundation depth of 1.5 feet for an allowable soil bearing capacity of 1500 psf. The preceding table shall govern the thickness of 2-sack ABC/cement slurry depending on the allowable soil bearing capacity selected.

The weight of the foundation below grade may be neglected in dead load computations. The above recommended bearing capacities should be considered allowable maximums for dead plus design live loads. The allowable bearing may be increased by a factor of 1.33 for resistance to wind loads and/or temporary eccentric loading. We recommend that continuous footings and stem walls are reinforced, and bearing walls be constructed with frequent joints to better distribute stresses in the event of localized settlements. Similarly, all masonry walls should be provided with both vertical and horizontal reinforcement. It is recommended that the footing excavations be inspected to ensure that they are free of loose soil which may have blown or sloughed into the excavations and that all of the footings will bear upon engineered fill native undisturbed soil (Layer 1) at the above-described depths.

Code compliant concrete, with Type II cement, should be used for footings, stem walls and floor slabs. A maximum 4-inch slump should be used for footings and stem walls and a maximum 6-inch slump should be used for floor slabs.

5.9 Lateral Stability Analyses

All on-site retaining walls must be designed to resist the anticipated lateral earth pressures. Unrestrained (free-end) retaining walls should be designed for active earth pressures (K_a) and are assumed to allow small movement of the wall. Restrained (fixed-end) retaining walls should be designed for at-rest earth pressures (K_o) with no assumed wall movement. Soil or rock present in front of the toe of the retaining wall will provide resistance to movement and should be modeled as passive earth pressure (K_p). The following presents recommendations for lateral stability analyses for native undisturbed soil (Layer 1), Layer 2, and Layer 3:

Table 11: Lateral Stability

Parameter	Wall Type	Native Undisturbed Soil (Layer 1)	Layer 2 (Sandstone)	Layer 3 (Sandstone)
Active (K_a) Pressure[a]	Free-end	34 psf/ft (compacted backfill)		
At-Rest (K_o) Pressure[a]	Fixed-end[b]	52 psf/ft (compacted backfill)		
Passive (K_p) Resistance	Free-end/Fixed-end independent of base friction	315 psf/ft	593 psf/ft	698 psf/ft
	Fixed-end in conjunction with base friction	211 psf/ft	398 psf/ft	468 psf/ft
Coefficient of Base Friction (μ)	Free-end/Fixed-end independent of passive resistance	0.58	0.81	0.87
	Free or Fixed-end in conjunction with passive resistance	0.39	0.54	0.58

[a]Equivalent fluid pressures for vertical walls and horizontal backfill surfaces (maximum 15.0 feet in height). Pressures do not include temporary forces during compaction of the backfill, expansion pressures developed by over-compacted clayey backfill, hydrostatic pressures from inundation of backfill, or surcharge loads. Walls should be suitably braced during backfilling to prevent damage and excessive deflection.

[b]The backfill pressure can be reduced to the unrestrained lateral pressure if the backfill zone between the wall and cut slope is a narrow wedge (width less than 1/2 the height).

[c]Values applicable to stable cut slopes as ensured through adherence to the safe cut slopes recommended herein.

The equivalent fluid pressures presented herein do not include the lateral pressures arising from the presence of:

- Hydrostatic conditions, submergence or partial submergence
- Sloping backfill, positively or negatively
- Surcharge loading, permanent or temporary
- Seismic or dynamic conditions

Placement of fill against footings, stem walls should be compacted to the densities specified herein. High plasticity clay soils should not be used as backfill against retaining walls. Compaction of each lift adjacent to walls should be accomplished with hand-operated tampers or other lightweight compactors. Overcompaction may cause excessive lateral earth pressures that could result in wall movements.

We recommend a free-draining soil layer or manufactured geosynthetic material, be constructed adjacent to the back of any retaining walls serving as basement walls. A filter fabric may be required between the soil backfill and drainage layer. The drainage zone should help prevent development of hydrostatic pressure on the wall. This vertical drainage zone should be tied into a gravity drainage system at the base of the wall.

5.10 Rockery Walls

Rockery walls are gravity walls that consist of uncemented, interlocking rows of large rocks or boulders that are not tied together and have a low tolerance for movement. The rocks are typically natural shaped quarry stone or boulders. The rock to be utilized should be blocky in shape to allow stacking. In rockery wall locations, some excavation will be required to allow for proper placement of the boulders. The temporary cut slope angle behind the stacked boulder wall shall conform to the requirements herein.

The excavation should create a foundation trench at the base of the rockery wall. The foundation trench should be excavated to a minimum depth of 12 inches into the existing soil or rock. The base of the excavation should be level or slightly inclined back into the slope.

If the ground surface is sloping beyond the toe, the embedment depth should be increased to 30 inches. The foundation trench must be stepped down the slope to maintain level placement of the base boulders along the toe of the rockery wall. The bottom of each foundation trench must also slope towards (into) the slope at a minimum inclination of 5%. The foundation trench should be excavated to the width necessary to accommodate the width of the base rock and be set 12 inches away from the face of the cut slope to allow for the crushed rock backdrain behind the boulders (typically 4 to 5 feet depending on the size of the base boulder).

The first course of rock, the base rock, should be placed into the foundation trench on firm, unyielding soil or bedrock with full contact between the rock and the subgrade. Excavate any loose, soft or otherwise unsuitable material present at foundation grade and replace with fill compacted to 95% of maximum dry density as determined by the ASTM D698 standard proctor test, or 12 inches of crushed, screened 4 to 6-inch backdrain rock (to create an n-value greater than 30). As the rockery wall is constructed, place the rocks so that there are no continuous joints in either the vertical or lateral direction. Stockpile a sufficient number of rocks to provide a good selection for placement. To obtain a better fit, place rocks that do not match the spaces offered by the previous course in a different location.

Avoid placing rocks or boulders that have shapes that create voids with a linear dimension greater than 12 inches. Except in isolated cases, place each rock so that it bears on at least two rocks below it. Locate at least one bearing point a distance no greater than 6 inches from the average front face of the rockery wall. The long rock dimension should be placed perpendicular to the slope. Slope the top surface of each rock towards the back of the rockery wall at an inclination of at least 5 percent.

The allowable tolerance for base rock widths is shown on the attached Rockery Wall Cross Section detail in Section IV. Do not place two or more consecutive base rocks with a width less than the minimum specified. The minimum rockery wall thickness is based on the minimum base rock width, and the required width for each lift above. The batter of the wall face must be maintained at a maximum 1:4 (Horizontal to vertical) slope angle or 74 degrees. Securely place the rock or boulders so that the rocks are unable to be moved with a pry bar after the rockery wall is complete.

Where voids with a minimum dimension of 6 inches or greater exist in the face of the rockery, chink the voids with smaller rock. Chinking rocks does not provide primary structural support for the overlying rock. Chinking rocks cannot be moved or removed by hand after the rockery wall is complete. Chinking rocks that are loose can be tapped in place with a small sledge hammer until they are secure, or they may be grouted in place. Do not allow any grout placed to secure chink rocks to be readily visible from the face of the rockery wall.

Backdrain construction should involve placing a 4-inch diameter perforated drain pipe, surrounded on the sides by at least 4 inches of drain rock, and be placed at the base of the drain rock zone. The drain pipe should be placed with the perforations facing to either side. For a tiered rockery wall, separate discharge pipes should be used for the upper and lower walls and should not interconnect.

The drain pipe, drain rock and nonwoven geotextile filter fabric should be placed as indicated by the following cross section.

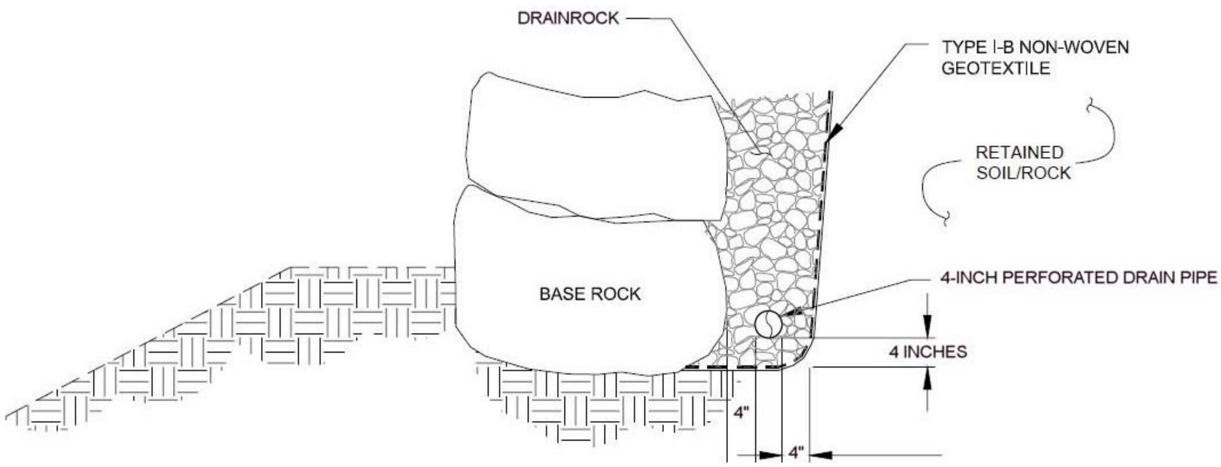


Figure 11: Cross-section showing drain pipe, drain rock, and nonwoven geotextile

Install the granular rock back drain between the rockery wall and the backfill or cut face behind the rock wall that is being supported. The granular rock back drain layer must be at least 12 inches thick, measured horizontally from the back of rock to the cut slope. Place the granular rock back drain concurrent with the rocks for the wall so that at no time will either one be more than 24 inches higher than the other. At least 12 inches of impermeable compacted soil or lean slurry should overlie the drain rock and non-woven geotextile, followed by the construction of a concrete lined v-ditch as shown below. The surrounding site shall be graded such that surface water cannot flow over the top of the wall. The non-woven geotextile should be placed as indicated in the preceding and following cross sections.

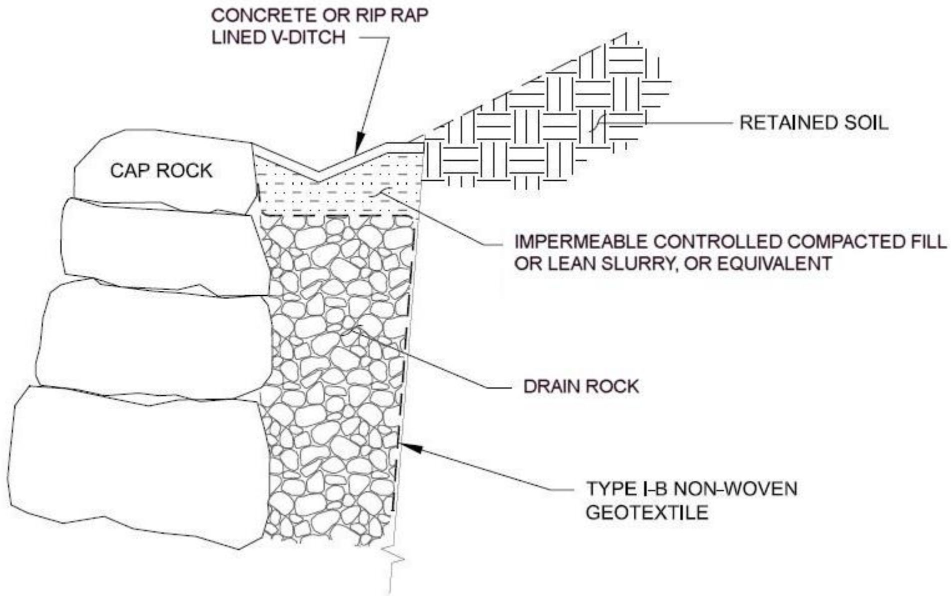


Figure 12: Cross-section showing impermeable compacted fill / slurry with the concrete lined v-ditch

An engineer from Vann Engineering, Inc. should monitor the rockery wall construction until this work is complete. Problems that may be encountered as the work is accomplished, due to conditions that are not currently visible or anticipated, will need to be dealt with at that time, to prevent delays to the construction. The following photographs depict situations to avoid; rocks that tilt outward from the slope, and vertical seams.

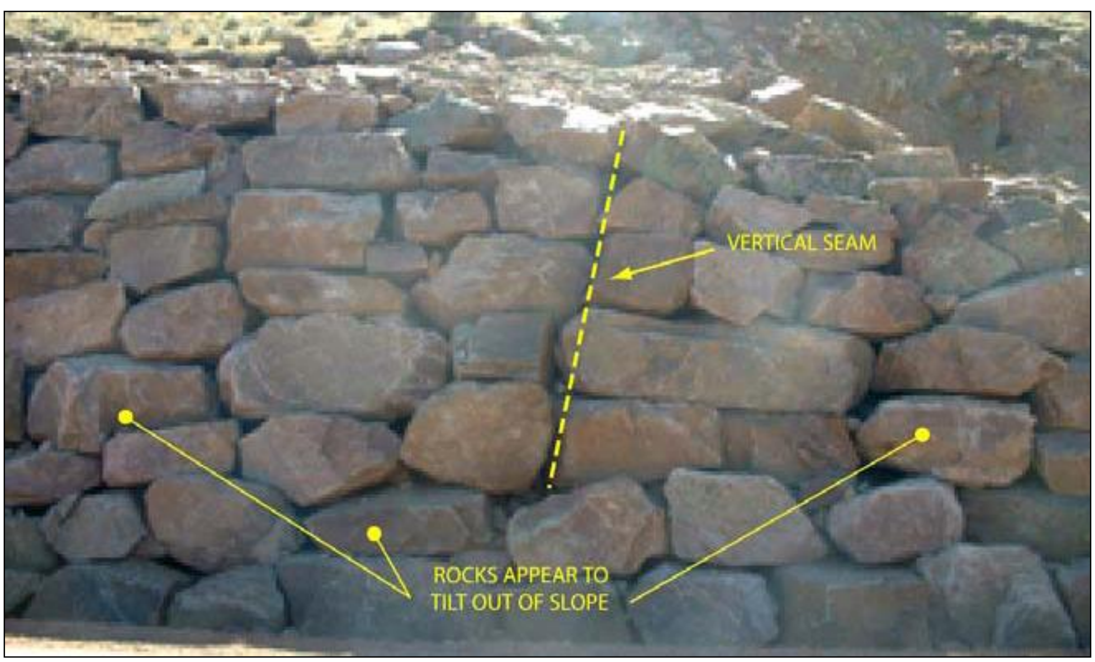


Figure 13: Image depicting unacceptable rockery wall construction featuring vertical seams & rocks tilted out of slope

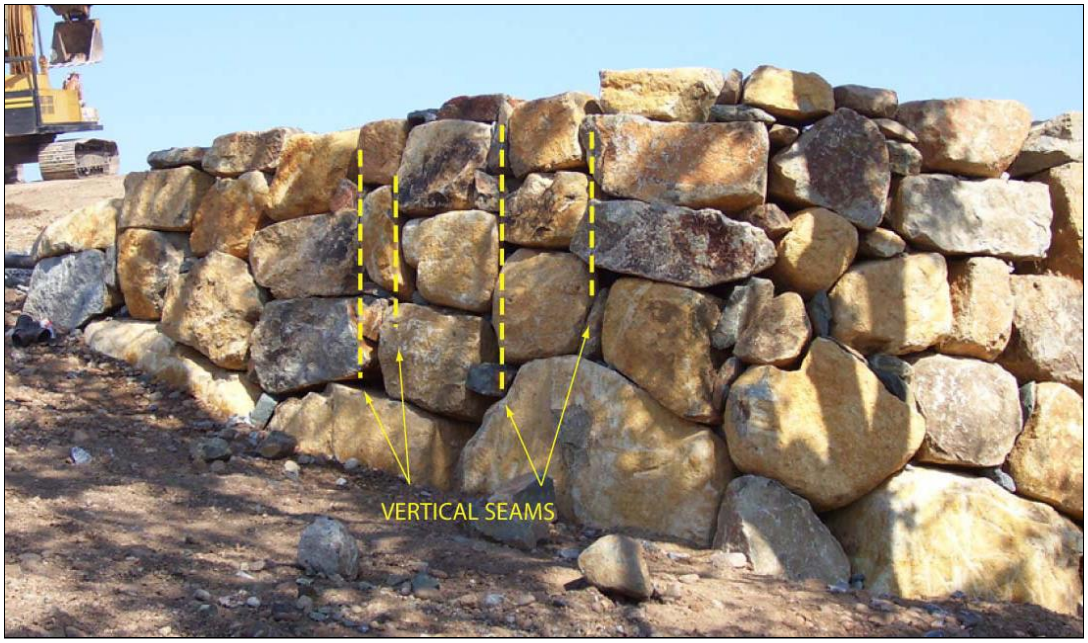


Figure 14: Image depicting unacceptable rockery wall construction featuring vertical seams

Material for the granular rock back drain must conform to following specification:

Table 12: Back Drain Specifications

Sieve Size	Percent Passing
6”	100
4”	0-25
¾”	0-15
#4	0-5
#200	0-2.0

The height of any single rockery wall may not exceed 15 feet. The face of the rockery wall shall have a batter no steeper than 1:4 (horizontal to vertical, or 74 degrees) measured at the exposed face of the wall.

This design complies with following safety factors:

Table 13: Factors of Safety

Mode of Failure	Static Factor of Safety (Minimum)
Sliding	1.5
Overturning	2.0
Bearing	2.0
Global Stability	1.5

Please review the attached rockery wall details for clarification of construction details.

Reference: Rockery Design and Construction Guidelines, Publication No. FHWA-CFL/TD-06-006, November 2006.

5.11 Conventional Slab Support

4.0 inches of aggregate base course (ABC) floor fill should immediately underlie floor slabs. The aggregate base material should conform to the requirements of local practice.

Building pads for conventional systems may be constructed with sufficient lateral pad "blow-up" to accommodate the entire perimeter slab width. To further reduce the potential for slab related damage in conjunction with conventional systems, we recommend the following:

1. Placement of effective control joints on relatively close centers
2. Proper moisture and density control during placement of subgrade fills
3. Provision for adequate drainage in areas adjoining the slabs
4. Use of designs that allow for the differential vertical movement described herein between the slabs and adjoining structural elements, **i.e. ¼ inch**
5. Utilization of 2-sack ABC/cement slurry as backfill at the intersection of utility trenches with the building perimeter

The use of vapor retarders may be considered for any slab-on-grade where the floor will be covered by products using water-based adhesives, wood, vinyl backed carpet, impermeable floor coatings (urethane, epoxy, or acrylic terrazzo). When used, the design and installation should be in accordance with the recommendation given in ACI 302.1R-96.

5.12 Off-Site Pavement Design

Site grading within pavement areas should provide requisite subgrade support for flexible pavements as defined herein. A compacted subgrade of on-site soils or soils with comparable properties is assumed. Pavement materials and placement requirements should be in accordance with local practice, or equivalent. Asphaltic concrete surfacing must comply with local standard guidelines.

The stability of compacted pavement subgrade soils is reduced under conditions of increased soil moisture. Therefore, base course or pavement materials should not be placed when the surface is in a wet condition. Adequate surface drainage should be provided away from the edge of paved areas to minimize lateral moisture transmission into the subgrade.

No traffic data was provided during the course of this report's preparation. However, there are only three basic options to consider; rural with relatively low traffic, rural with higher traffic and urban.

The following table presents minimum recommended pavement sections for the possible traffic conditions.

Table 5. Off-site Pavement Design Recommendations – Forest Road

Traffic Loading Conditions for Major Collector Streets	Minimum Structural Number (SN)	Prepared Subgrade (Inches)*	ABC (Inches)	Asphaltic Concrete (Inches)
Urban	2.25	8.0	9.0	3.0
Rural with ESAL ≥ 750,000	2.25	8.0	9.0	3.0
Rural with ESAL < 750,000	2.0	8.0	7.0	3.0

*All existing compacted subgrade soils must be removed prior at the start of the earthwork efforts. Approximately 6.0 to 9.0 inches of compacted subgrade soils were encountered at the locations of the TB-1 & TB-2. Native undisturbed soil must be exposed at the base of the compacted subgrade excavation. Refer to the Site Preparation section herein for further earthwork requirements.

Compaction of subbase fill and base course materials should be accomplished to the density and moisture criteria listed herein. Compaction of asphalt surfacing should be accomplished to 95% (minimum) using the 75-blow method.

5.13 Foundations and Risks

The factors that aid in the design and construction of lightly loaded foundations include economics, risk, soil type, foundation shape and structural loading. Most of the time, foundation systems are selected by the owner/builder, who as a result of economic considerations, accepts higher risks in foundation design. It should be noted that some levels of risk are associated with all foundation systems and there is no such thing as a "zero-risk" foundation. It also should be noted that the foundation recommendations presented herein are not designed to resist soil movements as a result of sewer/plumbing leaks, excessive irrigation, poor drainage, or water ponding near the foundation system.

It is recommended that the owner/builder implement a foundation maintenance program to help reduce potential future unwanted foundation/slab movements throughout the useful life of the structure. The owner should conduct yearly observation of foundations and slabs and perform any maintenance necessary to improve drainage and minimize infiltrations of water from precipitation and/or irrigation. Irrigation/sprinkler systems should be periodically monitored for leaks and malfunctioning sprinkler heads, which should be repaired immediately. Post-construction landscaping should be carefully designed to preserve initial site grading.

6.0 ADDITIONAL SERVICES

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. Vann Engineering, Inc. should be retained to provide documentation that the recommendations set forth are met. These include but are not limited to documentation of site clearing activities, verification of fill suitability and compaction, and inspection of footing excavations.

Relative to field density testing, a minimum of 1 field density test should be taken for every 2500 square feet of building area, per 6-inch layer of compacted fill. 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 as related to fill control, foundation inspections and concrete sampling. 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. Proof rolling and fill subgrade conditions
B. Suitability of borrow materials
C. Fill control for building pads (verification of overexcavation depths and lateral extents, compaction testing, and the general monitoring of fill placement)
D. Foundation observations (compliance with the General Structural Notes, depths, bearing strata, etc.)
E. Basement, structural or retaining wall backfill testing
F. Backfilling and compaction of excavations (e.g. Utility trench backfill)
G. Special inspections as dictated by the local municipality
H. Concrete sampling and testing for footings, stem walls and floor slabs
I. Subgrade testing for proposed pavement areas
J. ABC testing for proposed pavement areas
K. Asphaltic concrete testing for proposed pavement areas
L. Subgrade preparation for on-site sidewalk areas
M. Grout sampling and testing, where applicable
N. Mortar sampling and testing, where applicable
O. Compliance with the geotechnical recommendations

7.0 LIMITATIONS

The materials encountered on the subject 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 are based on the assumption 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.

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 to 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. Prior to construction, we recommend the following:

1. 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.
2. 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).

3. This firm be notified of all specific areas to be treated as special inspection items (designated by the architect, structural engineer or governmental agency).

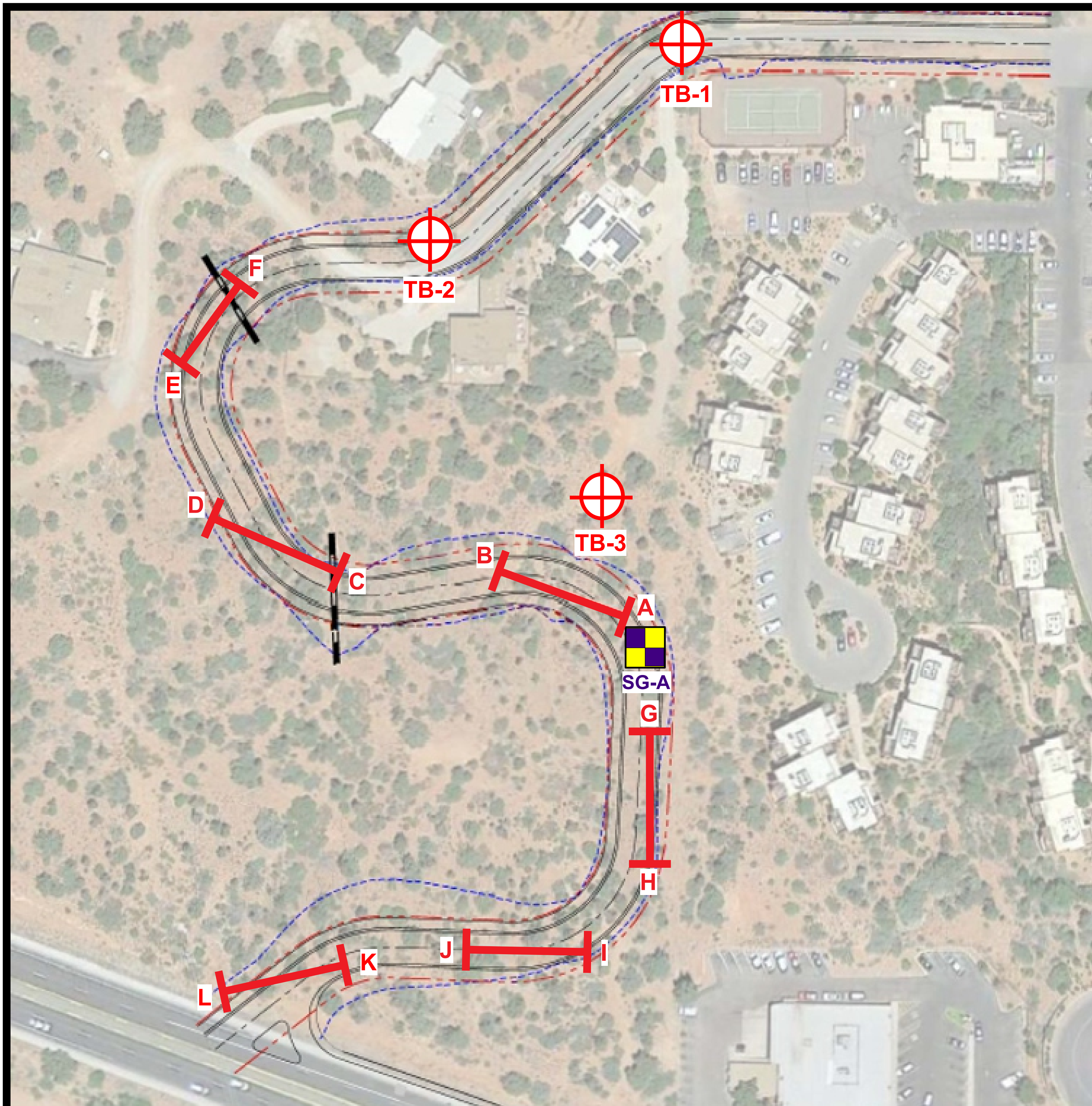
Relative to this firm's involvement with the project during the course of construction, we offer the following recommendations:

1. 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.
2. 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.
3. All parties concerned should understand that there exists a priority surrounding the testing and observation services completed at the site.

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



LEGEND:

TEST BORING LOCATIONS

SEISMIC SURVEY LOCATIONS

SUBGRADE SAMPLE LOCATION

N

SITE PLAN

SCALE: N.T.S.

DATE: 4/27/2020

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILTY, SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A,
NORTHWEST OF HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

PREPARED BY: BW

PROJECT 27266

VANN ENGINEERING INC

Vann Engineering, Inc.
9013 North 24th Avenue, Suite 7
Phoenix, Arizona
www.vannengineeringing.com

TEST BORING 1

CLIENT KIMLEY-HORN

PROJECT NAME Sandstone Retaining Wall Options

PROJECT NUMBER 27266

PROJECT LOCATION Forest Road Connection to Highway 89A, NW of Highway 89A

DATE STARTED 4/20/20 **COMPLETED** 4/20/20

GROUND ELEVATION

HOLE SIZE 4.5 inches

DRILLING CONTRACTOR VEI

LOGGED BY MM

CHECKED BY JDV

DRILLING METHOD 4.5 Inch Continuous Flight Auger

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	REMARKS	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	SPT N VALUE □ 0 10 20 30 40 50 60 / MOISTURE (%) △ 0 5 10 15 20 25 30 / PLASTICITY INDEX ○ 0 9 18 27 36 45 54
0		2" ASPHALT				
		COMPACTED SUBGRADE (9") slightly damp to damp, 80% sand, 20% silt and clay, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 5, non-cemented				
1		(SS) SANDSTONE, highly weathered and fractured, red-brown, slightly damp, 57% gravel, 22% sand, 21% silt and clay, poorly graded, subangular to subrounded coarse-grained particles, dense to very dense, PI of 5, non-cemented	Gradation is representative of rock flower			
2		very dense below 2.0 feet		R	26-36	
3				GB		
4		Discontinued test boring at 4.0 feet.	Auger refusal at 4.0 feet on highly to moderately weathered and fractured sandstone rock			

This boring log is considered invalid if detached from the original report. This report is not intended as a bidding document.

Vann Engineering, Inc.
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Phoenix, Arizona
www.vannengineeringing.com

TEST BORING 2

CLIENT KIMLEY-HORN

PROJECT NAME Sandstone Retaining Wall Options

PROJECT NUMBER 27266

PROJECT LOCATION Forest Road Connection to Highway 89A, NW of Highway 89A

DATE STARTED 4/20/20 **COMPLETED** 4/20/20

GROUND ELEVATION

HOLE SIZE 4.5 inches

DRILLING CONTRACTOR VEI

LOGGED BY MM

CHECKED BY JDV

DRILLING METHOD 4.5 Inch Continuous Flight Auger

NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	REMARKS	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	SPT N VALUE □ 0 10 20 30 40 50 60 / MOISTURE (%) △ 0 5 10 15 20 25 30 / PLASTICITY INDEX ○ 0 9 18 27 36 45 54
0		COMPACTED SUBGRADE (6") slightly damp, 30% gravel, 45% sand, 25% silt and clay, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 4, non-cemented				
		(SS) SANDSTONE, heavily weathered and fractured, red-brown, damp, 49% gravel, 27% sand, 24% silt and clay, poorly graded, subangular to subrounded coarse-grained particles, PI of 4, non-cemented	Gradation is representative of rock flower			
1						
2				R	27-50	
3				GB		

Discontinued test boring at 3.5 feet.

Auger refusal at 3.5 feet on highly to moderately weathered and fractured sandstone rock

Vann Engineering, Inc.
9013 North 24th Avenue, Suite 7
Phoenix, Arizona
www.vannengineeringing.com

TEST BORING 3

CLIENT KIMLEY-HORN
PROJECT NAME Sandstone Retaining Wall Options
PROJECT NUMBER 27266
PROJECT LOCATION Forest Road Connection to Highway 89A, NW of Highway 89A
DATE STARTED 4/20/20 **COMPLETED** 4/20/20
GROUND ELEVATION
HOLE SIZE 4.5 inches
DRILLING CONTRACTOR VEI
LOGGED BY MM
CHECKED BY JDV
DRILLING METHOD 4.5 Inch Continuous Flight Auger
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	REMARKS	SAMPLE TYPE NUMBER	BLOW COUNTS (N VALUE)	SPT N VALUE □ (0 10 20 30 40 50 60) / MOISTURE (%) △ (0 5 10 15 20 25 30) / PLASTICITY INDEX ○ (0 9 18 27 36 45 54)
0		(SS) SANDSTONE, heavily weathered and fractured, red-brown, damp, 80% sand, 20% silt and clay, poorly graded, subangular to subrounded coarse-grained particles, PI of 4, non-cemented				
1		slightly damp, 42% gravel, 25% sand, 33% silt and clay, very dense below 1.0 feet	Gradation is representative of rock flower			
2				R	11-19	
3				GB		
		Discontinued test boring at 3.5 feet.	Auger refusal at 3.5 feet on highly to moderately weathered and fractured sandstone rock			

VELOCITY CLASSIFICATION DATA

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

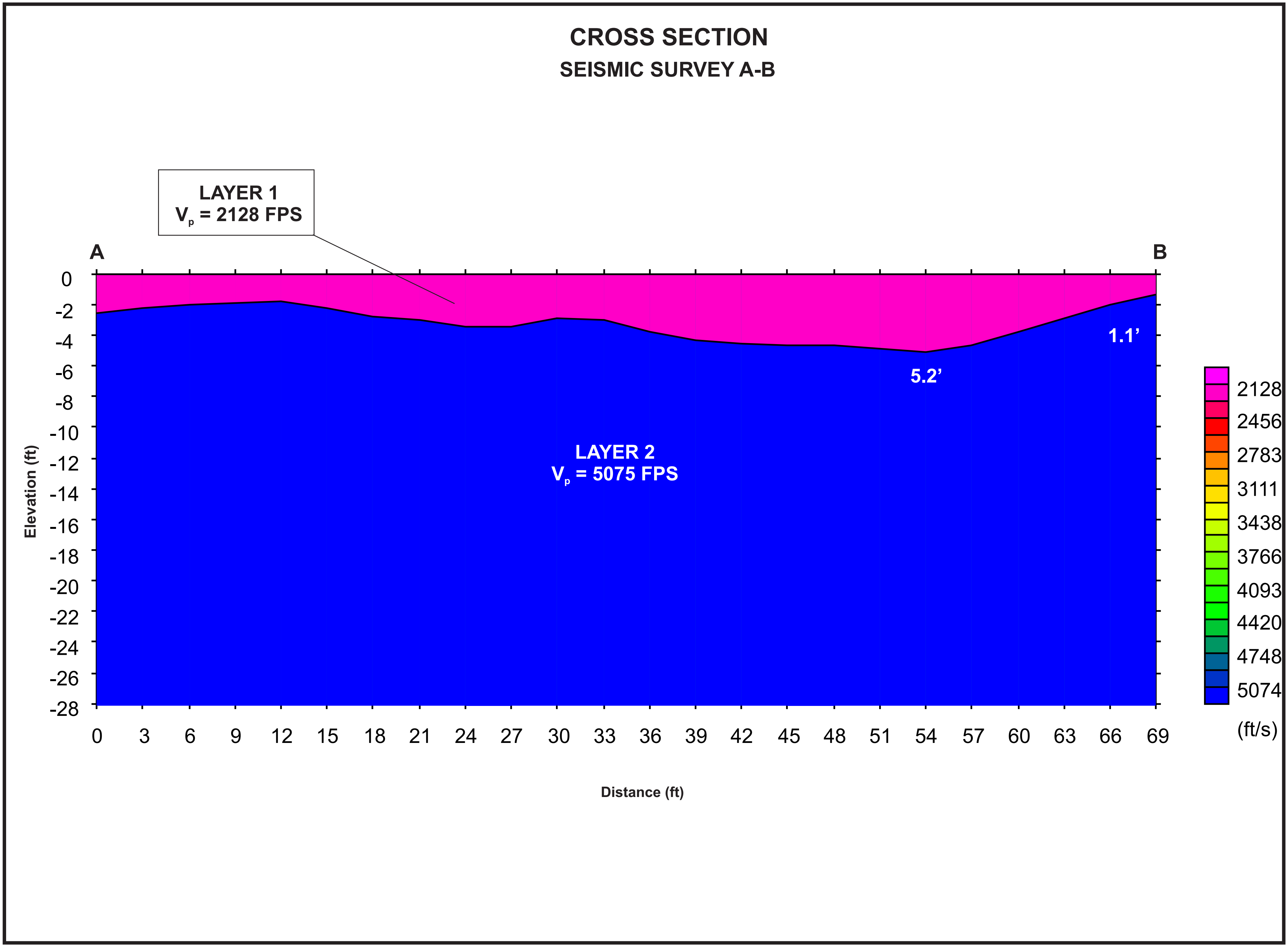
Average Velocity of Layer 1:	2021 fps
Average Velocity of Layer 2:	5082 fps
Average Depth to Layer 2:	3.1 feet
Range:	0.4 to 5.9 feet
Average Velocity of Layer 3:	6739 fps
Average Depth to Layer 3:	4.7 feet
Range:	0.9 to 8.8 feet

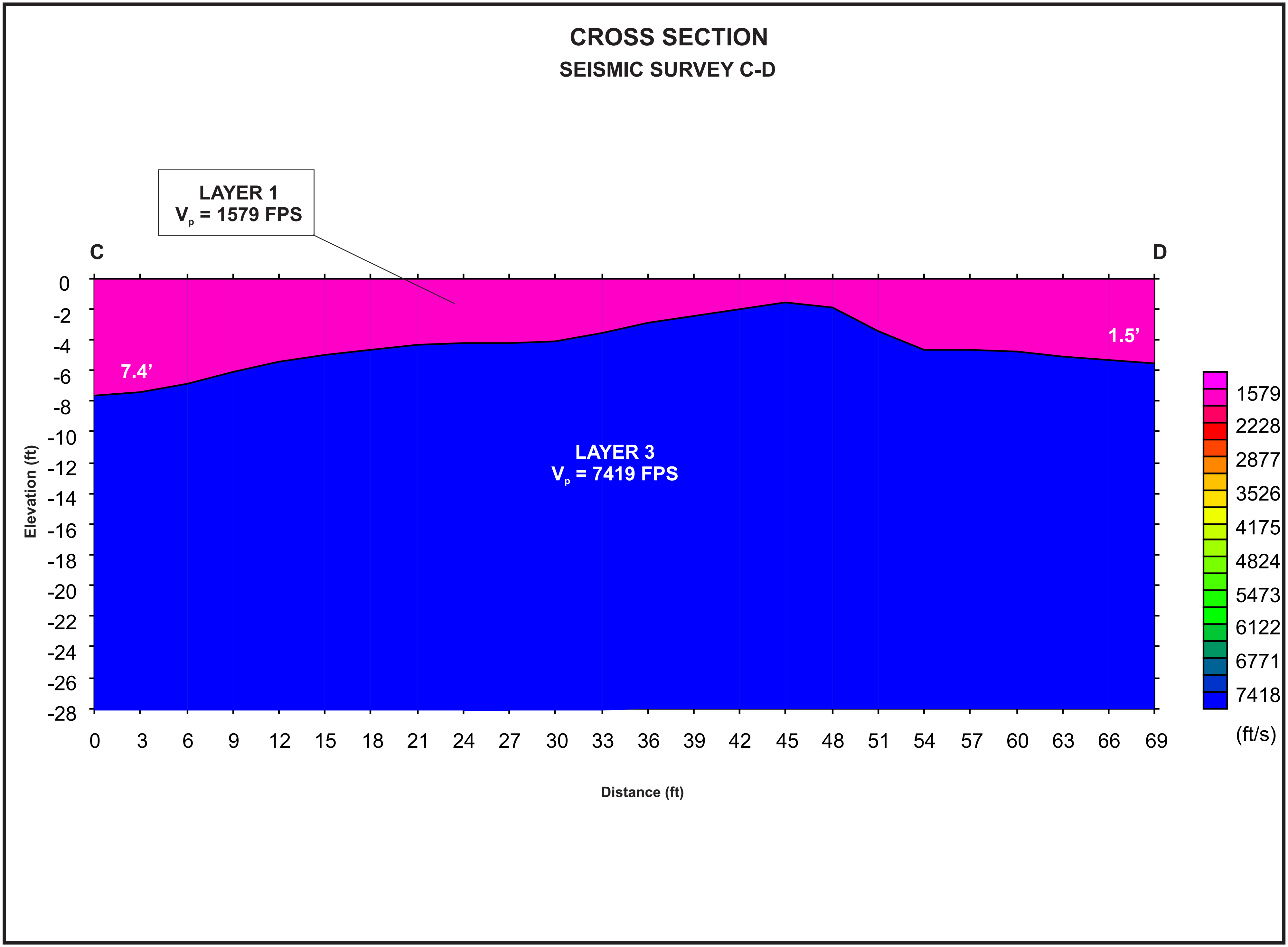
Layer 1: MODERATELY DENSE COARSE-GRAINED ALLUVIAL DEPOSITS COMPRISED OF SILTY CLAYEY SANDY GRAVEL (GC-GM)

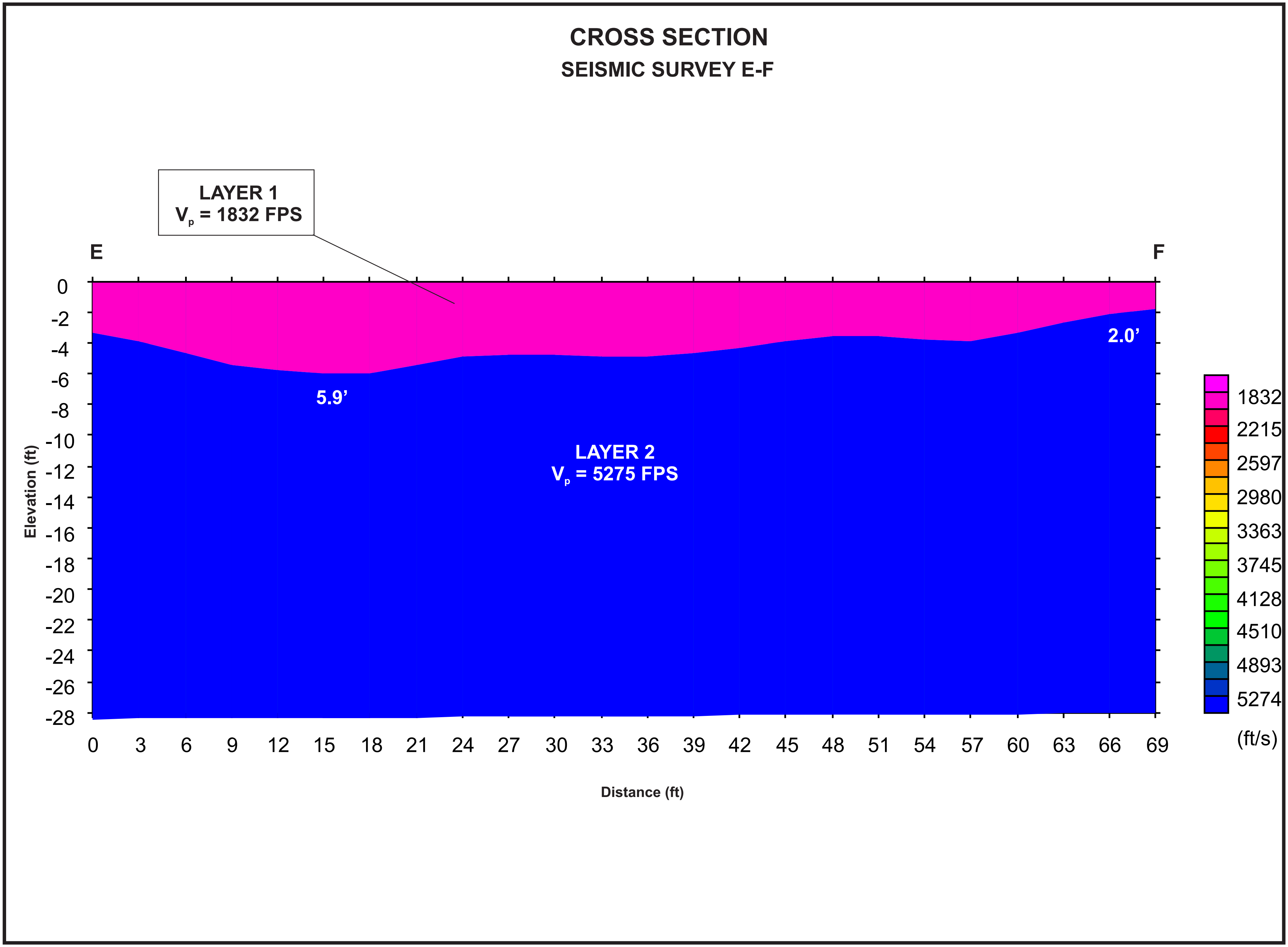
Layer 2: HIGHLY TO MODERATELY WEATHERED AND FRACTURED, POOR, WEAK SANDSTONE ROCK (PSM)

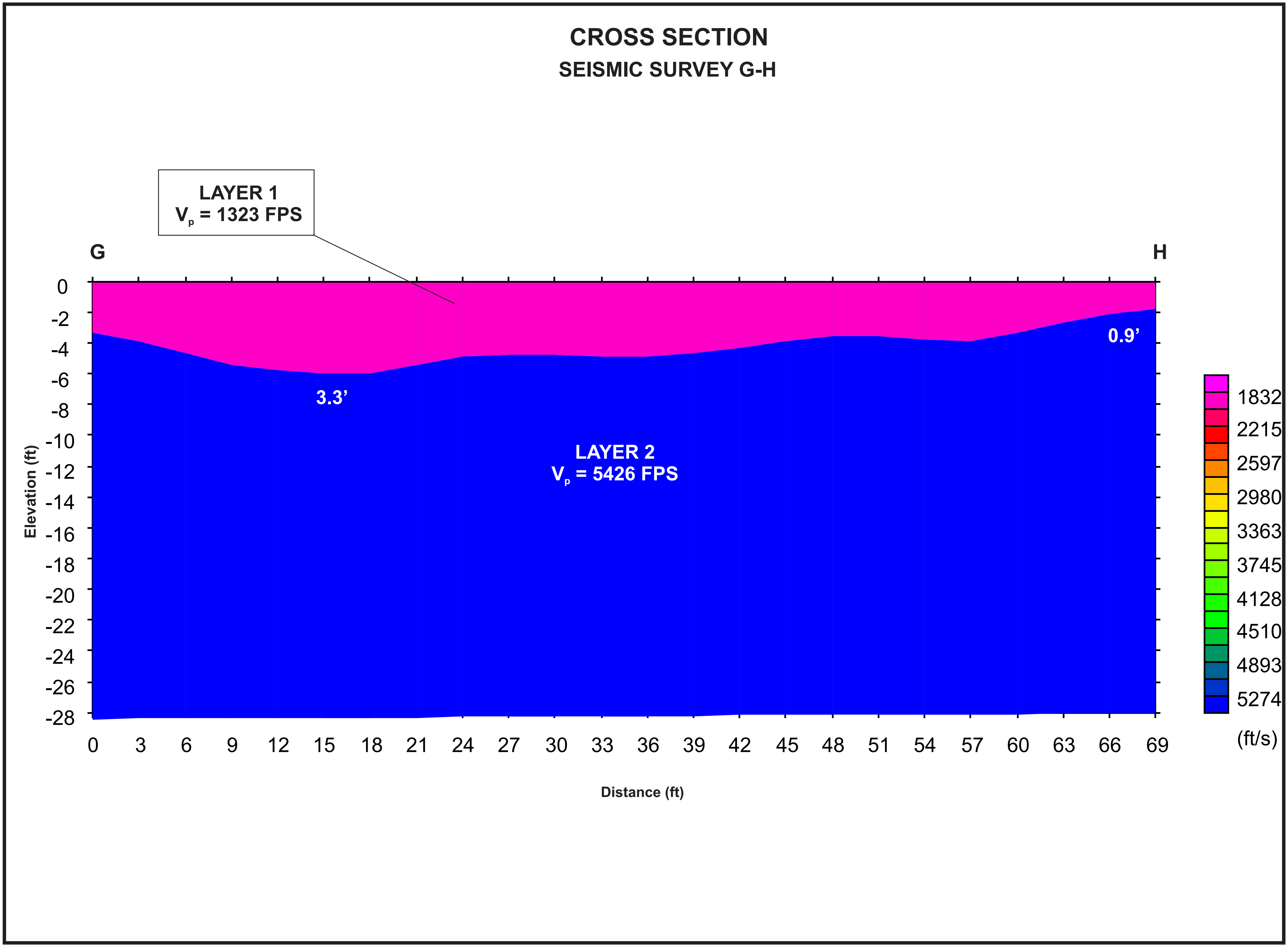
Layer 3: MODERATELY WEATHERED AND FRACTURED, FAIR, MODERATELY STRONG SANDSTONE ROCK (PSM)

Line	Layer 1			Layer 2			Layer 3		
	Velocity	Depth (ft)		Velocity	Depth (ft)		Velocity	Depth (ft)	
A-B	2128	-	-	5075	1.1	5.2	-	-	-
C-D	1579	-	-	-	-	-	7419	1.5	7.4
E-F	1832	-	-	5275	2.0	5.9	-	-	-
G-H	1323	-	-	5426	0.9	3.3	-	-	-
I-J	2457	-	-	-	-	-	6058	0.9	8.8
K-L	2809	-	-	4552	0.4	4.1	-	-	-
TB-1	-	-	-	-	4.0	-	-	-	-
TB-2	-	-	-	-	3.5	-	-	-	-
TB-3	-	-	-	-	3.5	-	-	-	-
Avgs	2021	-		5082	3.1		6739	4.7	


CROSS SECTION
SEISMIC SURVEY A-B
LAYER 1
V_p = 2128 FPS
A
B
LAYER 2
V_p = 5075 FPS
5.2'
1.1'
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57 60 63 66 69
Distance (ft)
2128
2456
2783
3111
3438
3766
4093
4420
4748
5074
(ft/s)


CROSS SECTION
SEISMIC SURVEY C-D
LAYER 1
V_p = 1579 FPS
C
D
7.4'
1.5'
LAYER 3
V_p = 7419 FPS
Elevation (ft)
Distance (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57 60 63 66 69
1579
2228
2877
3526
4175
4824
5473
6122
6771
7418
(ft/s)

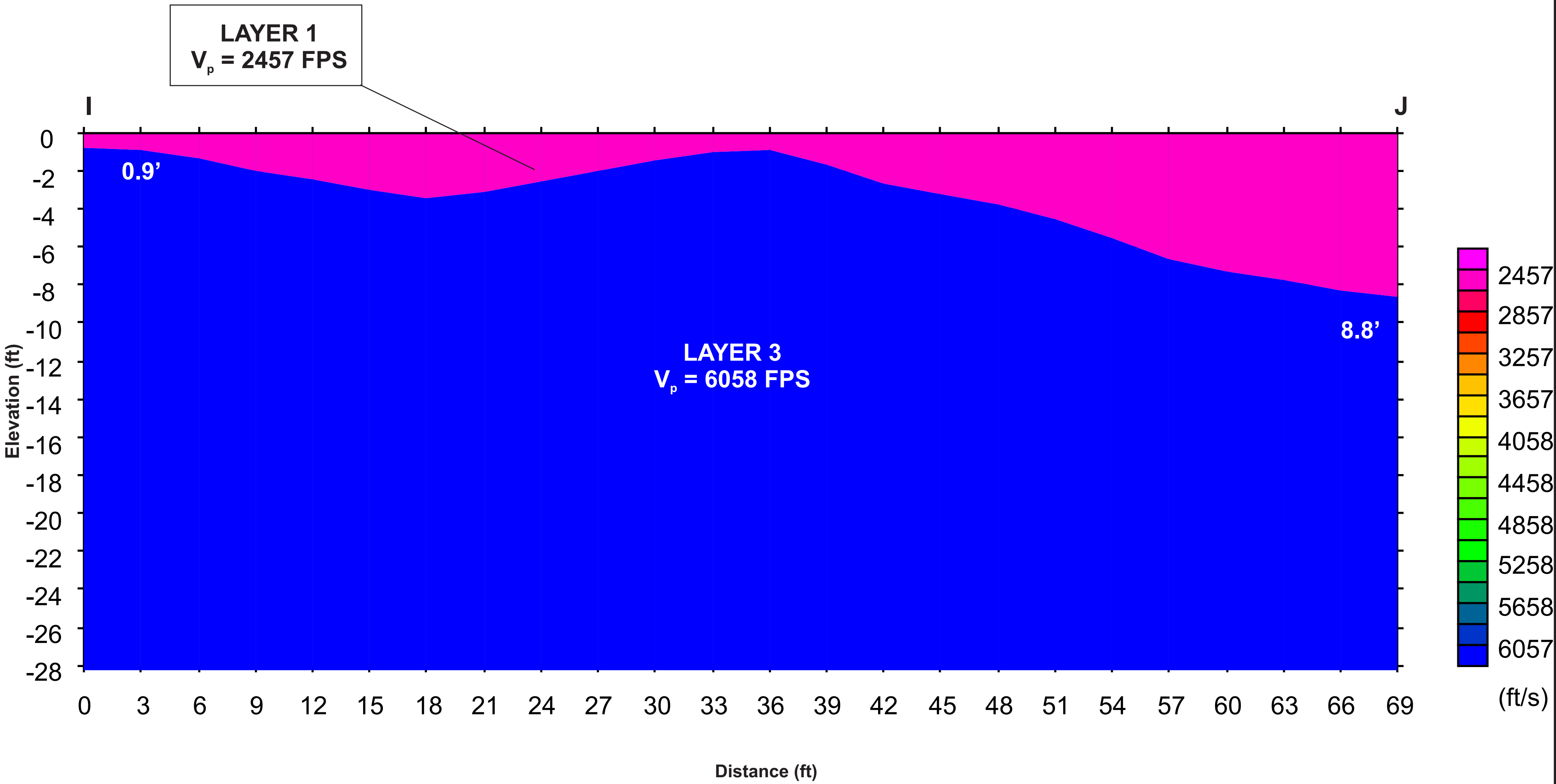

CROSS SECTION
SEISMIC SURVEY E-F
LAYER 1
V_p = 1832 FPS
E
F
5.9’
2.0’
LAYER 2
V_p = 5275 FPS
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57 60 63 66 69
Distance (ft)
1832
2215
2597
2980
3363
3745
4128
4510
4893
5274
(ft/s)

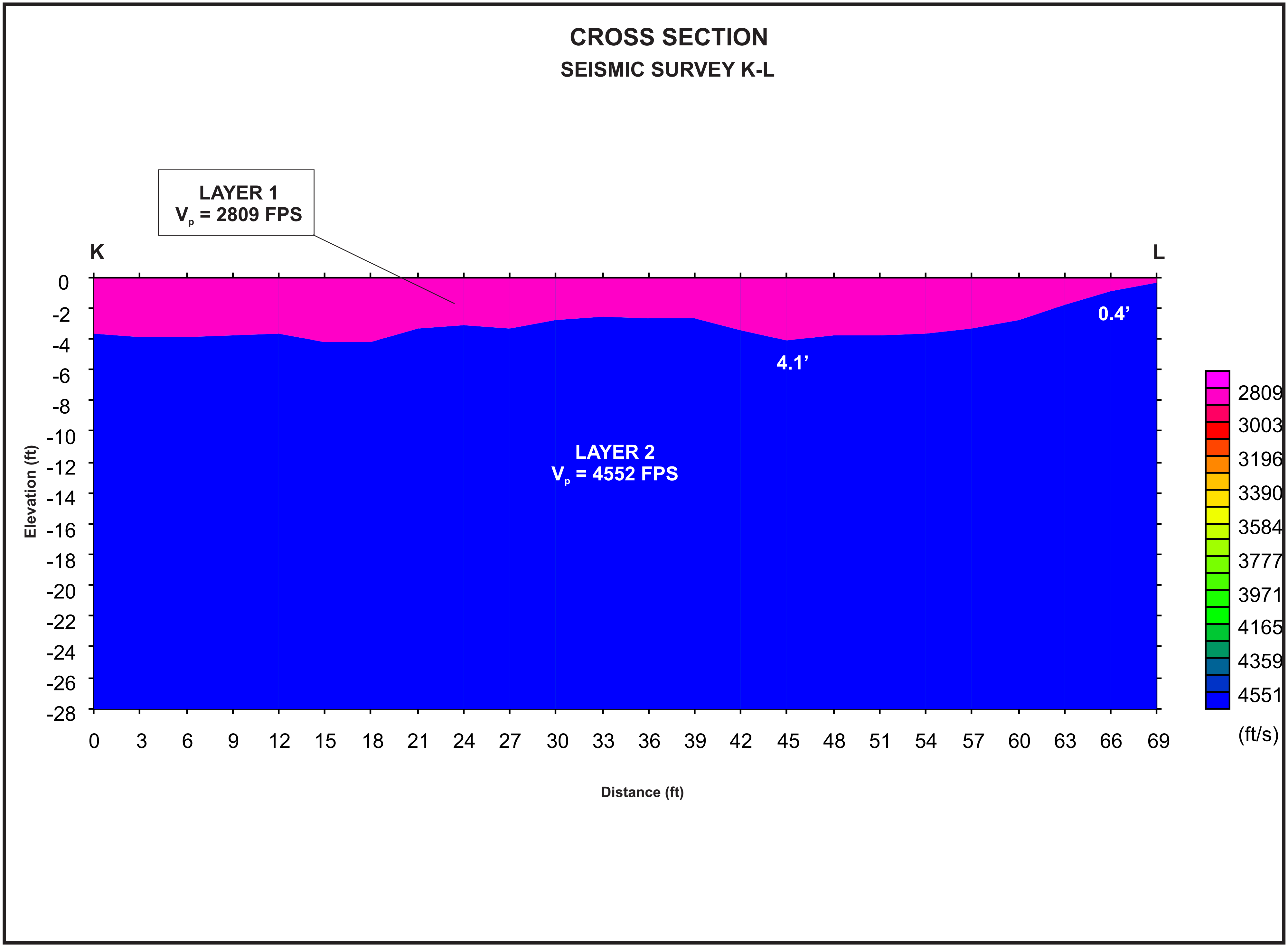


CROSS SECTION
SEISMIC SURVEY G-H
LAYER 1
V_p = 1323 FPS
G
H
0.9'
3.3'
LAYER 2
V_p = 5426 FPS
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57 60 63 66 69
Distance (ft)
1832
2215
2597
2980
3363
3745
4128
4510
4893
5274
(ft/s)

CROSS SECTION

SEISMIC SURVEY I-J

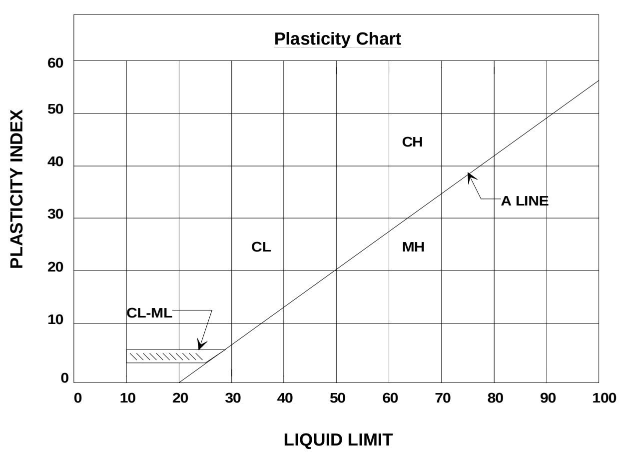



CROSS SECTION
SEISMIC SURVEY K-L
LAYER 1
V_p = 2809 FPS
K
L
0.4'
4.1'
LAYER 2
V_p = 4552 FPS
Elevation (ft)
Distance (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-22
-24
-26
-28
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57 60 63 66 69
2809
3003
3196
3390
3584
3777
3971
4165
4359
4551
(ft/s)

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.

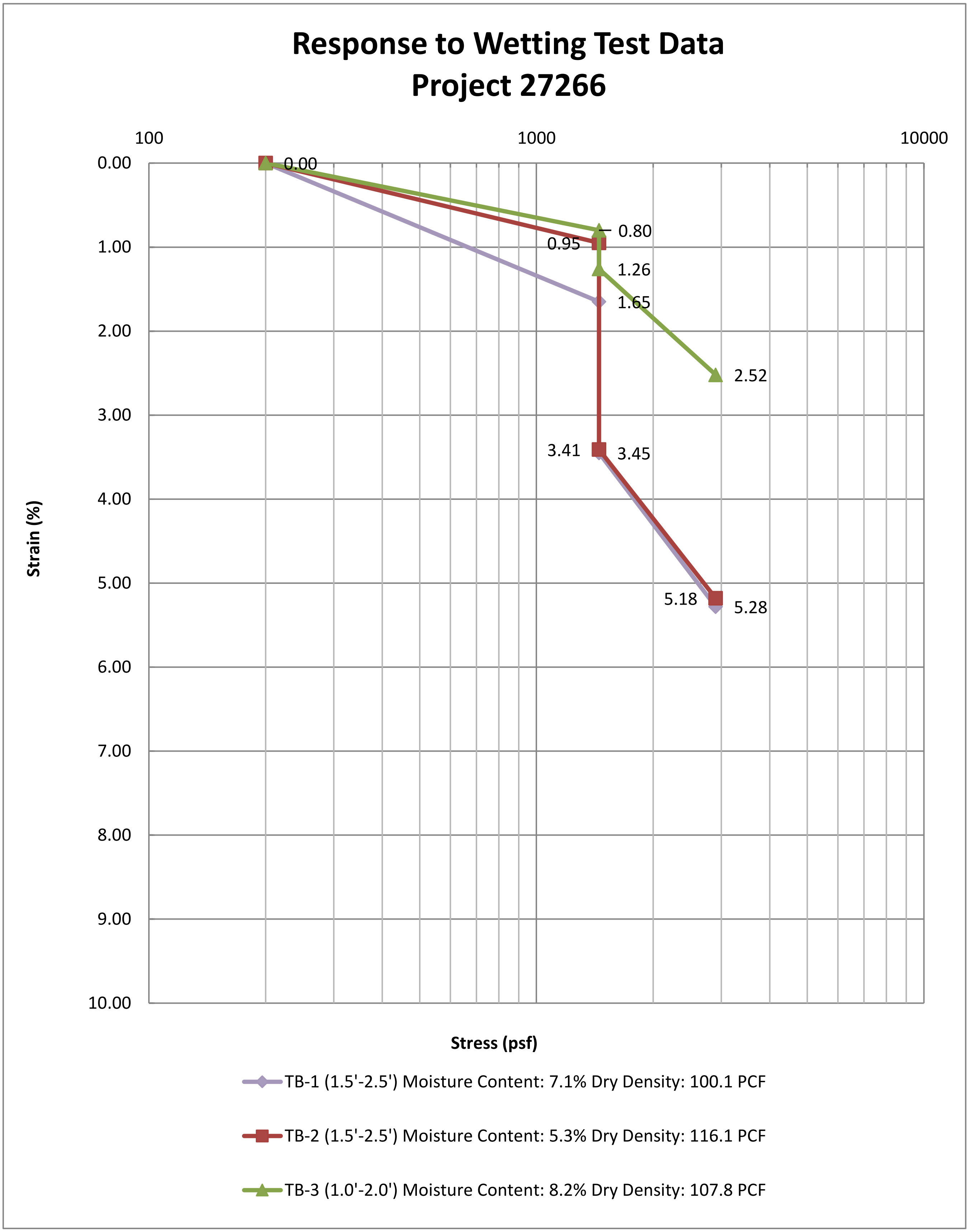
All geologic materials exhibit P-Wave velocities in certain ranges, which relate to the density, specific gravity, elastic modulus, and moisture content of the specific material. As a material density and specific gravity increase so does its P-Wave velocity. Similarly, an increase in moisture content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

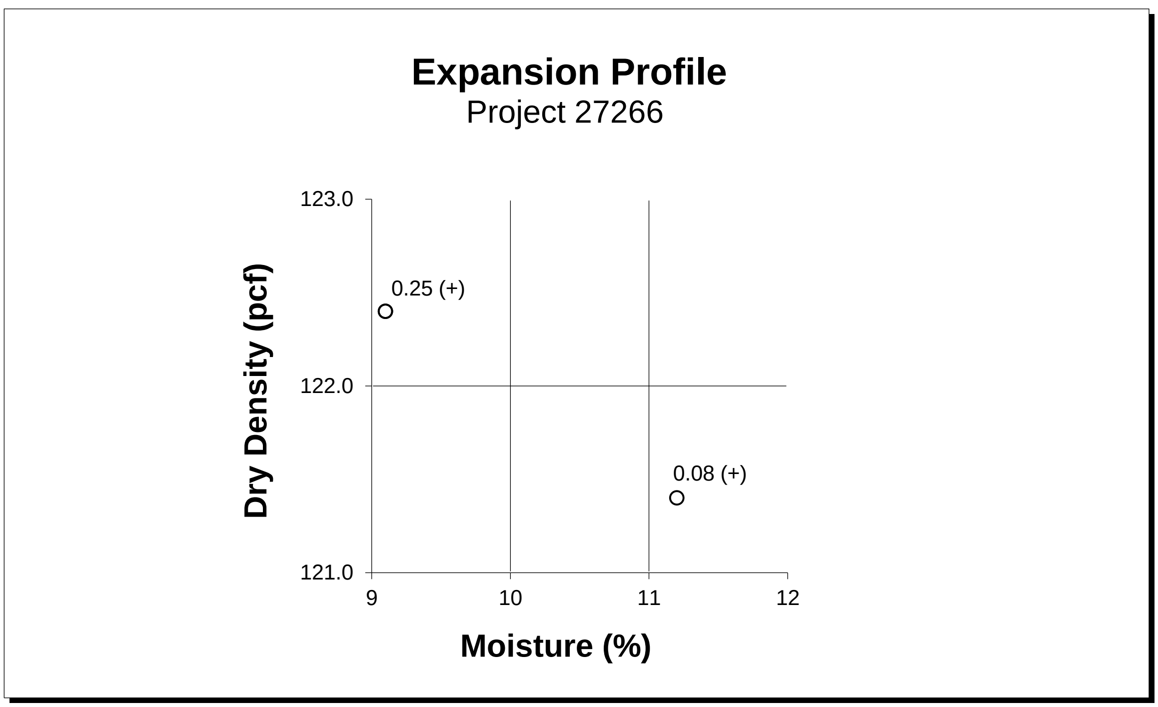

Response to Wetting Test Data
Project 27266
100
1000
10000
0.00
1.00
2.00
3.00
4.00
5.00
6.00
7.00
8.00
9.00
10.00
Strain (%)
Stress (psf)
0.00
0.80
0.95
1.26
1.65
2.52
3.41
3.45
5.18
5.28
TB-1 (1.5'-2.5') Moisture Content: 7.1% Dry Density: 100.1 PCF
TB-2 (1.5'-2.5') Moisture Content: 5.3% Dry Density: 116.1 PCF
TB-3 (1.0'-2.0') Moisture Content: 8.2% Dry Density: 107.8 PCF

EXPANSION TEST DATA

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY, SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Sample Location	***Remolded Moisture Content (%)***	***Dry Density (PCF)***	***Volume Change After Saturation (%)***	***Adjusted Volume Change After Saturation (%)***
SG-A (0.0'-2.0')	11.2	121.4	0.11 (+)	0.08 (+)
SG-A (0.0'-2.0')	9.1	122.4	0.34 (+)	0.25 (+)

(+) denotes expansion
(-) denotes compression

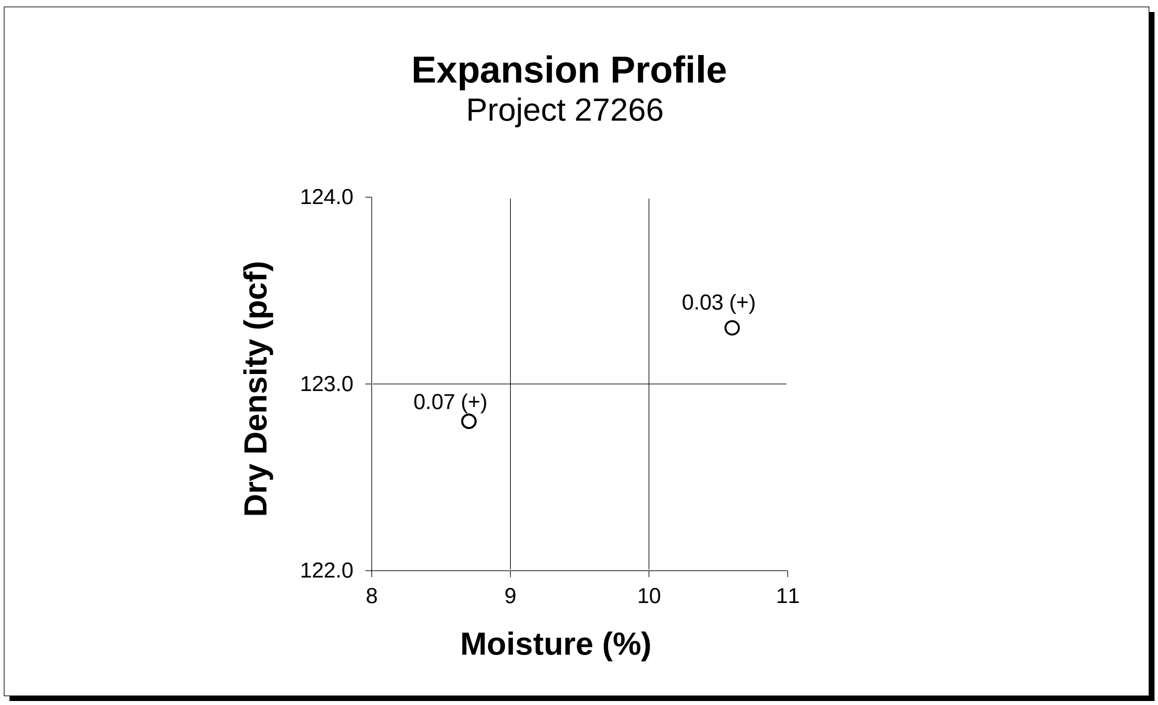


EXPANSION TEST DATA

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILTY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Sample Location	***Remolded Moisture Content (%)***	***Dry Density (PCF)***	***Volume Change After Saturation (%)***	***Adjusted Volume Change After Saturation (%)***
TB-1 (2.5'-3.5')	10.6	123.3	0.06 (+)	0.03 (+)
TB-1 (2.5'-3.5')	8.7	122.8	0.16 (+)	0.07 (+)

(+) denotes expansion
(-) denotes compression

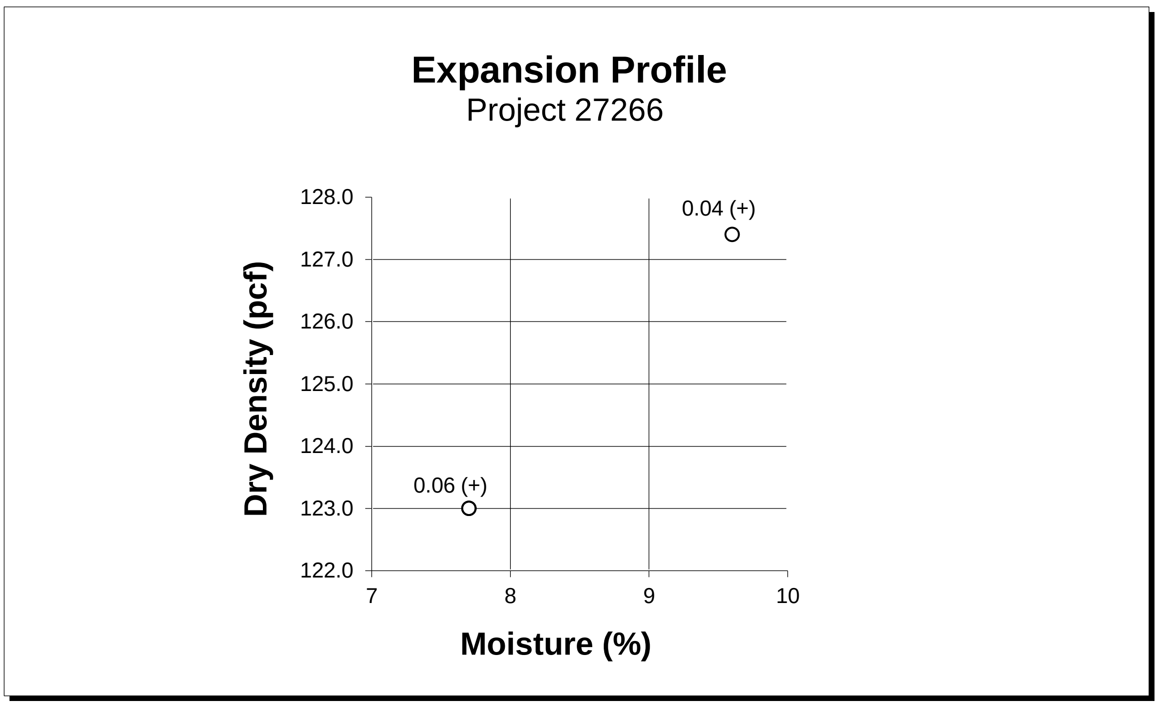


EXPANSION TEST DATA

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILTY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Sample Location	***Remolded Moisture Content (%)***	***Dry Density (PCF)***	***Volume Change After Saturation (%)***	***Adjusted Volume Change After Saturation (%)***
TB-2 (2.5'-3.5')	9.6	127.4	0.08 (+)	0.04 (+)
TB-2 (2.5'-3.5')	7.7	123.0	0.11 (+)	0.06 (+)

(+) denotes expansion
(-) denotes compression

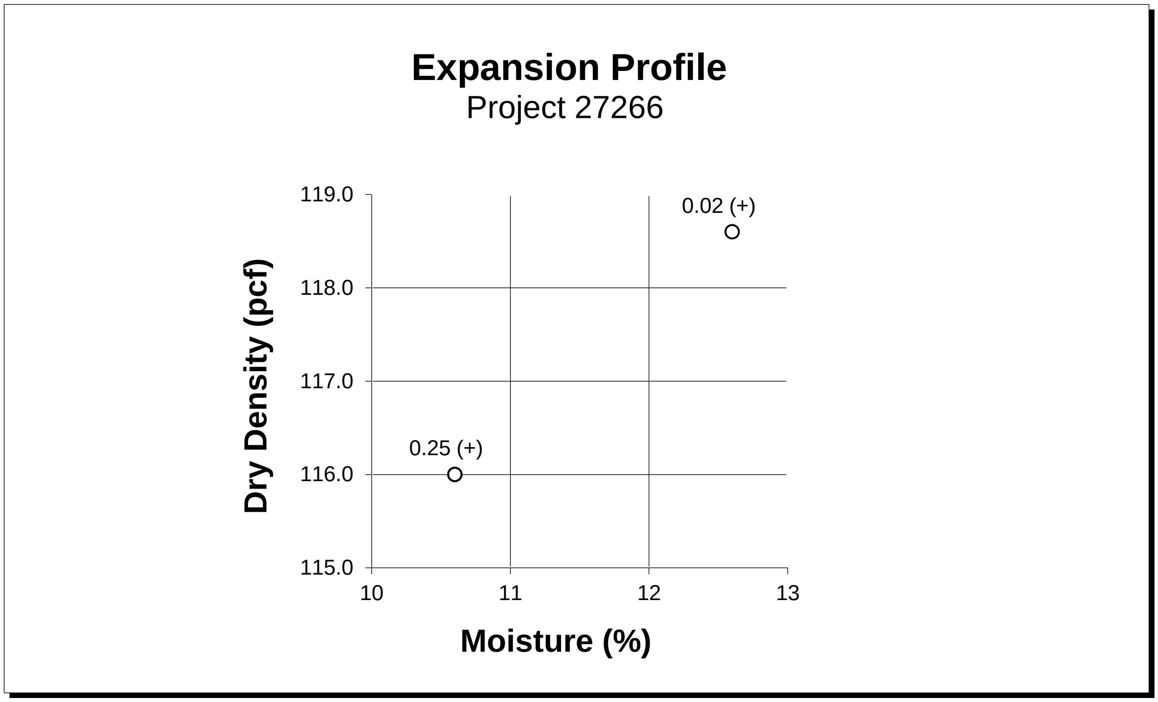


EXPANSION TEST DATA

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILTY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Sample Location	*Remolded Moisture Content (%)*	*Dry Density (PCF)*	*Volume Change After Saturation (%)*	*Adjusted Volume Change After Saturation (%)*
TB-3 (2.5'-3.5')	12.6	118.6	0.04 (+)	0.02 (+)
TB-3 (2.5'-3.5')	10.6	116.0	0.43 (+)	0.25 (+)

(+) denotes expansion
(-) denotes compression



CLASSIFICATION TEST DATA

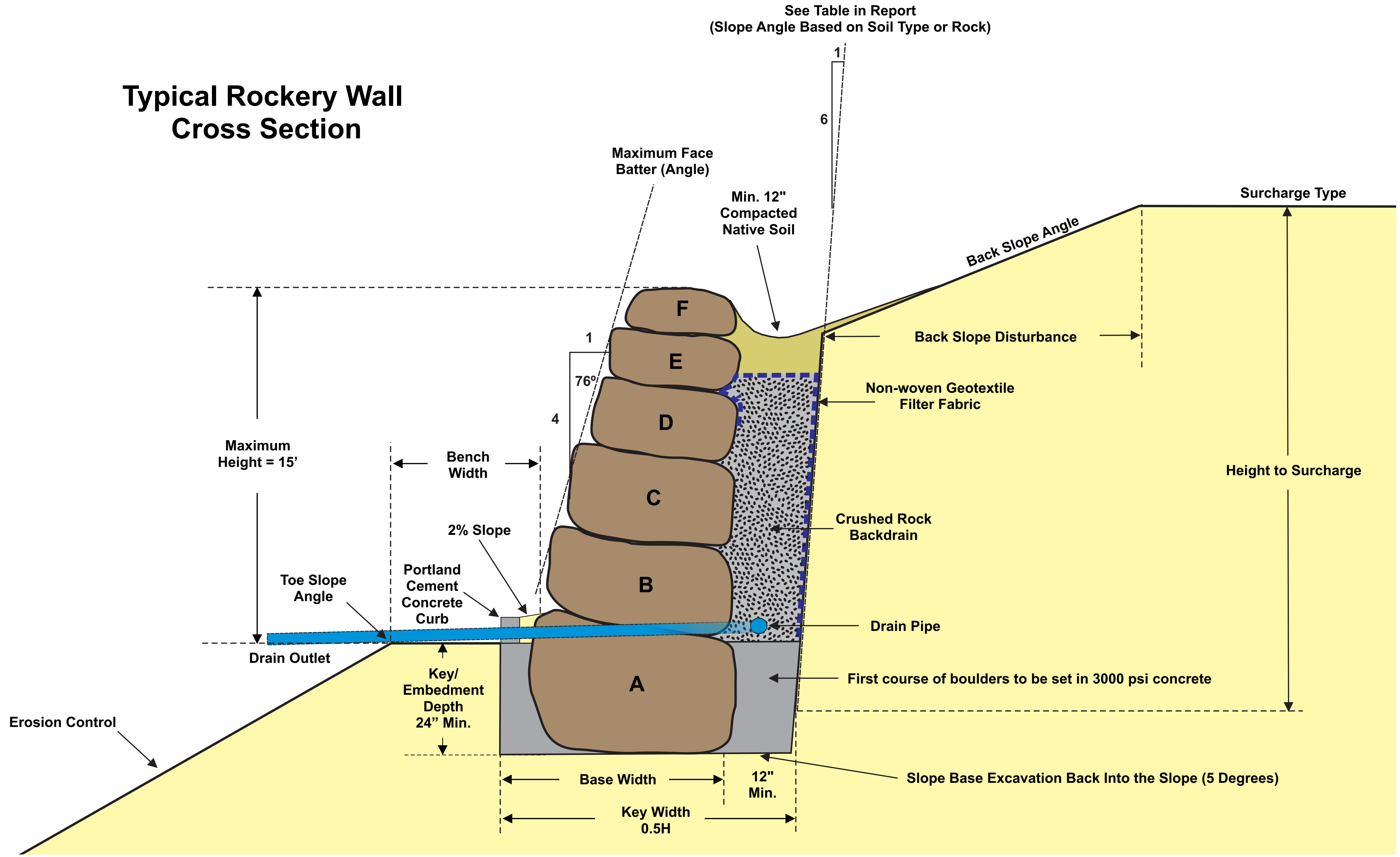
RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

Sample Location	***Sieve Analysis*** (% Passing Sieve Size)																		***Atterberg Limits***		***USCS***	***Moisture Content %***
	3”	***2”***	***1 ½”***	***1 1/4”***	***1”***	***¾”***	***½”***	***3/8”***	***¼”***	***#4***	***#8***	***#10***	***#16***	***#30***	***#40***	***#50***	***#100***	***#200***	***LL***	***PI***		
SG-A (0.0’-2.0’)	-	-	-	-	100	-	-	-	-	74	-	66	-	-	56	-	-	41	26	10	SC	7.7
TB-1 (2.5’-3.5’)	100	91	84	73	68	63	56	53	47	43	38	37	33	30	29	28	24	21	23	5	GC-GM	6.7
TB-2 (2.5’-3.5’)	100	88	82	77	74	68	63	60	55	51	45	44	39	36	34	33	29	24	23	4	GC-GM	5.0
TB-3 (2.5’-3.5’)	100	83	83	80	78	74	68	65	60	58	53	52	49	46	44	42	39	33	26	9	GC-GM	9.5

SECTION IV

9013 NORTH 24TH AVENUE, SUITE 7, PHOENIX, ARIZONA 85021
PHONE: 602.943.6997 ▪ VANNENGINEERINGINC.COM

Typical Rockery Wall Cross Section



WALL HEIGHT (FT)	ROCK SIZE DESIGNATION					
	A	B	C	D	E	F
3	3	2	-	-	-	-
6	4	3	2	-	-	-
9	5	4	3	2	-	-
12	6	5	4	3	2	-
15 (max.)	6	6	5	4	3	2

ROCK SIZE		
ROCK SIZE ID	ROCK WEIGHT (LBS)	AVERAGE DIMENSION (FT)
1	50 to 200	0.7 to 1.1
2	200 to 650	1.1 to 1.6
3	650 to 1950	1.6 to 2.3
4	1950 to 3900	2.3 to 2.9
5	3900 to 5850	2.9 to 3.3
6	5850 to 7800	3.3 to 3.7

TYPICAL ROCKERY WALL CROSS SECTION

SCALE: N.T.S.

PREPARED BY: MS

DATE: 8-28-19

RECOMMENDATIONS FOR SANDSTONE MATERIAL TYPE, RIPPABILITY,
SLOPE STABILITY AND RETAINING WALL OPTIONS
FOREST ROAD CONNECTION TO HIGHWAY 89A, NORTHWEST OF
HIGHWAY 89A AND HIGHWAY 179
SEDONA, ARIZONA

PROJECT 27266

VANN ENGINEERING INC

VANN ENGINEERING INC

Geotechnical Engineering Enviromental Consulting Construction Testing & Inspection

9013 North 24th Avenue, Suite 7, Phoenix, Arizona 85021-2851