VANN ENGINEERING INC

UPDATED GEOTECHNICAL INVESTIGATION REPORT

Proposed Custom Residence
APN 172-47-086, Stone Canyon, Lot 29
5338 East San Miguel
Paradise Valley, Arizona 85253

Prepared for:

Drew Bausom
The Construction Zone, LTD.
1729 East Osborn Road
Phoenix, Arizona 85016

December 5, 2024

Project 25355

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RE: Updated Geotechnical Investigation Report
Proposed Custom Residence
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Drew,

Transmitted herewith is a copy of the final report of the updated 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. 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 any required supplemental recommendations. 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 with 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 12/5/2024
ARIZONA, U.S.A.

Jeffry D. Vann, PhD PE D.GE F.ASCE
Principal Engineer

Distribution: Addressee via email, drew@czphx.com

TABLE OF CONTENTS

TABLE OF CONTENTS

SECTION I

1.0 INTRODUCTION

Vann Engineering, Inc. understands that a new custom residence is proposed for construction at the above-mentioned site, with no planned basement levels. The former residence has been razed. This document presents the results of a geotechnical investigation conducted by Vann Engineering, Inc. for the:

Proposed Custom Residence
APN 172-47-086, Stone Canyon, Lot 29
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The following aerial photograph shows the site (outlined in red) and the immediate vicinity.

Figure 1: Aerial photograph of the site (outlined in red) and the immediate vicinity

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 site, and 2) to provide final geotechnical recommendations. The maximum column and wall loads have been assumed to be as summarized below.

Table 1: Anticipated Loads

Foundation Type	Maximum Column Load (KIPS)	Maximum Wall Load (KLF)
Conventional surface-level spread foundations bearing on native undisturbed soil or engineered fill with total and differential settlements limited to ½ inch and ¼ inch, respectively.	100	5.0

Anticipated structural loads more than those stated above will need to be addressed in an addendum, since they are not covered by the scope of services of this effort.

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
- Site Plan indicating the locations of all points of exploration
- Recommendations for conventional surface-level spread foundations; allowable bearing capacity based on settlement analysis of ½ inch total settlement and ¼ inch differential settlement (allowable bearing pressure and depth for shallow spread foundations)
- General excavation conditions
- Lateral stability analyses including active pressure, passive pressure, and base friction
- Recommendations for fixed-end and free-end retaining walls
- Recommendations for site grading - necessary earthwork for conventional systems
- Recommendations for drainage and slab support
- Anticipated shrinkage of the surface soil
- Recommendations for swimming pool backfill
- Limited soil-related corrosion discussion
- IBC Seismic Site Classification

Note: This report does not include, either specifically or by implication, any environmental assessment of the site or identification of contaminated 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 have not been included in our scope of services.

Vann Engineering is not a corrosion engineering firm. A corrosion engineer must be consulted if the potential corrosion of construction materials, underground utilities, and structures is a concern. Additionally, any corrosion related laboratory testing 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.

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 **(Project 25355 dated September 30, 2024)** authorized by **Drew Bausom on November 11, 2024** 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 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, 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 a new custom residence is proposed for construction at the above-mentioned site, with no planned basement levels. The former residence has been razed.

2.2 Site Description

A review of historical aerial photographs shows that the site was occupied by a single-family residence, detached garage, block walls, pavement, swimming pool, spa, landscaping, and hardscape areas in 2022 (Figure 2). The site was razed and rough graded in 2023, leaving only a portion of the rock wall and 3.5 feet road cut on the western parcel boundary (Figure 3).

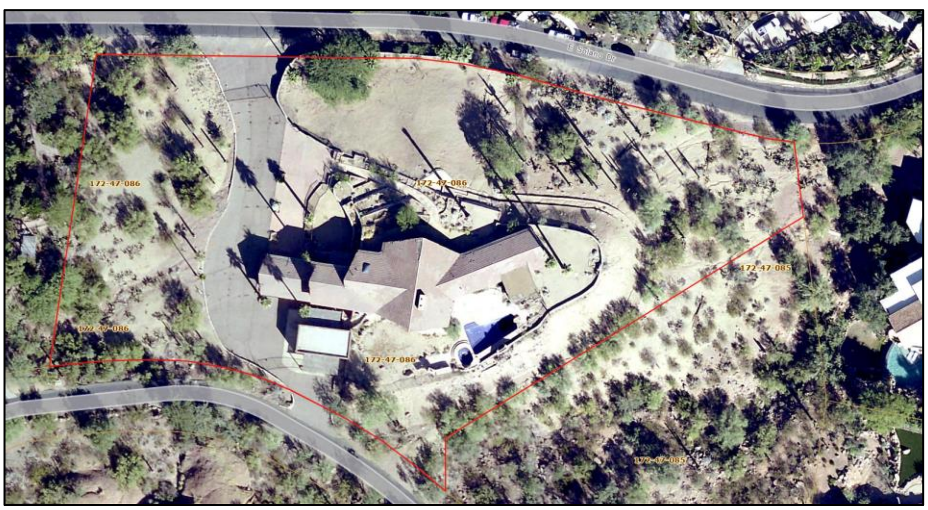


Figure 2: 2022 historical aerial photograph

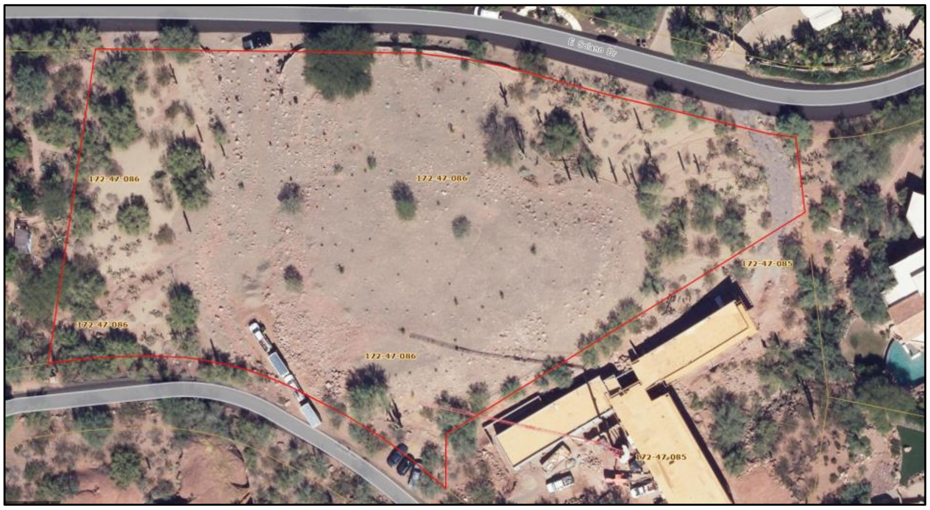


Figure 3: 2023 historical aerial photograph

Currently the disturbed portions of the site consist of generally flat topography that slopes gently down to the north. The site is sparsely vegetated with desert brush, cacti, and trees. The native undisturbed portions of the site slope down to the north-northeast. Roughly 2.0 to 3.0 inches of pea gravel were observed scattered across the former driveway location. Also, fragments of concrete and asphalt (associated with past demolition efforts) were observed scattered across the disturbed portion of the site. It should be noted that the maximum depth of the spread fill ranged in thickness from approximately 10.0 to 12.0 feet (based on visual observations made during the 2017 field investigation). In addition, approximately 8.0 to 19.0 inches of spread fill were encountered at the locations of the test borings and hand samples during the 2017 field investigation.

During the demolition of the previously existing residence, a significant portion of the existing spread fill believed to be 10.0 to 12.0 feet thick has been removed from some areas and spread out across other areas of the site. The previously existing swimming pool has been backfilled as well. At the locations of the most recent seismic survey lines (E-F, G-H, and I-J) roughly 2.0 to 6.0 feet of spread fill were detected. Note: Greater thicknesses of spread fill may be encountered at locations not specifically investigated by this firm.

It should be noted that the results for the most recent seismic survey lines indicate a lower overall density of Layer 1 as compared to the original site investigation. This is a result of the disturbance to the site during the demolition phase as well as the rough grading operation. The spread fill currently ranging in thickness from 2.0 to 6.0 feet has been rough graded and not properly moisture processed and compacted. As such, this firm considers the existing spread fill that is spread across the disturbed portions of the site (including the swimming pool/spa backfill), to be uncontrolled and uncompacted (undocumented), and must be removed in its entirety.

Over-sized aggregate (cobbles and small-sized boulders - particles that are greater than 3.0 inches) were observed scattered across the surface of the site and should be anticipated throughout Layer 1 (native undisturbed and existing spread fill soils). These oversized particles must not be used as structural fill.

The following images depict the site conditions at the time of our field effort:

Figure 4: General site conditions

Figure 5: General site conditions

Figure 6: General site conditions

Figure 7: General site conditions

3.0 SUBSURFACE INVESTIGATION AND LABORATORY TESTING

3.1 Subsurface Investigation

In 2017, the site's subsurface was explored through the utilization of two (2) exploratory test borings for examination of the subsurface profile to depths ranging from 10.0 to 15.0 feet below the existing site grade. A test boring depth shallower than 15.0 feet corresponds to the depth of auger refusal in highly to moderately weathered and fractured arkosic sandstone. In addition, the site's subsurface was explored through the utilization of two (2) hand-advanced test borings for examination of the subsurface profile to depths ranging from 1.0 to 2.0 feet below the existing site grade. A hand-advanced test boring depth shallower than 10.0 feet corresponds to the depth of auger refusal on highly to moderately weathered and fractured arkosic sandstone. The locations of the test borings are shown on the Site Plan in Section II of this report, and presented as TB-1, TB-2, HS-1, and HS-2.

The soils encountered were examined, visually classified and wherever applicable, sampled. Field logs were prepared for each test boring. The field logs contain visual classifications of the materials encountered during drilling as well as interpolation of the subsurface conditions between samples. Final logs, included in Section II, and tests of the field samples. The final logs describe the materials encountered, their thicknesses represent our interpretation of the field logs and may include modifications based on laboratory observation, and the locations where samples were obtained. The sample locations are noted graphically on the final logs. The Unified Soil Classification System was used to classify soils. The soil classification symbols are presented on the final logs and are briefly described in Section II.

Also in 2017, the site's subsurface was explored through the utilization of two (2) 24-channel refraction seismic survey lines, denoted on the Site Plan in Section II of this report. Each seismic survey line involved the retrieval of data in two separate directions (*forward and reverse*). As such, four (4) 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.

In 2024, the site's subsurface was explored through the utilization of three (3) refraction seismic survey lines, denoted on the Site Plan in Section II of this report. The seismic survey lines involved the retrieval of data in two separate directions (*forward and reverse*). As such, six (6) refraction seismic surveys were conducted at the site. The seismic survey lengths were 60.0 feet, thereby allowing an examination of the subsurface to a depth of 20.0 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.

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 to one-fifth 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.

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 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.

A representative sample obtained during the field investigation was subjected to the following laboratory analyses:

Table 2: Laboratory Testing

Test	Sample(s)	Purpose
Response to Wetting	Undisturbed native soils (1)	Settlement analysis and soil bearing capacity
Sieve Analysis, Atterberg Limits, and Moisture Content	Native Subgrade Soils (2)	Soil Classification
Soluble Sulfates and Chlorides	Native Subgrade Soils (1)	Limited Soil Corrosion Potential

Refer to Section III of this report for the complete results of the laboratory testing. The 2024 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 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, and seismic refraction data indicate the following physical and mechanical properties of the subsurface soil and rock:

Table 3: Site Stratigraphy

Layer	Depth of Occurrence[1]	Velocity Range (FPS)	Classification
1	Layer 1 currently occurs to depths ranging from 1.0 to 6.0 feet below the existing site surface at the locations of the test borings and seismic survey lines. Prior to the demolition effort, Layer 1 was encountered at depths ranging from 1.3 to 4.8 feet.	1019 to 1224 (Based on the post demolition site conditions)	Moderately dense coarse-grained alluvium and spread fill comprised of gravelly silty sand and gravelly sand, with fines (SC-SM)[2]
2	Layer 2 occurs below depths ranging from 1.0 to 6.0 feet from the existing site grade at the locations of the test borings and seismic survey lines	4124 to 5294	Highly to moderately weathered and fractured, poor, weak arkosic sandstone

[1]Average calculated depth below the existing site surface at the locations of the test borings and seismic surveys. 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.

[2]Over-sized aggregate (particle size that is greater than 3.0 inches) is scattered across the site surface and should be anticipated throughout Layer 1 during the earthwork process. Over-sized particles must not be used as structural fill.

Refer to the following tomographic cross sections and the general layered cross sections and test boring logs 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 Plan in Section II.

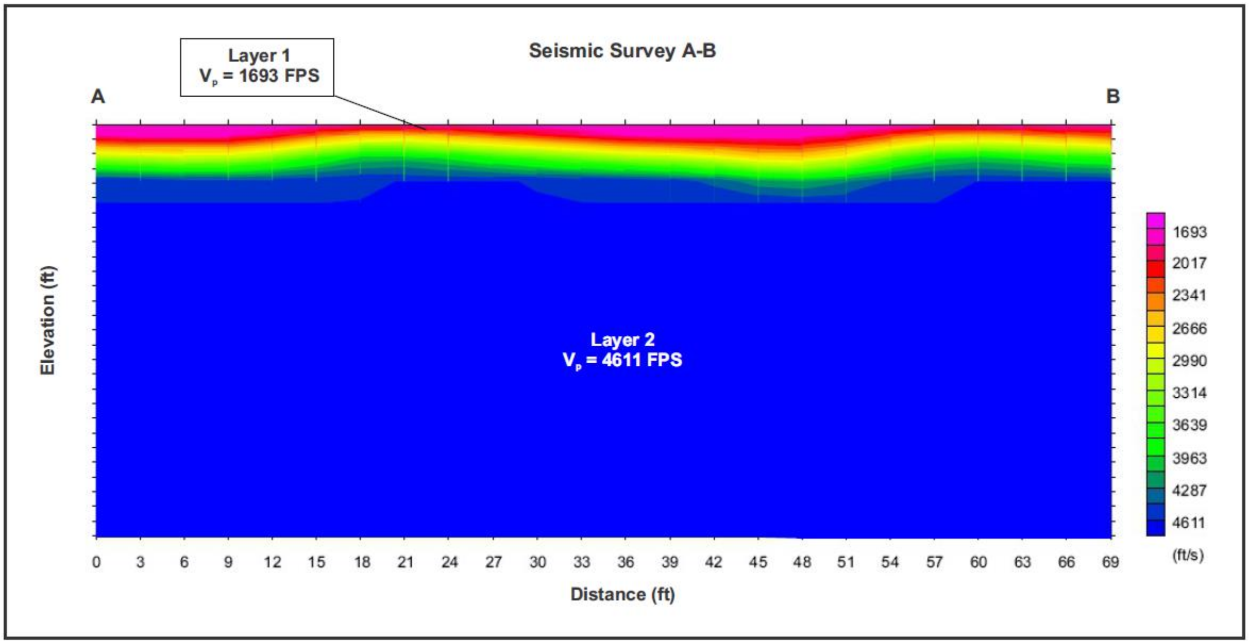


Figure 8: Tomographic cross section of Seismic Survey Line A-B

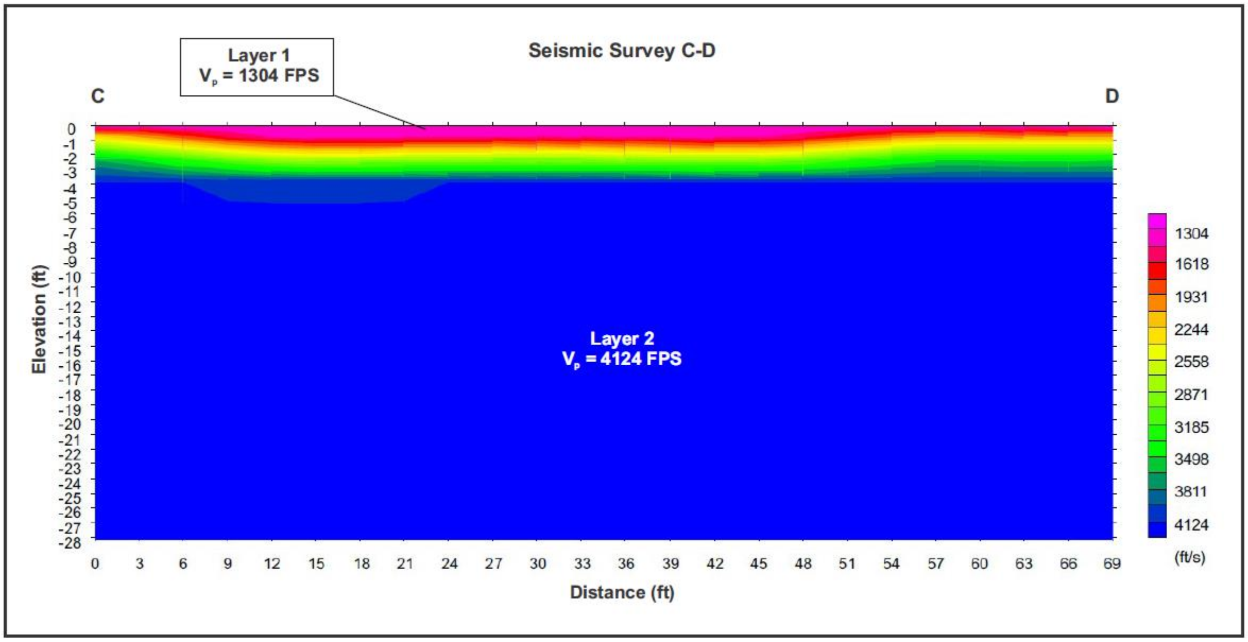


Figure 9: Tomographic cross section of Seismic Survey Line C-D

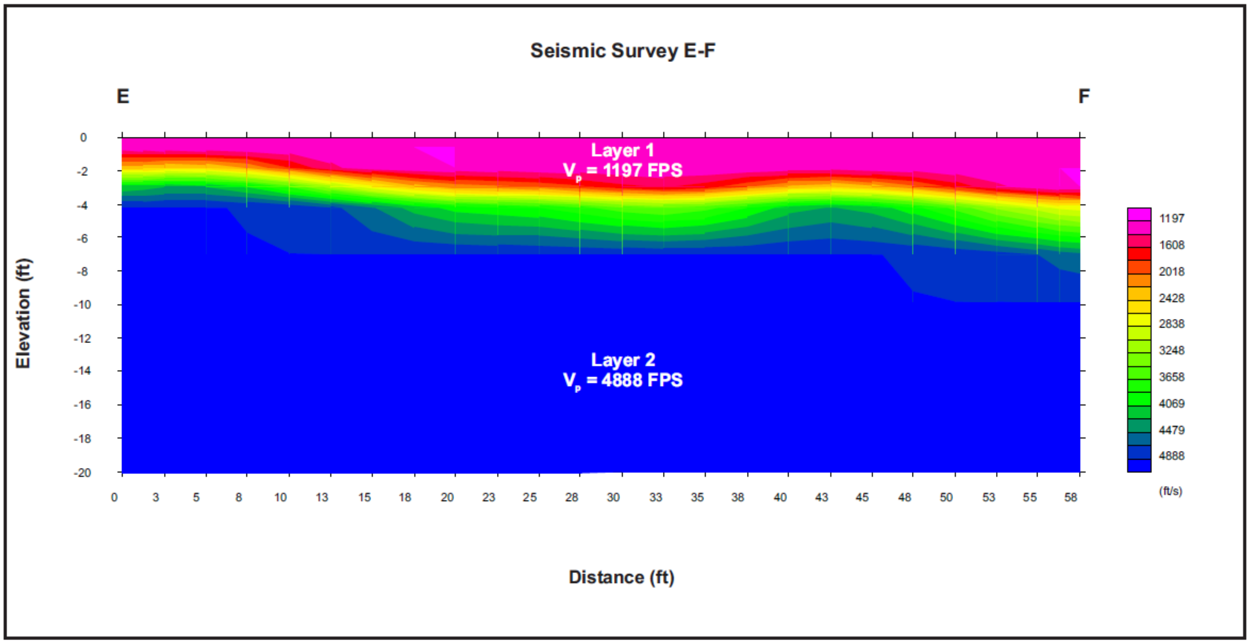


Figure 10: Tomographic cross section of Seismic Survey Line E-F

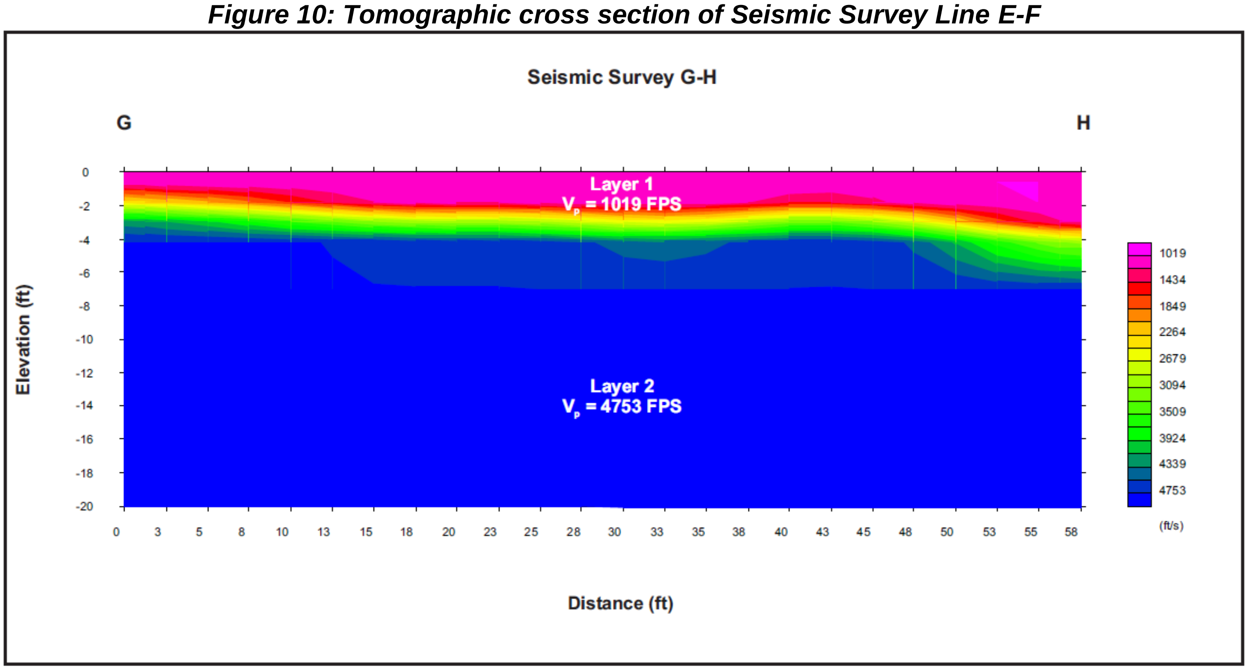


Figure 11: Tomographic cross section of Seismic Survey Line G-H

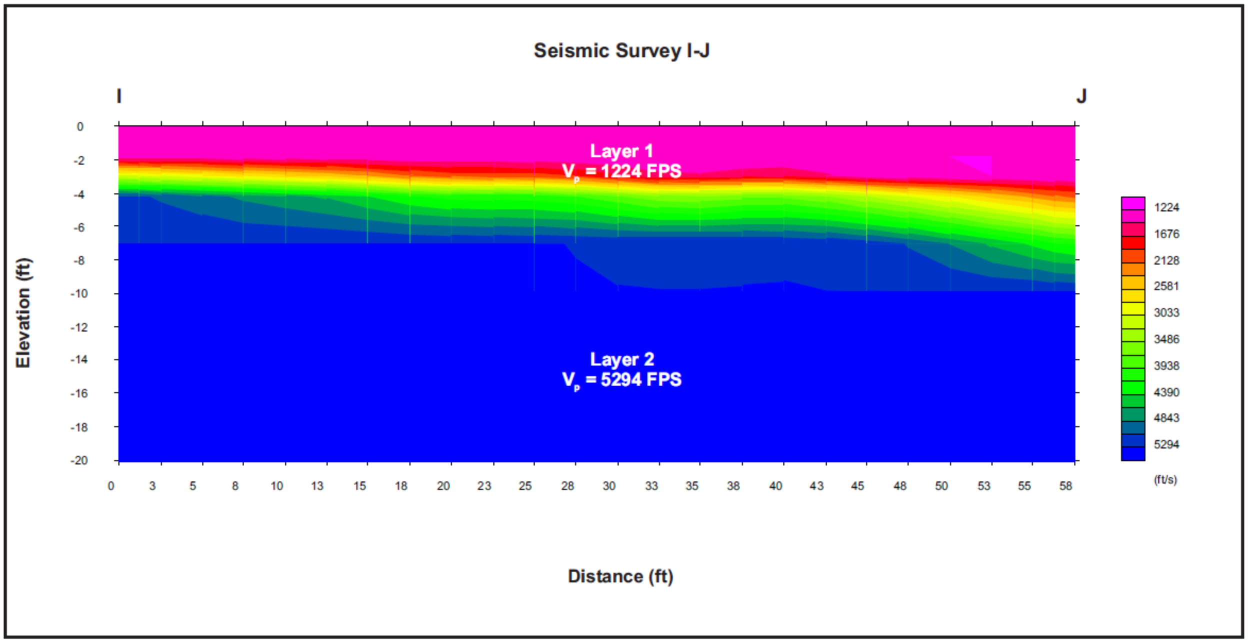


Figure 12: Tomographic cross section of Seismic Survey Line I-J

4.2 Local Geology

The local geology and our field investigation indicate that a thin layer of overburden soil (alluvium and spread fill - defined herein as Layer 1) overlies a rock mass comprised of highly to moderately weathered and fractured arkosic sandstone rock (defined herein as Layer 2).

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 native site soils encountered at the site are considered cohesionless based on the laboratory testing (i.e., plasticity index values of 5 and 7). Based on the laboratory data and measured soil properties, this firm has determined that the potential for soil expansion in conjunction with conventional applications is low.

Collapsible soils are typically comprised of silt and sand size grains with lesser 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 refraction data the native and existing fill soils encountered at the site are considered to have a high potential for collapse and excessive differential soil movement (mitigated by the foundation recommendations contained herein). The collapsible soils (denoted herein as Layer 1) extend to depths ranging from 1.0 to 6.0 feet at the locations of the seismic surveys and test borings.

Special note: This firm considers the existing spread fill that is spread across the disturbed portions of the site (including the swimming pool/spa backfill), to be uncontrolled and uncompacted (undocumented), and must be removed in its entirety.

It should be noted that the site soils (Layer 1), whether they are utilized for foundation support alone, or as engineered fill, will need to be recompacted through hand-tamping efforts, following the completion of the foundation excavation. This is necessary because of the inability of the site soils to maintain stability while withstanding the adverse effects of backhoe teeth. Hence the need for hand-tamping to regain soil bearing. Therefore, the bottom of the footing excavations must be hand-tamped to eliminate the probable adverse effects of the disturbance due to the backhoe. Prior to the placement of reinforcing steel, the base of all foundation excavations must be compacted with a "jumping jack" or plate tamper, resulting in compaction of the foundation bearing soils to a depth of 6.0 inches. The final compaction must be to at least 95% of the ASTM D698 maximum density. Some degree of moisture processing may be required to facilitate proper compaction, although no moisture specification will apply. This condition does not apply to foundations bearing on Layer 2 rock.

4.4 Groundwater

No groundwater was encountered during the course of this firm's site investigation. Groundwater is expected to be at a depth of approximately 216.3 feet according to nearest relevant well data in the area (GWSI Registry ID: 55-638750).

Also, refer also to the following Arizona Groundwater Site Inventory (GWSI) map for an approximate location of the site in relation to the nearby well.

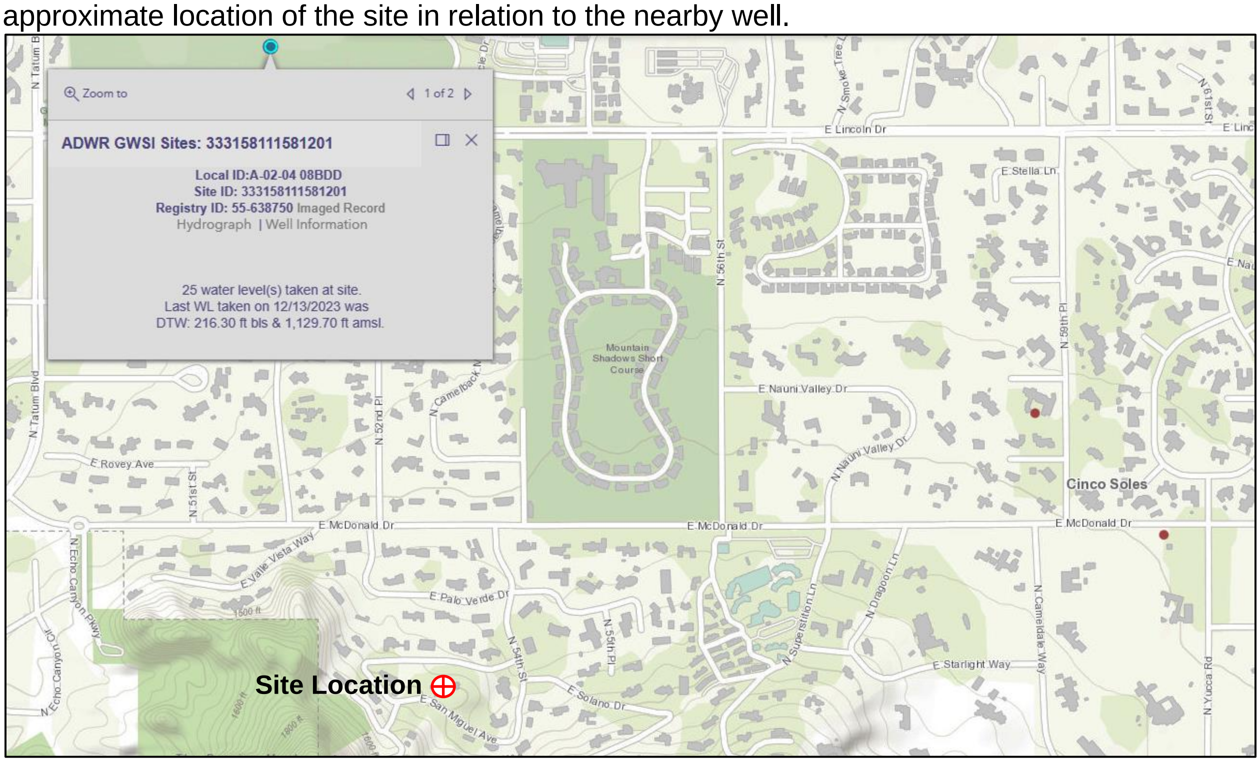


Figure 13: Groundwater Map

4.5 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 4: Soil Corrosion Test Results Summary

Sample Location	Depth Interval (feet)	Sulfate (%)	Chloride (ppm)
SG-B	0.5 – 1.5	0.089	10

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. Landscape material, including but not limited to decorative gravel, sand, and fill soils, may contain substantially higher concentrations of corrosive elements than the native site soils. The landscaping contractor must have all materials to be utilized in the landscape design tested for corrosion properties and submit the test results to the project general contractor for review prior to their use at the site.

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. 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.

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. and included in their "Handbook of Ripping."

Table 5: Excavating Conditions

Layer	Depth of Occurrence[1]	Seismic Wave Velocity (feet per second)	Remarks Relative to Rippability
1	Layer 1 currently occurs to depths ranging from 1.0 to 6.0 feet below the existing site surface at the locations of the test borings and seismic survey lines. Prior to the demolition effort, Layer 1 was encountered at depths ranging from 1.3 to 4.8 feet.	1019 to 1224 (Based on the post demolition site conditions)	Hard dig is not anticipated[2]
2	Layer 2 occurs below depths ranging from 1.0 to 6.0 feet from the existing site grade at the locations of the test borings and seismic survey lines	4124 to 5294	Hard dig (Refer to the Rippability Charts)

[1]Average calculated depth below the existing site surface at the locations of the test borings and seismic surveys. 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.
[2]Over-sized aggregate (particle size that is greater than 3.0 inches) is scattered across the site surface and should be anticipated throughout Layer 1 during the earthwork process. Over-sized particles must not be used as structural fill.

The subsurface soils (Layer 1) will be highly susceptible to sloughing. As such, we recommend that appropriate measures be incorporated into the final design and construction to avoid mishaps associated with caving.

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 (Layer 1) are considered to be OSHA Type C soil. Temporary excavations into Type C (Layer 1) soils are to be configured no steeper than a 1.5H:1V incline. Temporary excavations into Layer 2 rock are to be configured no steeper than a 1H:2V 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 rippability charts from the Caterpillar Performance Handbook and excavating conditions encountered at the site, are presented below.

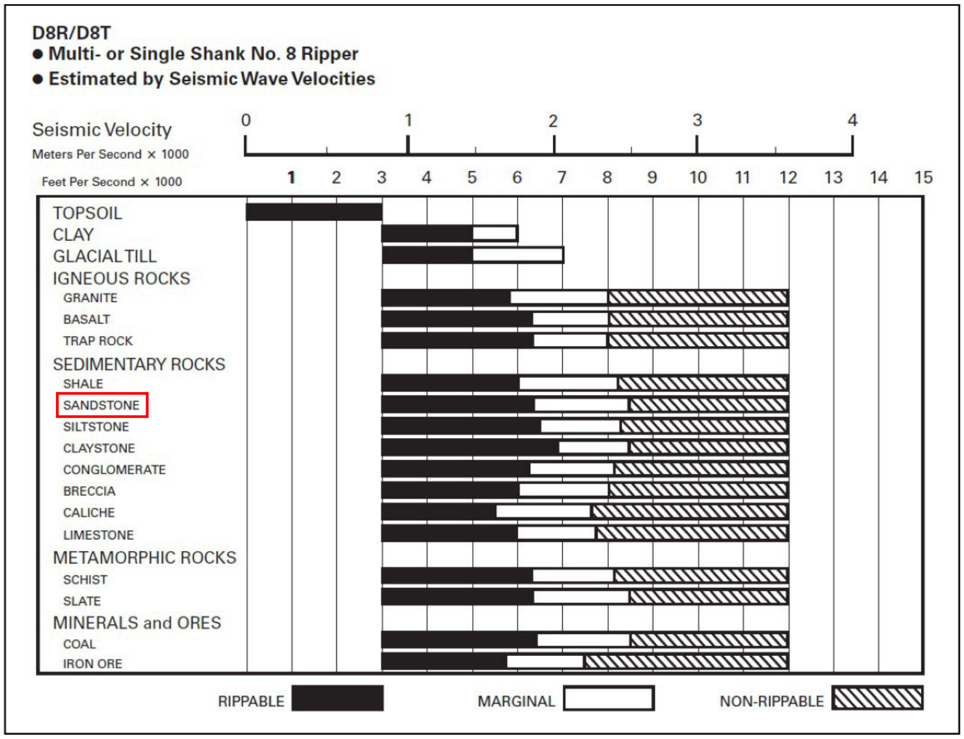


Figure 14: D8R/D8T Rippability Chart

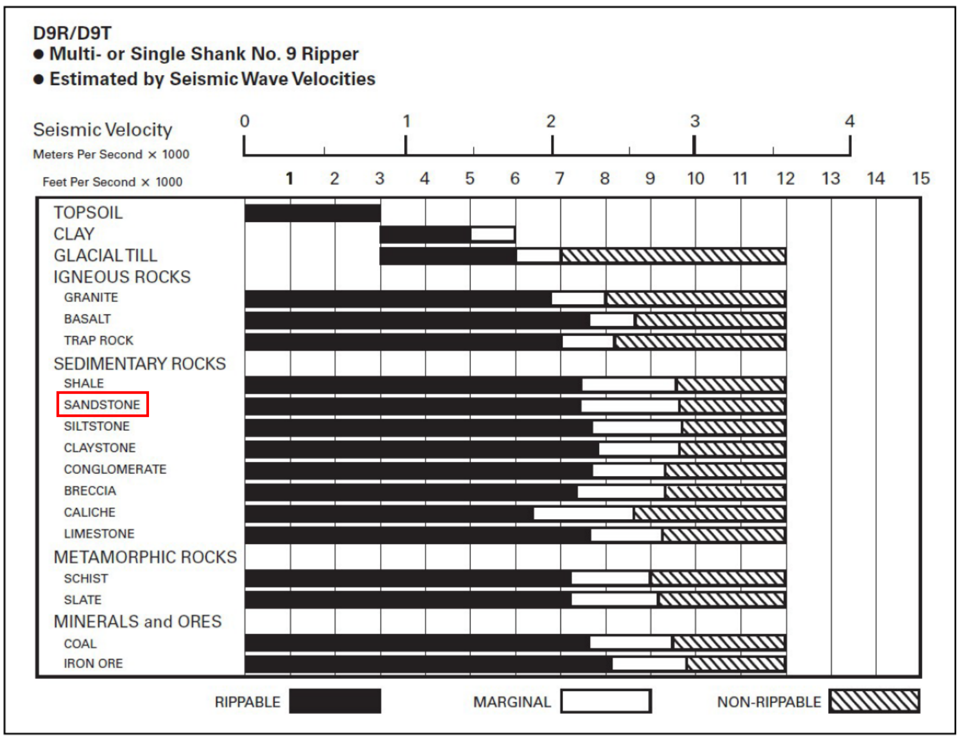


Figure 15: D9R/D9T Rippability Chart

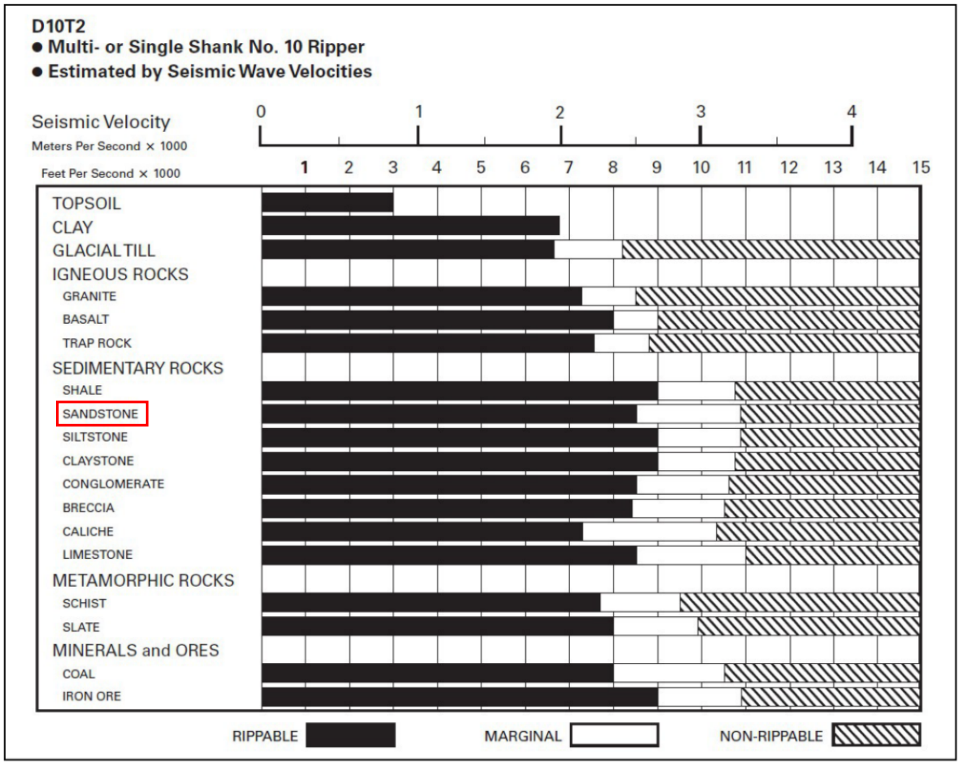


Figure 16: D10T2 Rippability Chart

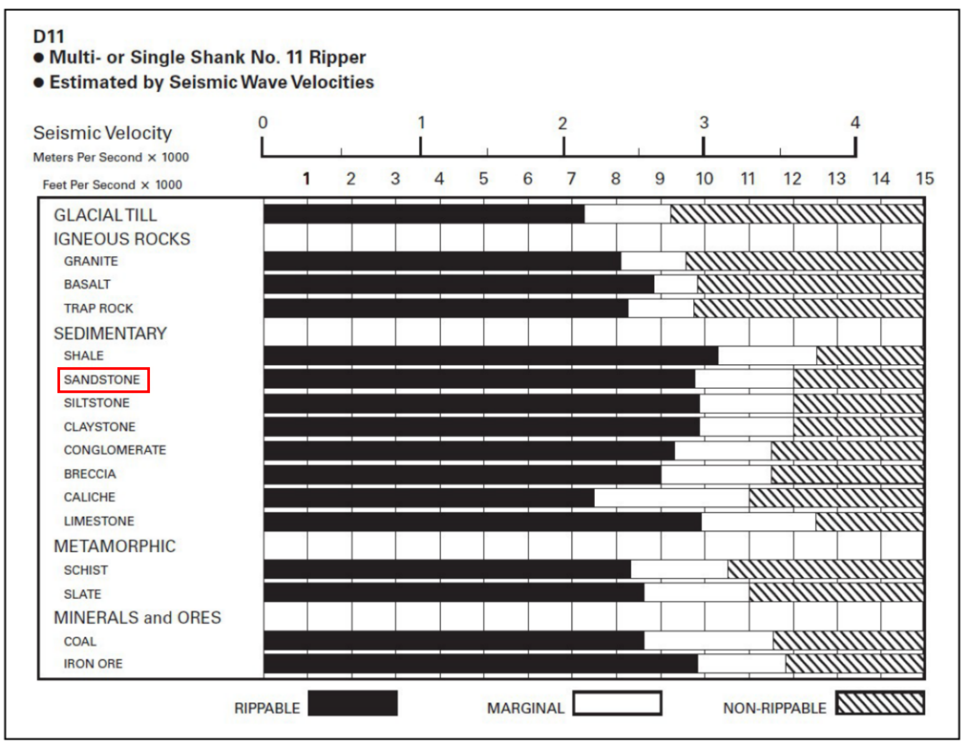


Figure 17: D11 Rippability Chart

5.2 Site Preparation

It is recommended that all vegetation, rock wall, asphalt fragments, concrete fragments, pea gravel, any remnants associated with the demolition of the former structures (inclusive of slabs, foundations, buried utilities, etc.), 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 might be encountered during construction. These features should be demolished or abandoned in accordance with the recommendations of the Vann Engineering, Inc. project geotechnical engineer. Such measures may include backfill with 2-sack ABC/cement slurry.

Following the removal of the above-listed items, all spread fill soils must be stripped from the proposed structure, pavement and hardscape areas as they are considered by this firm to be uncontrolled and uncompacted (undocumented). According to the results of the field effort, this will result in the removal of up to 1.0 to 6.0 feet of spread fill. Greater thicknesses of spread fill may exist at other locations on the site not explored by this firm, most notably at the location of the footprint of former structures. Native undisturbed soils must be exposed at the bottom and sides of the spread fill removal excavations. The presence of native undisturbed soils at the base of the spread fill removal excavations must be verified by a representative of this firm prior to backfilling.

Following the removal of the above listed items, at a minimum, the uppermost 8.0 inches of the surface soils must be reworked to establish a stable condition. The scarification and compaction requirements apply to cut situations as well as fill situations.

Any site cut soils may be reused as structural supporting fill provided that it is free of any and all vegetation and debris, the maximum particle size is 3.0 inches, and a suitable percentage of fines will be generated to ensure a stable mixture. All final compaction shall be as specified herein. **Over-sized aggregate (cobbles and small-sized boulders - particles that are greater than 3.0 inches) were observed scattered across the surface of the site and should be anticipated throughout Layer 1 (native undisturbed and existing spread fill soils). These oversized particles must not be used as structural fill.**

Special note for surface-level foundations:

It is necessary that a minimum of 1.5 feet of engineered fill lie beneath all conventional foundations for the structures in order to utilize the bearing capacity for engineered fill. The engineered fill should have a lateral extent of at least 3.0 feet beyond the edges of wall or column footing pads. If there is less than 1.5 feet of engineered fill beneath the footings, consider the bearing condition to be unacceptable. The base of the zone of subexcavation (cut surface below foundations) must be moisture processed and compacted to a depth of 8.0 inches.

It should be noted that the site soils (Layer 1), whether they are utilized for foundation support alone, or as engineered fill, will need to be recompacted through hand-tamping efforts, following the completion of the foundation excavation. This is necessary because of the inability of the site soils to maintain stability while

withstanding the adverse effects of backhoe teeth. Hence the need for hand-tamping to regain soil bearing. Therefore, the bottom of the footing excavations must be hand-tamped to eliminate the probable adverse effects of the disturbance due to the backhoe. Prior to the placement of reinforcing steel, the base of all foundation excavations must be compacted with a "jumping jack" or plate tamper, resulting in compaction of the foundation bearing soils to a depth of 6.0 inches. The final compaction must be to at least 95% of the ASTM D698 maximum density. Some degree of moisture processing may be required to facilitate proper compaction, although no moisture specification will apply. This condition does not apply to foundations bearing on Layer 2 rock.

Special note for pool abandonment:

If new surface level structures are to be constructed within the footprint of the currently existing swimming pool, removal of the existing swimming pool and backfill (if applicable) must be completed prior to and during the earthwork process. The following recommendations should be implemented:

If the pool shell is to be removed:

- Remove the pool backfill soils and pool shell
- Below 5.0 feet, backfill the pool in 6-inch lifts to 98% compaction and ± 2% of optimum moisture (D698A).
- Upper 5.0 feet, backfill the pool in 6-inch lifts to 95% compaction and ± 2% of optimum moisture (D698A). The upper 5.0 feet of backfill must be benched into the native soils.

If the pool shell is to be left in-place:

- Remove the pool backfill soils
- Remove the upper 3.0 feet of the pool shell.
- Perforate the bottom of the pool with 6-inch core holes to allow for drainage. The locations of the cores should be placed on 5.0 feet on-center, each way.
- Below 5.0 feet, backfill the pool in 6-inch lifts to 98% compaction and ± 2% of optimum moisture (D698A).
- Upper 5.0 feet, backfill the pool in 6-inch lifts to 95% compaction and ± 2% of optimum moisture (D698A).

Any foundations traversing the pool backfilled area should be double-reinforced (top and bottom) and tied to the slab, wherever possible. The double reinforcement should extend 10.0 feet past the limits of the pool and basement backfill area. Refer to Section IV for the Swimming Pool Removal and Backfill Detail.

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. All fills shall be properly moistened and compacted as specified in the section on compaction and moisture recommendations. All subbase fill required to bring the structure areas up to subgrade elevation should be placed in horizontal lifts not exceeding 6.0 inches compacted thickness or in horizontal lifts with thicknesses compatible with the compaction equipment utilized.

Fill placement in wash areas, trench 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.

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). An inspection of the site should be performed during the grubbing process to ensure that all applicable materials have been removed.

To avoid distress due to differential settlement, we recommend that all foundations bear on a like stratum, or strata that will produce similar settlements, and that all foundations use the same bearing capacity throughout the project.

It is the understanding of this firm that various utility trenches may traverse the completed pad(s). The backfill of all utility trenches, if not in conformance with this report, may adversely impact the integrity of the completed pad(s). This firm recommends that all utility trench backfill crossing the pad(s) 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 engineered 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 and moisture criteria prior to concrete placement:

Table 6: Compaction Requirements

Material	Building Area	Percent Compaction (ASTM D698)	Compaction Moisture Content Range (%)
On-site soils and import fill material with 12 ≤ PI < 15	Below Foundation Level	95 min	optimum -1 to optimum +3
	Above Foundation Level[1]	92 - 97	optimum -2 to optimum +2
On-site soils and import fill material with PI < 12	Below Foundation Level	95 min	optimum -2 to optimum +2
	Above Foundation Level[1]	95 min	optimum -2 to optimum +2
Base course	Below Interior Concrete Slabs	95 min	-

[1]Also applies to the subgrade in exterior slab, sidewalk, curb, gutter, and pool deck 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 5.0 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 (engineered) 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 7: Imported Fill Soil Parameters

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

*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.

Please note that all imported fill material is to be tested for soluble sulfate and chloride content (corrosion testing). Results of the corrosion testing must be presented to the project structural engineer in order to utilize the appropriate exposure class per Table 19.3.1.1 of Section 318 of the American Concrete Institute (ACI) Building Code Requirements for Structural Concrete. All concrete for the project should be designed (by others) in accordance with the provisions presented in Section 318, Chapter 19 of the ACI Building Code Requirements for Structural Concrete.

Water settling and/or slurry shall not, in any case, be used to compact or settle surface soils, fill material, or trench backfill within 10.0 feet of a structure area or within an area, which is to be paved. When trench backfill consists of permeable materials that would allow percolation of water into a structure or pavement area, water settling shall not be used to settle such materials in any part of the trench.

5.3 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.

5.4 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). Assuming the average degree of compaction will approximate 97 percent of the standard maximum density, the approximate shrinkage of the reworked on-site soil (Layer 1) are as follows:

Table 8: Shrinkage

Material	Estimated Shrinkage (Based on ASTM D698A)
On-Site Soil (Layer 1)	18% ± 3

The above value does not consider 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.

5.5 Site Classification

This project is not located over any known active faults or fault associated disturbed zones. Please refer to the following table contained in ASCE 7:

Table 9: ASCE 7 Section 20.3 Table 20.3-1 Site Classification

Site Class		$\overline{V}_s$	$\overline{N}$ or $\overline{N}_{ch}$	$\overline{S}_u$
A	Hard Rock	>5,000 ft/s	NA	NA
B	**Rock**	**2,500 to 5,000 ft/s**	**NA**	**NA**
C	Very Dense Soil and Soft Rock	1,200 to 2,500 ft/s	>50 blows/ft	>2,000 lb/ft^2
D	Stiff Soil	600 to 1,200 ft/s	15 to 50 blows/ft	1,000 to 2,000 lb/ft^2
E	Soft Clay Soil	<600 ft/s	<15 blows/ft	<1,000 lb/ft^2
		Any profile with more than 10 feet of soil that has the following characteristics: • Plasticity Index PI>20 • Moisture Content w≥40% • Undrained Shear Strength $\bar{S}_u$ <500 lb/ft^2		

The formula to determine the representative seismic shear wave velocity is defined below:

$$\overline{V_s} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

Where d_s is the total thickness (uppermost 100 feet), V_{si} is the shear wave velocity measured in the field, and d_i is the thickness of any layer between 0 and 100 feet.

It is assumed that the shear wave value will only increase with depth, as stated above based on the known geologic conditions at the site. Therefore, based on the shear wave velocity results and the known local geologic conditions at the site the calculation for the representative is shown below.

$$\overline{V_s} = \frac{100\,ft}{\frac{6\,ft}{710\,fps} + \frac{94\,ft}{3071\,fps}}$$

$$\overline{V_s} = 2560\,fps$$

By calculation of the shear wave, the representative shear wave velocity equals 2560 feet per second for the uppermost 100 feet. The IBC Site Class **B** may be utilized.

5.6 Conventional Surface-Level Spread Foundations

To avoid distress due to differential settlement, we recommend that all foundations bear on a like stratum, or strata that will produce similar settlements, and that all foundations use the same bearing capacity throughout the project.

It is recommended that all perimeter foundations and isolated exterior foundations bearing on 1.5 feet of engineered fill that have been hand-tamped post footing excavation be embedded a minimum of 1.5 feet below the lowest adjacent finish pad grade within 5.0 feet of proposed exterior walls. Interior footings bearing on 1.5 feet of engineered fill that have been hand-tamped post footing excavation should be founded a minimum of 1.5 feet below finish floor level.

Foundations bearing on native undisturbed soil that have been hand-tamped post footing excavation must be embedded a minimum depth of 3.0 feet.

Foundation excavations may be terminated upon contact with Layer 2 rock provided an adequate foundation depth has been achieved (to be field verified by a representative of this firm). Where footings will bear on Layer 2, foundations must have a minimum footing embedment of 1.5 feet.

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

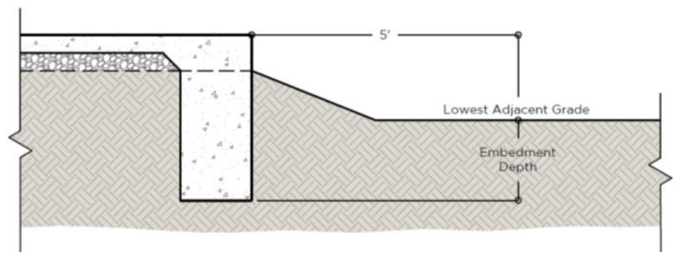
Table 10: Conventional Surface Level Foundations

Foundation Embedment Depth[1]	Bearing Stratum[2,7]	Allowable Soil Bearing Capacity[3]
3.0 Feet	Native undisturbed soil that has been hand-tamped post footing excavation[4, 6, 8]	1500 PSF
1.5 Feet	1.5 feet of engineered Fill that has been hand-tamped post footing excavation[5, 6, 8]	1500 PSF
Bearing at the surface of Layer 2, with a minimum footing embedment of 1.5 feet	Layer 2 occurs below depths ranging from 1.0 to 6.0 feet from the existing site grade at the locations of the seismic surveys and test borings	4000 PSF

[1]Conditions for foundation embedment depth:

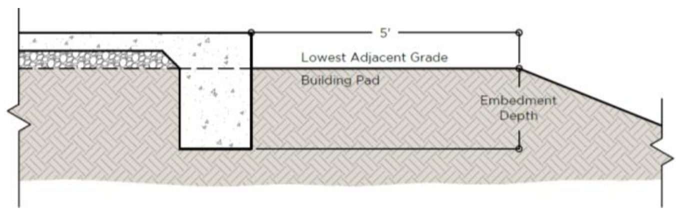
a) The depth below the lowest adjacent exterior pad grade within 5.0 feet of proposed exterior walls.

Condition A



b) The depth below finished compacted pad grade provided that a sufficient pad blow-up (the lateral extent to which the building pad is constructed beyond the limits of the exterior walls or other structural elements, inclusive of exterior column foundations) has been incorporated into the grading and drainage scheme (5.0 feet or greater);

Condition B



c) The depth below finish floor level for interior foundations.

[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 stratum.

[3]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.

[5]It is necessary that a minimum of 1.5 feet of engineered fill lies beneath all foundations for the structures in order to utilize the bearing capacity for engineered fill. The engineered fill should have a lateral extent of at least 3.0 feet beyond the edges of all footings. If there is less than 1.5 feet of engineered fill beneath the footings, consider the bearing condition to be unacceptable. The base of the zone of subexcavation (cut surface below foundations) must be moisture processed and compacted to a depth of 8.0 inches.

[6]It should be noted that the site soils (Layer 1), whether they are utilized for foundation support alone, or as engineered fill, will need to be recompacted through hand-tamping efforts, following the completion of the foundation excavation. This is necessary because of the inability of the site soils to maintain stability while withstanding the adverse effects of backhoe teeth. Hence the need for hand-tamping to regain soil bearing. Therefore, the bottom of the footing excavations must be hand-tamped to eliminate the probable adverse effects of the disturbance due to the backhoe. Prior to the placement of reinforcing steel, the base of all foundation excavations must be compacted with a "jumping jack" or plate

tamper, resulting in compaction of the foundation bearing soils to a depth of 6.0 inches. The final compaction must be to at least 95% of the ASTM D698 maximum density. Some degree of moisture processing may be required to facilitate proper compaction, although no moisture specification will apply. This condition does not apply to foundations bearing on Layer 2 rock.

[7]To avoid distress due to differential settlement, we recommend that all foundations bear on a like stratum, or strata that will produce similar settlements, and that all foundations use the same bearing capacity throughout the project.

[8]Any foundations traversing the pool backfilled area should be double-reinforced (top and bottom) and tied to the slab, wherever possible. The double reinforcement should extend 10.0 feet past the limits of the pool and basement backfill area. Refer to Section IV for the Swimming Pool Removal and Backfill Detail.

Special note: Foundations for free-end retaining walls may utilize allowable soil / rock bearing capacities that are double the above listed values, corresponding to 1" of allowable total settlement and 1/2" of allowable differential settlement.

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 live loads. The maximum allowable foundation bearing pressure for foundation toe pressures may be increased by ⅓ for resistance to short-term/temporary wind loads and or eccentric or lateral loading.

Retaining wall or building foundations to be constructed in close proximity to retention basins (within 5.0 feet) should be embedded 1.0 feet deeper than the stated depths in the preceding bearing capacity tables.
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 by the Vann Engineering Inc. project geotechnical engineer or their representative to ensure that they are free of loose soil which may have blown or sloughed into the excavations. It will also be necessary for the geotechnical engineer to verify that the footing embedment depths and bearing stratum adhere to the recommendations presented herein.

Foundation stepping will be required to prevent any transitional foundation from bearing on fill or retaining wall backfill soil. Specifically, this refers to a footing that will transition from the retaining wall level to the house level. At all times, footings installed throughout the step must bear on native undisturbed soil, as outlined in Surface to Retaining Wall Footing Transitions, Option A (Included in Section IV). If footings must bear on or in retaining wall backfill, the recommendations included in Surface to Retaining Wall Footing Transitions, Options B and C, must be followed. Note: retaining wall backfill is not considered engineered fill. Furthermore, the recommendations in Section IV are preliminary and must be reviewed and finalized by the project structural engineer.

All concrete must conform with the requirements established by the governing building code or agency.

5.7 Lateral Stability Analyses

All on-site retaining walls must be designed by the project structural engineer to resist the anticipated lateral earth pressures. Unrestrained (free-end) retaining walls should be designed by the project structural engineer for active earth pressures (K_a) and are assumed to allow small movement of the wall. Restrained (fixed-end) retaining walls should be designed by the project structural engineer 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 table presents recommendations for lateral stability analyses:

Table 11: Lateral Stability

Parameter	Wall Type	Native Undisturbed Soil (Layer 1)	Layer 2[3]
Active (K_a) Pressure[1]	Free-end retaining conditions	34 psf/ft	
At-Rest (K_o) Pressure[2]	Fixed-end retaining conditions	52 psf/ft	
Passive (K_p) Resistance	Free-end conditions, and Fixed-end conditions that are entirely independent of base friction	358 psf/ft	593 psf/ft
	Free-end conditions, and Fixed-end conditions in conjunction with base friction	240 psf/ft	398 psf/ft
Coefficient of Base Friction (μ)	Free-end conditions, and Fixed-end conditions that are entirely independent of passive resistance	0.62	0.81
	Free-end conditions, and Fixed-end conditions in conjunction with passive resistance	0.42	0.54

[1]Equivalent fluid pressures for vertical walls and horizontal backfill surfaces (*maximum 12.0 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.

[2]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 ½ the height*)

[3]The values listed are predicated on conformance to the recommended cut slope ratios provided herein. Non-conformance to the recommended cut slope ratios will result in significantly higher active stresses.

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

Fill against footings, stem walls, and any retaining walls should be compacted to the densities specified in Site Preparation. **High plasticity clay soils should not be used as backfill against basement and 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.

5.8 Conventional Slab Support

Site grading within the building areas should be accomplished as recommended herein. Four inches of aggregate base course (ABC) floor fill should immediately underlie interior grade floor slabs. The aggregate base material should conform to the requirements of local practice.

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 (by others) and installation should be in accordance with the recommendation given in ACI 302.1R. 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 systems that allow for the differential vertical movement described herein between the slabs and adjoining structural elements, i.e., ¼ inch.
5. 2-sack ABC/cement slurry should be utilized as backfill at the intersection of utility trenches with the building perimeter.

All concrete must conform with the requirements established by the governing building code or agency.

5.9 Drainage

The major cause of soil problems in this locality is moisture increase in soils below structures. Therefore, it is extremely important that positive drainage be provided during construction and maintained throughout the life of any proposed development. In no case should long-term ponding be allowed near structures. Infiltration of water into utility or foundation excavations must be prevented during construction. Planters or other surface features that could retain water adjacent to buildings should not be constructed. **In areas where sidewalks or paving do not immediately adjoin structures, protective slopes should be provided with an outfall of at least 5 percent for at least 10 feet from perimeter walls.**

Backfill against footings, exterior walls, retaining walls, and in utility or sprinkler line trenches should be well compacted and free of all construction debris to minimize the possibility of moisture infiltration through loose soil. Roof drainage systems, such as gutters or rain dispenser devices, are recommended all around the roofline. Rain runoff from roofs should be discharged at least 10.0 feet from any perimeter wall or column footing. If a roof drainage system is not installed, rainwater will drip over the eaves and fall next to the foundations resulting in sub-grade soil erosion, creating depressions in the soil mass, which may allow water to seep directly under the foundations and slabs.

5.10 Landscaping Considerations

The potential for unwanted foundation and slab movements can often be reduced or minimized by following certain landscape practices. The main goal for proper landscape design (by others) should be to minimize fluctuations in the moisture content of the soils surrounding the structure. In addition to maintaining positive drainage away from the structure, appropriate plant/tree selections and sprinkler/irrigation practices are extremely important to the long-term performance of the foundations and slabs. The conventional practice of planting near foundations is not recommended.

Flower, shrub, and tree distances should be maintained according to the following table. Note that for planting distances less than 5.0 and 10.0 feet for flowers/shrubs and trees respectively, the adjoining foundation embedment depths will need to increase as indicated in the following table:

Table 12: Foundation Alterations Due to Landscaping

Flowers and Shrub Planting Distance	Tree Planting Distance	Foundation Alterations Due to Landscaping
5 feet	10 feet	-
4 feet	9 feet	Increase footing embedment depth by 6.0 inches[1]
3 feet	8 feet	Increase footing embedment depth by 12.0 inches[1]
2 feet	7 feet	Increase footing embedment depth by 18.0 inches[1]

[1]The use of 2-sack ABC cement slurry may be implemented to provide the requisite embedment depth increase below a more conventional foundation detail.

In addition to the above recommendations, for flowers and shrubs installed within 5.0 feet of perimeter foundations, it is recommended that the landscape architect select plants with very low to low relative water use from the Arizona Department of Water Resources (ADWR) Low-Water-Use / Drought-Tolerant Plant List available at https://www.azwater.gov/conservation/landscaping. Limit the watering to the minimum needed to maintain the vegetation. For greater moisture control, water these areas by hand. For planters and general landscaping, we recommend the following:

- Planters should be sealed.
- Grades should slope away from the structures.
- Only shallow rooted landscaping material should be used.
- Watering should be kept to a minimum.

Some trees may have extensive shallow root system that may grow under and displace shallow foundations. In addition, tree roots draw moisture from the surrounding soils, which may exacerbate shrink/swell cycles of the surface soils. The amount of moisture drawn out of the soil will depend on the tree species, size, and location. If trees are planted well away from foundations in irrigated areas, the chances of foundation damage are greatly reduced. The sprinkler system should be checked for leakages once per month. Significant foundation movements can occur if the soils under the foundations are exposed to a source of free water.

In lieu of deepened footings, a root barrier system can be implemented on individual trees. In order to reduce the minimum distance of tree installation to 7.0 feet from the foundation of adjacent structures, UB 24-2 root barriers from DeepRoot Green Infrastructure, LLC (or equivalent) may be implemented in box formations, surrounding the protection sides of installed trees. A minimum depth of embedment of 23.5 inches of the DeepRoot UB 24-2 (or equivalent) root barriers, is required by this firm in order to redirect root growth downward and prevent moisture by landscape irrigation from entering the foundation zone of the adjacent structures. A minimum 0.5 inch of the root barrier must extend above the soil surface to prevent tree roots from growing over the top of the barrier. A minimum protection barrier around 3 sides of all installed trees must be utilized as a root barrier.

5.11 Foundations and Risks

The factors that aid in the design (by others) and construction of foundations include economics, risk, soil type, foundation shape and structural loading. 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 previous foundation recommendations are not permitted to resist soil movements as a result of sewer/plumbing leaks, excessive irrigation, poor drainage, and water ponding near the foundation system.

It is recommended that the owner 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 must 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 6.0-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 subexcavation depths and overexcavation 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. Backfilling and compaction of excavations (e.g., Utility trench backfill)
F. Special inspections as dictated by the local municipality
G. Concrete sampling and testing for footings, stem walls and floor slabs
H. Subgrade testing for proposed pavement areas
I. ABC testing for proposed pavement areas
J. Asphaltic concrete testing for proposed pavement areas
K. Subgrade preparation for on-site sidewalk areas
L. Grout sampling and testing, where applicable
M. Mortar sampling and testing, where applicable
N. 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.

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.

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 solely 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.
Over excavate	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 | PROJECT 25355
PROPOSED CUSTOM RESIDENCE
APN 172-47-086, STONE CANYON, LOT 29
5338 EAST SAN MIGUEL
PARADISE VALLEY, ARIZONA 85253

TEST BORING LOCATION (CONDUCTED IN 2017)

HAND-SAMPLE LOCATION (CONDUCTED IN 2017)

SEISMIC SURVEY LOCATION (CONDUCTED 2017)

SEISMIC SURVEY LOCATION (CONDUCTED IN 2024)

TB-1

Surface Elevation: See Site Plan

Boring Date: 08/09/17

Driller: M. Mewhinney

Drilling Method: CME-55

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table	USCS	Description
0						GWT not encountered	FL	Asphalt (2.5")
								SPREAD FILL (19") damp to moist, 20% gravel, 55% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 8, no cementation
	Grab sample obtained from 1.0 to 3.0 feet			26/30			SC-SM	GRAVELLY SILTY CLAYEY SAND, pinkish, damp, 35% gravel, 40% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, dense, PI of 5, weak to medium cementation (LAYER 1)
5				24/25/30				Pinkish to pink-brown, very dense, strong cementation below 2.5 feet
							AS	HIGHLY TO MODERATELY WEATHERED AND FRACTURED ARKOSIC SANDSTONE, pinkish (LAYER 2)
10				44/50/3"				Grayish below 10.0 feet
15								TEST BORING DISCONTINUED AT 15.0 FEET
20								
25								
30								
35								

LOG OF BORING

VANN ENGINEERING, INC.

Proposed Custom Residence

Project No. 25355

TB-2

Surface Elevation: See Site Plan
Boring Date: 08/09/17
Driller: M. Mewhinney
Drilling Method: CME-55

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table		
0						GWT not encountered	FL	Asphalt (2.5") SPREAD FILL (18") damp, 20% gravel, 55% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 8, no cementation
				8/ 17	■		SC-SM	GRAVELLY SILTY CLAYEY SAND, damp, 35% gravel, 40% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 5, weak to medium cementation (LAYER 1)
5							AS	HIGHLY TO MODERATELY WEATHERED AND FRACTURED ARKOSIC SANDSTONE, pinkish
10								AUGER REFUSAL AT 10.0 FEET IN HIGHLY TO MODERATELY WEATHERED AND FRACTURED ARKOSIC SANDSTONE (LAYER 2)
15								
20								
25								
30								
35								

LOG OF BORING

VANN ENGINEERING, INC.

Proposed Custom Residence
Project No. 25355

Plate 2

HS-1

Surface Elevation: See Site Plan
Boring Date: 08/09/17
Driller: M. Mewhinney
Drilling Method: CME-55

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table		Description
0						GWT not encountered	FL	SPREAD FILL (16") damp, 15% gravel, 60% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 8, no cementation
							AS	GRAVELLY SILTY CLAYEY SAND, pinkish-brown, damp, 35% gravel, 40% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, dense, PI of 5, weak cementation (LAYER 1)
								AUGER REFUSAL AT 2.0 FEET ON HIGHLY TO MODERATELY WEATHERED AND FRACTURED ARKOSIC SANDSTONE (LAYER 2)
5								
10								
15								
20								
25								
30								
35								

SuperLog CivilTech Software, USA www.civiltech.com File: \\vannsrv\Users\mark\Desktop\25355.log Date: 8/17/2017

LOG OF BORING

VANN ENGINEERING, INC.	**Proposed Custom Residence** **Project No. 25355**	Plate 3

HS-2

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
						GWT not encountered

Surface Elevation: See Site Plan
Boring Date: 08/09/17
Driller: M. Mewhinney
Drilling Method: CME-55

Depth: 0, 5, 10, 15, 20, 25, 30, 35

FL
SC-SM

SPREAD FILL (8") damp, 15% gravel, 60% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, medium dense, PI of 8, no cementation

GRAVELLY SILTY CLAYEY SAND, pinkish-brown, damp, 35% gravel, 40% sand, 25% fines, poorly graded, subangular to subrounded coarse-grained particles, dense, PI of 5, weak cementation (LAYER 1)

AUGER REFUSAL AT 1.0 FEET ON HIGHLY TO MODERATELY WEATHERED AND FRACTURED ARKOSIC SANDSTONE (LAYER 2)

LOG OF BORING

VANN ENGINEERING, INC.	Proposed Custom Residence Project No. 25355	

VELOCITY CLASSIFICATION DATA

Proposed Custom Residence
APN 172-47-086, Stone Canyon, Lot 29
5338 East San Miguel
Paradise Valley, Arizona 85253

Average Velocity of Layer 1:	1287 fps	(1019 to 1693)
Average Velocity of Layer 2:	4734 fps	(4124 to 5294)
Average Depth to Layer 2:	3.1 feet	
Range:	1.0 to 6.0 feet	

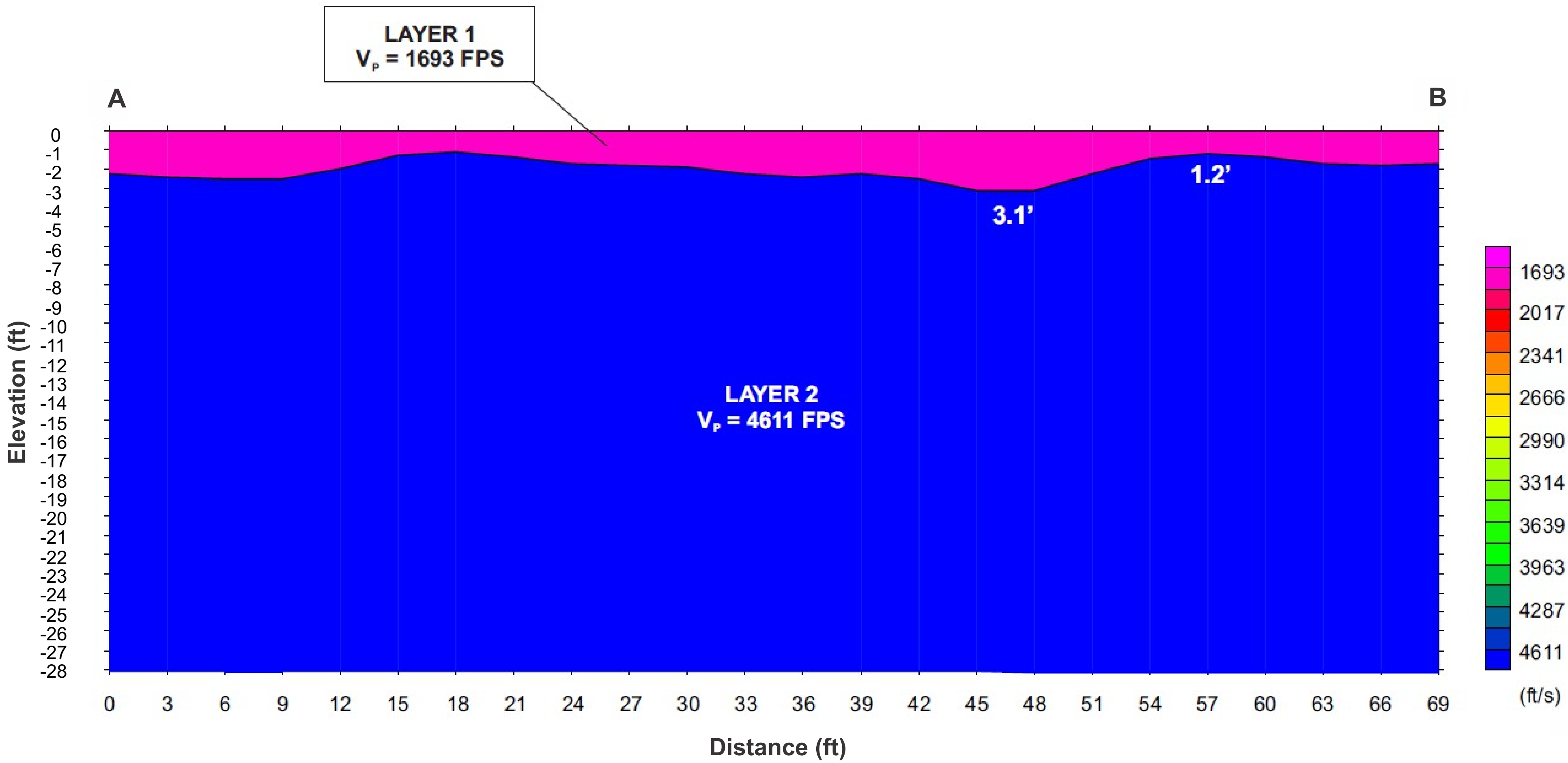
Layer 1: Moderately dense alluvium and spread fill comprised of gravelly silty sand and gravelly sand, with fines (SC-SM)

Layer 2: Highly to moderately weathered and fractured, poor, weak arkosic sandstone

Line	Layer 1			Layer 2		
	Velocity	Depth (ft)		Velocity	Depth (ft)	
A - B	1693	-	-	4611	3.1	1.2
C - D	1304	-	-	4124	1.3	2.3
E - F	1197	-	-	4888	2.0	4.8
G - H	1019	-	-	4753	2.3	4.2
I - J	1224	-	-	5294	3.1	6.0
TB-1	-	-	-	-	-	5.0
TB-2	-	-	-	-	-	5.0
HS-1	-	-	-	-	-	2.0
HS-2	-	-	-	-	-	1.0
Averages	1287	-		4734	3.1	

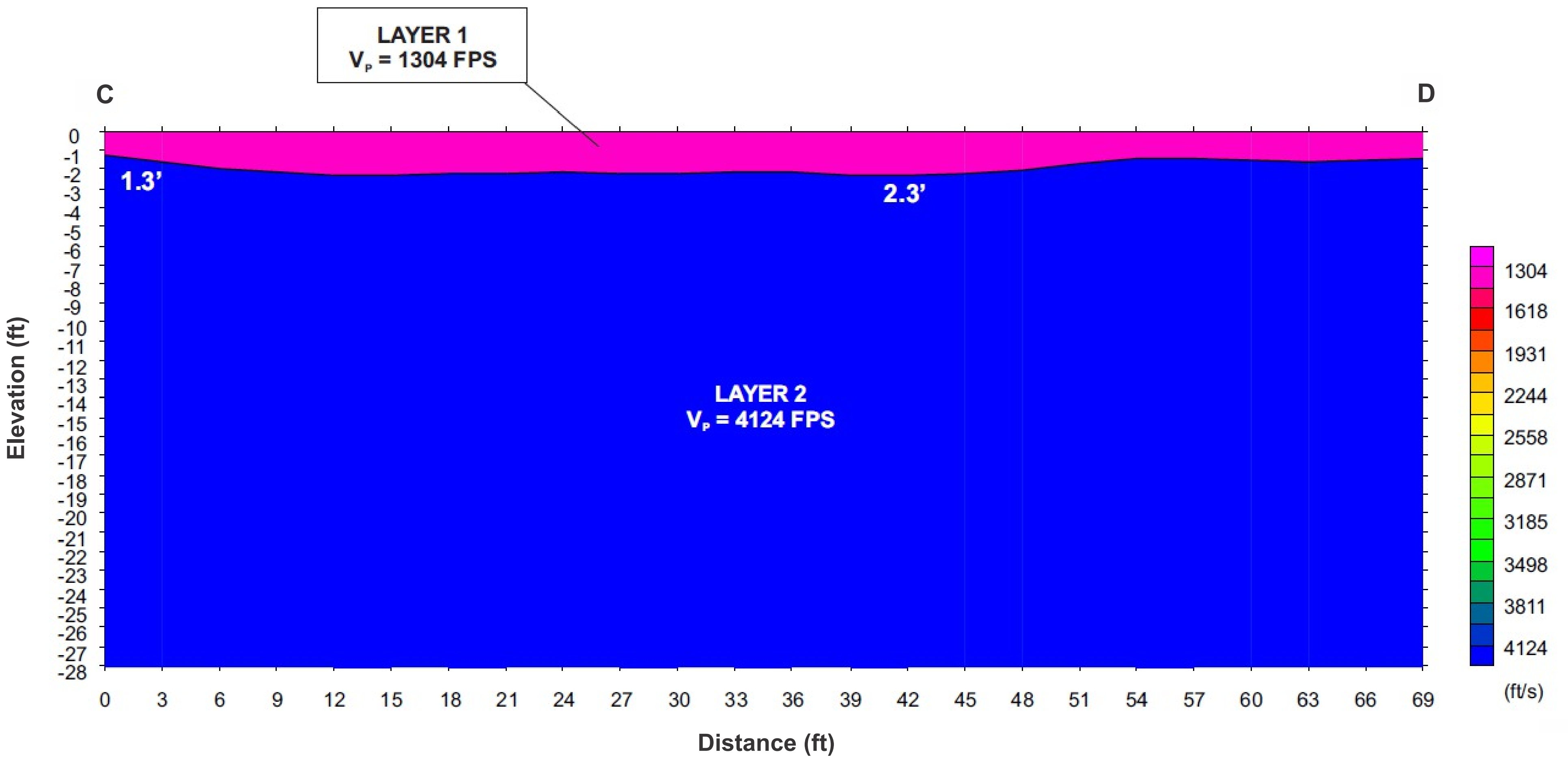
Cross Section

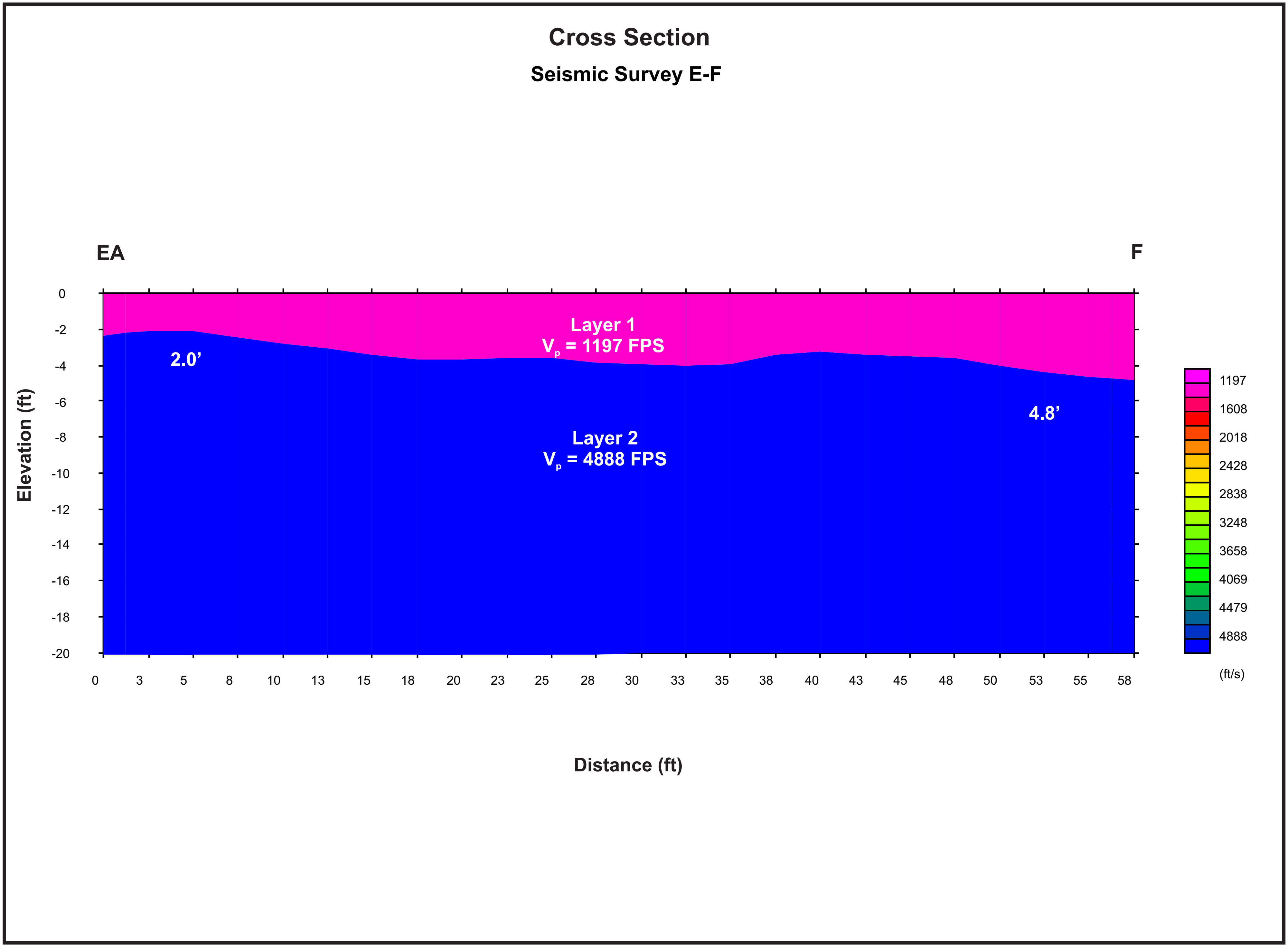
Seismic Survey A-B

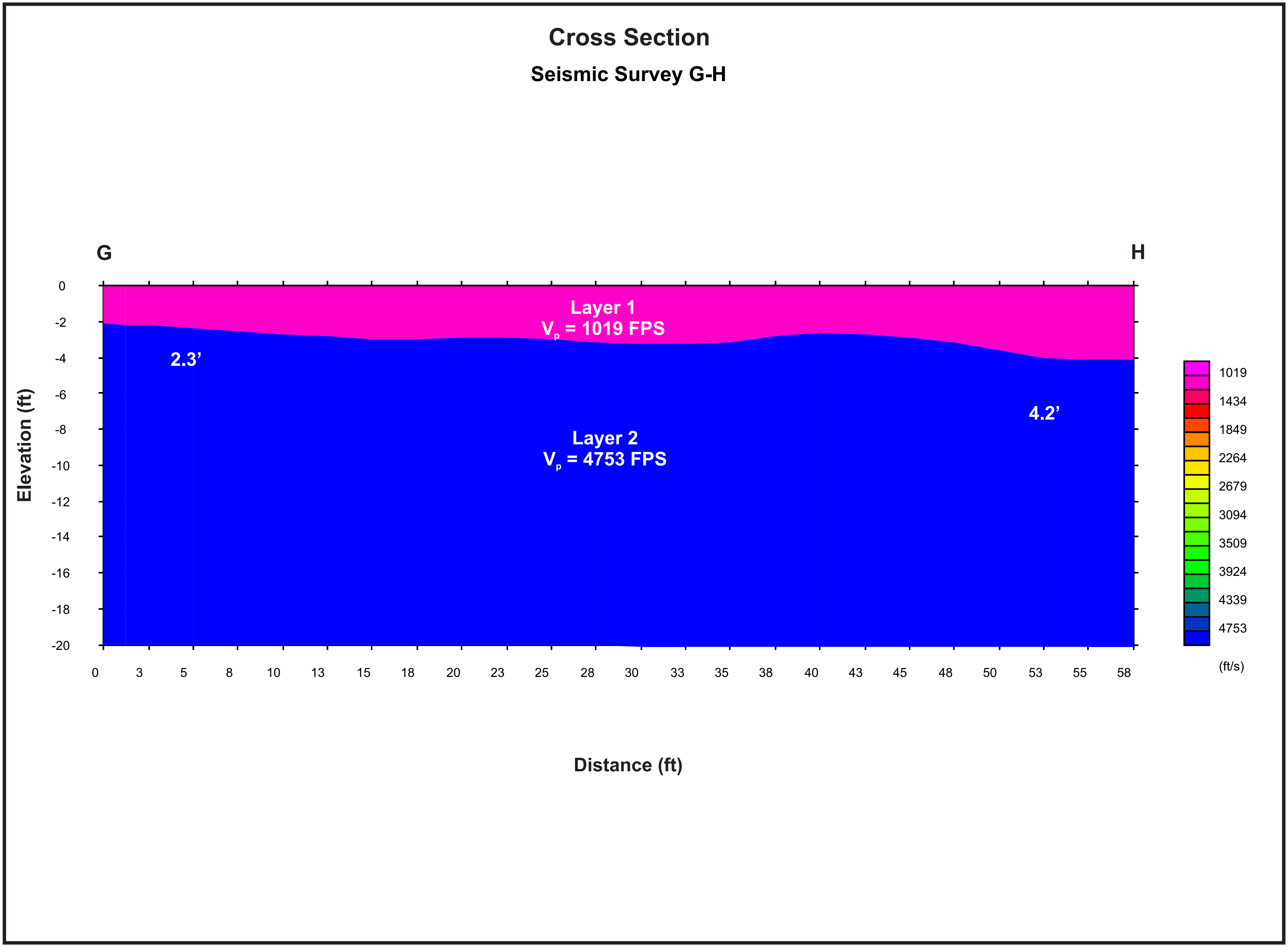


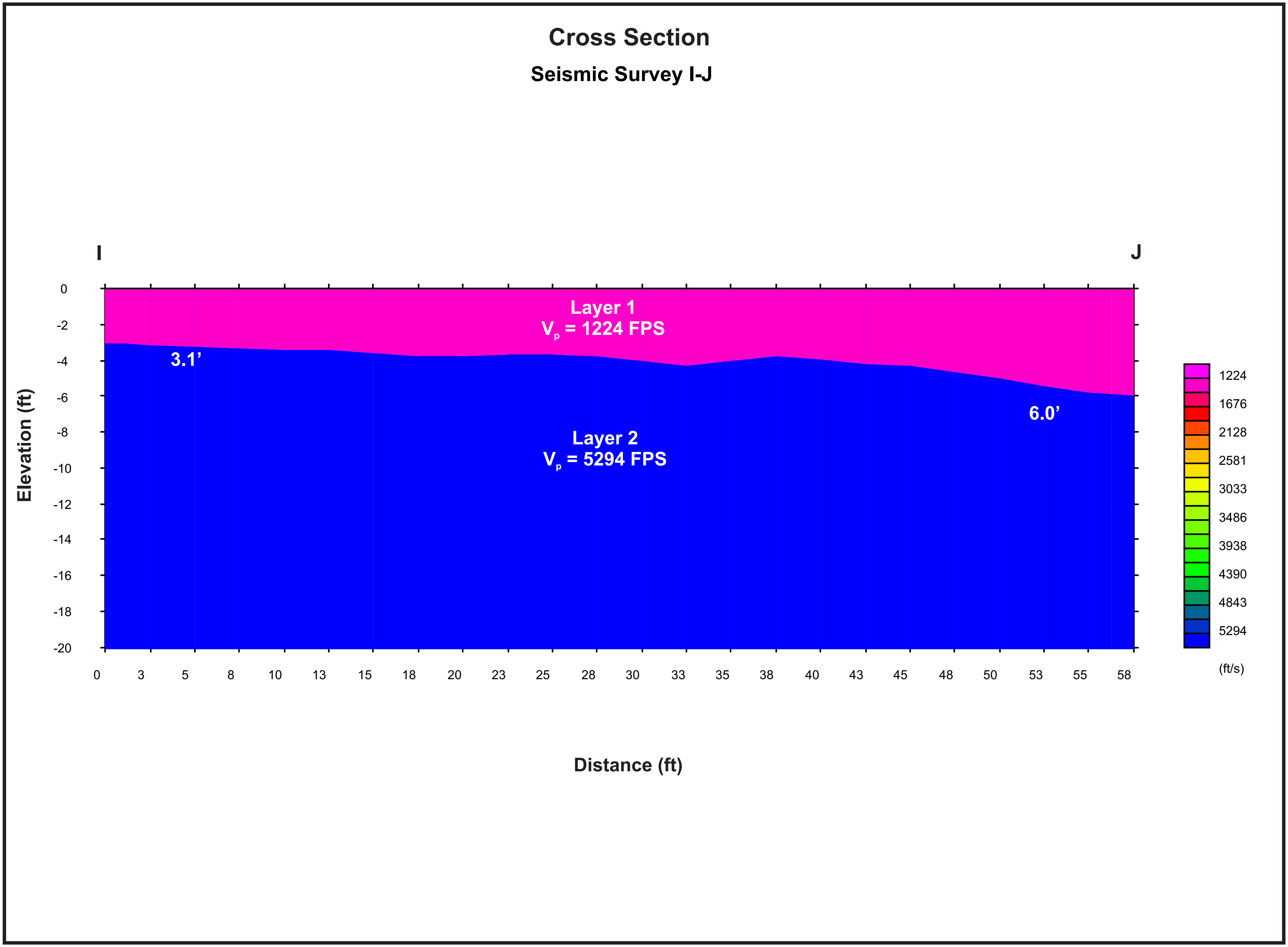
Cross Section

Seismic Survey C-D




Cross Section
Seismic Survey E-F
EA
F
Layer 1
Vp = 1197 FPS
2.0'
Layer 2
Vp = 4888 FPS
4.8'
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
3
5
8
10
13
15
18
20
23
25
28
30
33
35
38
40
43
45
48
50
53
55
58
Distance (ft)
1197
1608
2018
2428
2838
3248
3658
4069
4479
4888
(ft/s)

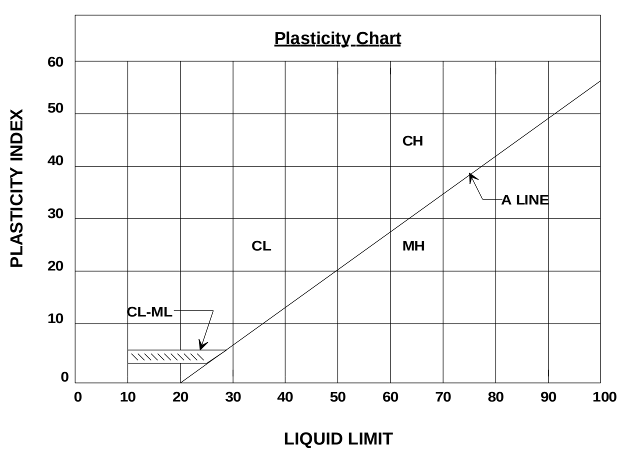

Cross Section
Seismic Survey G-H
G
H
Layer 1
V_p = 1019 FPS
2.3’
4.2’
Layer 2
V_p = 4753 FPS
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
3
5
8
10
13
15
18
20
23
25
28
30
33
35
38
40
43
45
48
50
53
55
58
Distance (ft)
1019
1434
1849
2264
2679
3094
3509
3924
4339
4753
(ft/s)


Cross Section
Seismic Survey I-J
I
J
Layer 1
Vp = 1224 FPS
3.1'
6.0'
Layer 2
Vp = 5294 FPS
Elevation (ft)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
3
5
8
10
13
15
18
20
23
25
28
30
33
35
38
40
43
45
48
50
53
55
58
Distance (ft)
1224
1676
2128
2581
3033
3486
3938
4390
4843
5294
(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.

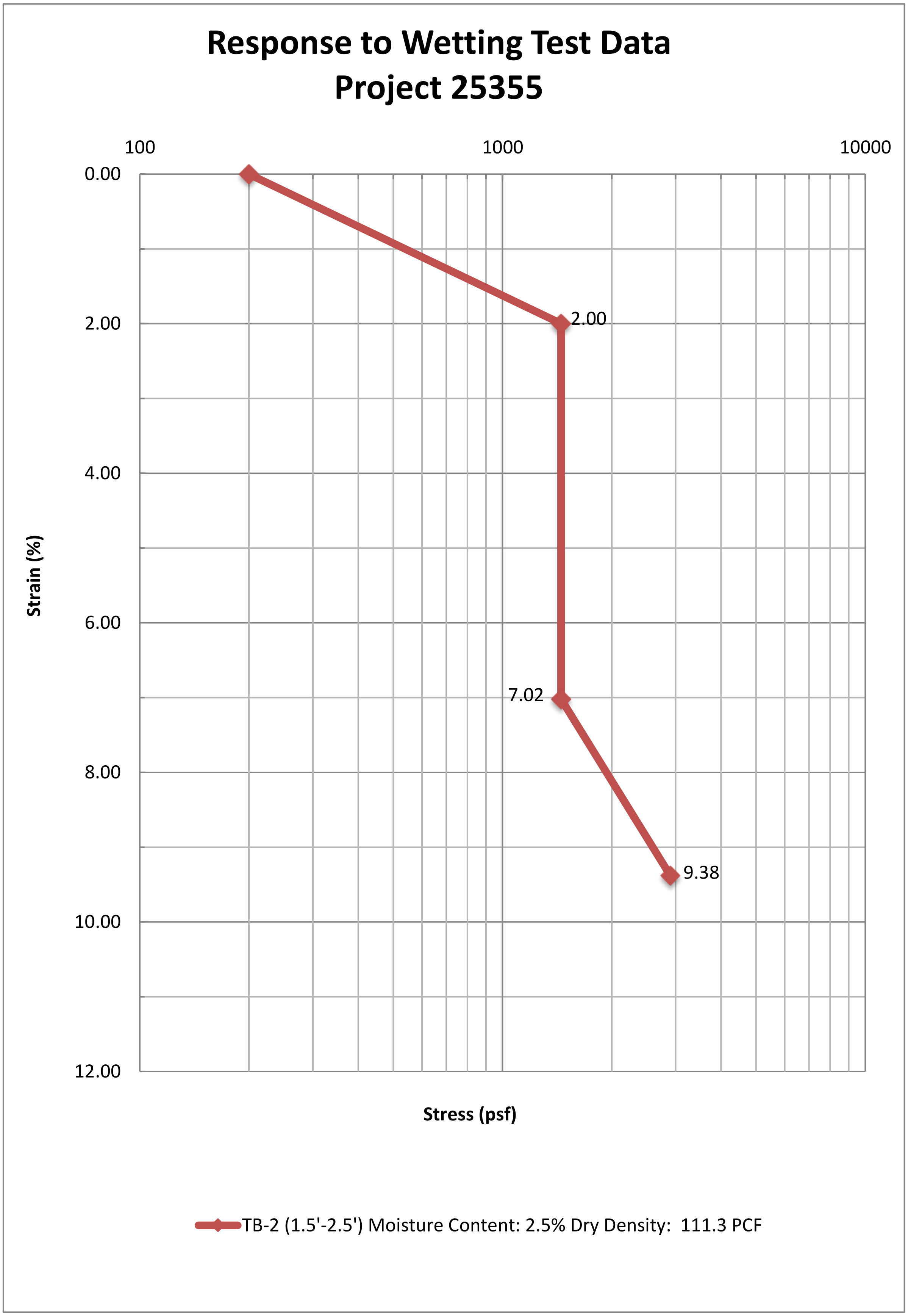
All geologic materials exhibit P-Wave velocities in certain ranges, which relate to the density, specific gravity, elastic modulus, and moisture content of the specific material. As a material density and specific gravity increase so does its P-Wave velocity. Similarly, an increase in moisture content 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 25355
100
1000
10000
0.00
2.00
4.00
6.00
8.00
10.00
12.00
Strain (%)
2.00
7.02
9.38
Stress (psf)
TB-2 (1.5'-2.5') Moisture Content: 2.5% Dry Density: 111.3 PCF

CLASSIFICATION TEST DATA

PROPOSED CUSTOM RESIDENCE
APN 172-47-086, STONE CANYON, LOT 29
5338 EAST SAN MIGUEL
PARADISE VALLEY, ARIZONA 85253

Sample Location	Sieve Analysis (% Passing Sieve Size) 3"	2"	1"	#4	#10	#40	#100	#200	Atterberg Limits LL	PI	USCS	Moisture Content %
SG-A (0.0'-2.0')	-	100	94	65	53	39	-	26	22	5	SC-SM	2.4
SG-B (0.5'-1.5')	-	100	98	62	49	28	19	15	25	7	SC-SM	1.8

SULFATES AND CHLORIDES TEST RESULTS

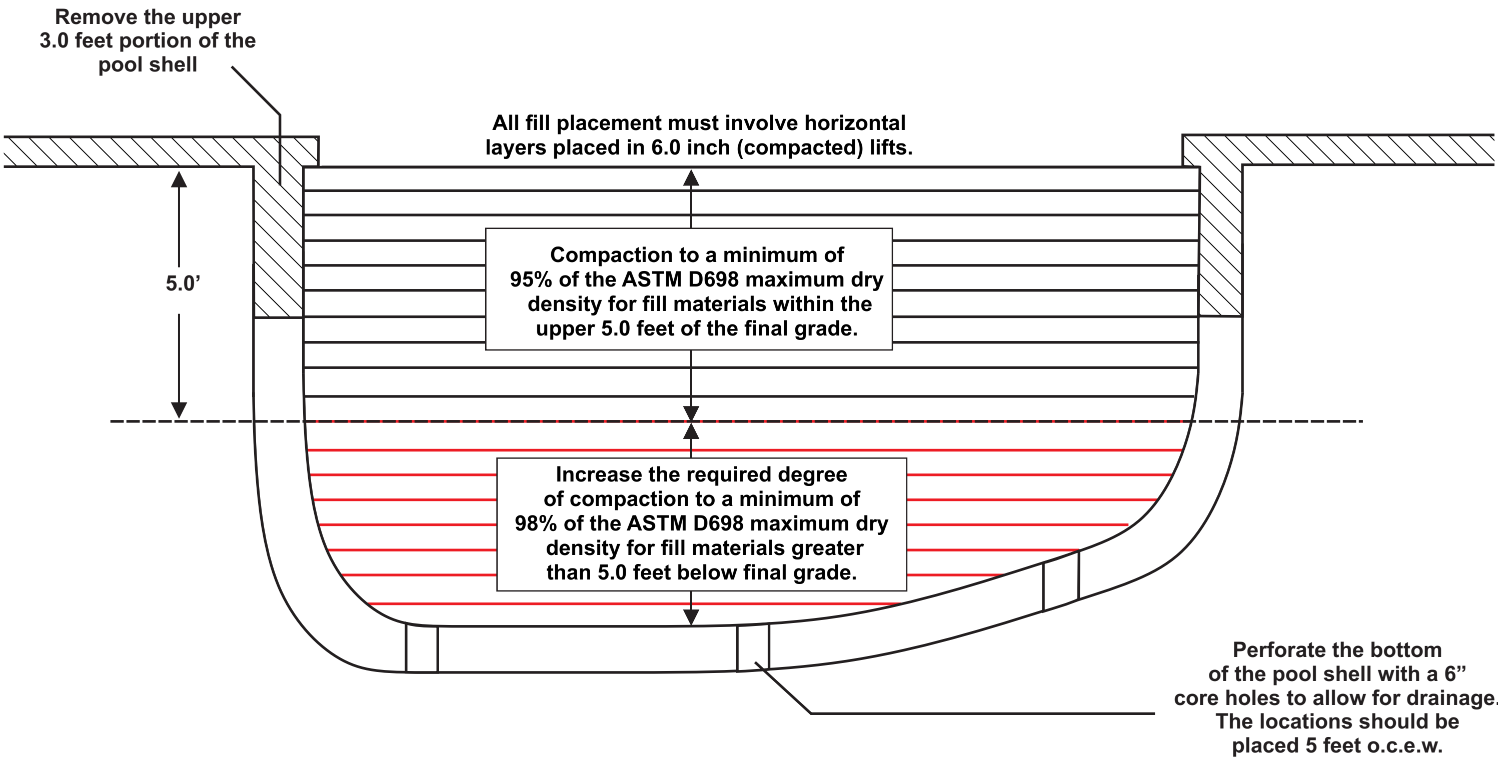
PROPOSED CUSTOM RESIDENCE
APN 172-47-086
5338 EAST SAN MIGUEL AVENUE
PARADISE VALLEY, ARIZONA 85253

Sample Location	***Test Interval (feet)***	***Sulfate (%)***	***Chloride (ppm)***
SG-B	0.5-1.5	0.089	10

SECTION IV

SWIMMING POOL REMOVAL AND BACKFILL DETAILS

POOL SHELL STILL IN-PLACE



POOL SHELL HAS BEEN REMOVED