VANN ENGINEERING INC

GEOTECHNICAL INVESTIGATION REPORT

Proposed Humphrey - Borgogni Residence
APN 408-19-009
Lot 10 – Cup of Gold Estates
90 Cayuse Trail
Sedona, Arizona 86336

Prepared for:

Bryan Humphrey and Rima Borgogni
AZ Best Plumbing
221 Canon Wren Drive
Sedona, Arizona 86366-5135

March 4, 2021

Project 28207

1980 – 2021
41
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Vann Engineering Inc.

GEOTECHNICAL ENGINEERING ▪ ENVIRONMENTAL CONSULTING
CONSTRUCTION TESTING & OBSERVATION

March 4, 2021 Project 28207

Bryan Humphrey and Rima Borgogni
AZ Best Plumbing
221 Canon Wren Drive
Sedona, Arizona 86366-5135

RE: Geotechnical Investigation Report
Proposed Humphrey - Borgogni Residence
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Bryan and Rima,

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. The materials encountered on the site are believed to be representative of the total area; however, soil and rock materials do vary in character between points of investigation. The recommendations contained in this report assume that the soil conditions do not deviate appreciably from those disclosed by the investigation. Should unusual material or conditions be encountered during construction, the soil engineer must be notified so that they may make supplemental recommendations if they should be required. We trust that this report will assist you in the design and construction of the proposed project. 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 when a proposal for these services is desired. 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. 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 3/4/2021
ARIZONA, U.S.A.

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SECTION I

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1.0 INTRODUCTION

Vann Engineering, Inc. understands that a new custom residence is proposed for construction at the above-mentioned site, with a planned basement-level. This document presents the results of a geotechnical investigation conducted by Vann Engineering, Inc. for the:

Proposed Humphrey - Borgogni Residence
APN 408-19-009
Lot 10 – Cup of Gold Estates
90 Cayuse Trail
Sedona, Arizona 86336

The following aerial photograph shows the site (outlined in red) and immediate vicinity.

Figure 1: Aerial photograph of the site (outlined in red) and 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 and rock underlying the site, and 2) to provide final recommendations for safe and economical foundation design and slab support.

For purposes of foundation design, the maximum column and wall loads have been assumed to be as summarized in the following table.

Table 1: Anticipated Design Loads

Foundation Type	Maximum Column Load (KIPS)	Maximum Wall Load (KLF)
Conventional, shallow, lightly loaded surface-level and basement-level spread foundations 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 site
- Description of the major soil layers
- Site Plan indicating the locations of all points of exploration
- Explanation of applicable geologic hazards
- Recommendations for surface-level and basement-level spread foundations; allowable bearing capacity based on settlement analysis of ½ inch total settlement and ¼ inch differential settlement (design data, 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 safe cut slopes
- Recommendations for site grading - necessary earthwork for conventional systems
- Recommendations for drainage and slab support
- Anticipated shrinkage of the surface soil
- 2018 IBC Seismic Site Classification

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

1.3 Authorization

The obtaining of data from the site and the preparation of this geotechnical investigation report have been carried out according to this firm's proposal **(VE20GT1231KM3 dated December 31, 2020)**, authorized by **Bryan Humphrey on January 29, 2021** 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 a planned basement-level.

2.2 Site Description

The site is comprised of undulating hillside terrain that slopes down to the east. The site is moderately vegetated with trees, scrub brush, and cacti. Numerous ledgy rock outcrops comprised of the Schnebly Hill Formation (part of the Supai Group) which includes siltstone, sandstone, claystone, mudstone were observed scattered across the site (defined herein as Layers 2 and 3). One natural drainage wash was observed traversing the southwestern portion of the site. Note: Significant over-sized aggregate (particles greater than 3.0 inches) was observed scattered across the surface of the site. These oversized particles must not be used as structural fill. The following images depict the current site conditions:

Figure 2: General site conditions showing over-sized aggregate (particles greater than 3.0 inches)

Figure 3: General site conditions showing over-sized aggregate (particles greater than 3.0 inches) and rock outcrops

Figure 4: General site condtions showing ledgy rock outcrops and over-sized aggregate (particles greater than 3.0 inches)

Figure 5: General site condtions the wash that traversaes the southwestern portion of the site

Figure 6: General site condtions the wash that traverses the southwestern portion of the site

Figure 7: General site condtions the wash that traverses the southwestern portion of the site

3.0 SUBSURFACE INVESTIGATION AND LABORATORY TESTING

3.1 Subsurface Investigation

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 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. Note: Changes in the calculated velocity indicate strata breaks or distinct changes within the same stratum. The important concept to remember with this method is that it is predominantly effective where velocities increase from layer to layer, moving downward from the surface.

Analytical methods are used by this firm for determining the depth to the various layers, even in the most complex multi-layer situations. However, when a denser, and hard soil or rock layer overlies a weaker or less dense soil or rock layer, the weaker or less dense layer is masked and not detected by the seismograph. If a weaker layer is encountered during the excavation efforts, this office should be contacted immediately for further recommendations.

Generally, the depth of a seismic survey investigation is approximately equal to one-third the length of the survey. 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. Subsurface soil/rock velocities are calculated by using a computer program (SeisImager 2D). 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 depth was 28.0 feet below the existing grade, regardless of the length of the survey lines.

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
Expansion	Remolded native soils (2)	Potential for heave upon wetting
Sieve Analysis, Atterberg Limits, and Moisture Content	Native subgrade soils (1)	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 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 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, 2]	Velocity Range (FPS)	Classification
1	Layer 1 occurs to depths ranging from 1.9 to 4.5 feet below the existing site surface	635 to 947	Loose coarse-grained alluvium / residual soil comprised of silty sandy clayey gravel (GC)[3]
2	Layer 2 occurs below depths ranging from 1.9 to 4.5 feet and above highly variable depths ranging from 7.3 to 21.4 feet from the existing site grade	3175 to 4927	Highly to moderately weathered and fractured, poor, weak Schnebly Hill Formation (part of the Supai Group) which includes siltstone, sandstone, claystone, mudstone
3	Layer 3 occurs below highly variable depths ranging from 7.3 to 21.4 feet from the existing site grade	9531 to 13695	Slightly weathered and fractured, excellent, extremely strong Schnebly Hill Formation (part of the Supai Group) which includes siltstone, sandstone, claystone, mudstone

[1]Calculated depth ranges 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 you approach visible outcrops, the depth to layer 2 approaches zero.

[3]Over-sized aggregate (particles greater than 3.0 inches) will occur within Layer 1 and should be anticipated during the excavation process.

Refer to the following tomographic cross sections and general layered cross sections located in Section II of this report for the subsurface layering determined by analysis of the seismic refraction survey data. The locations of the seismic surveys are depicted on the Site Plan in Section II.

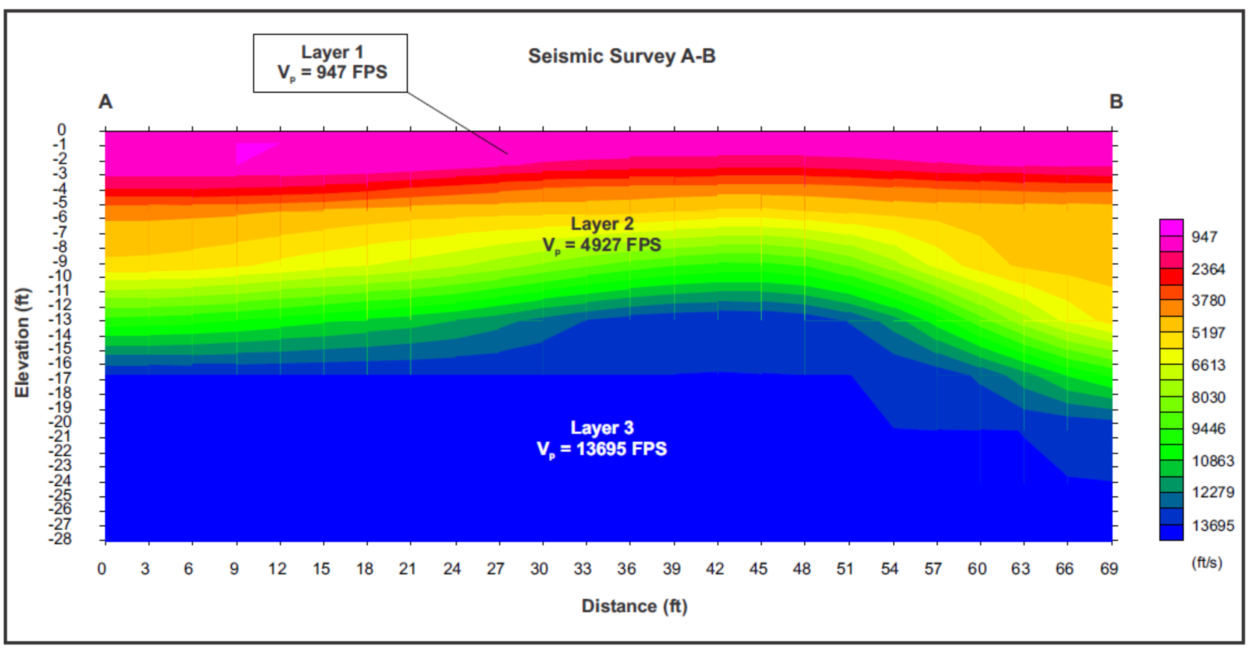


Figure 8: Tomographic cross section of Seismic Survey Line A-B

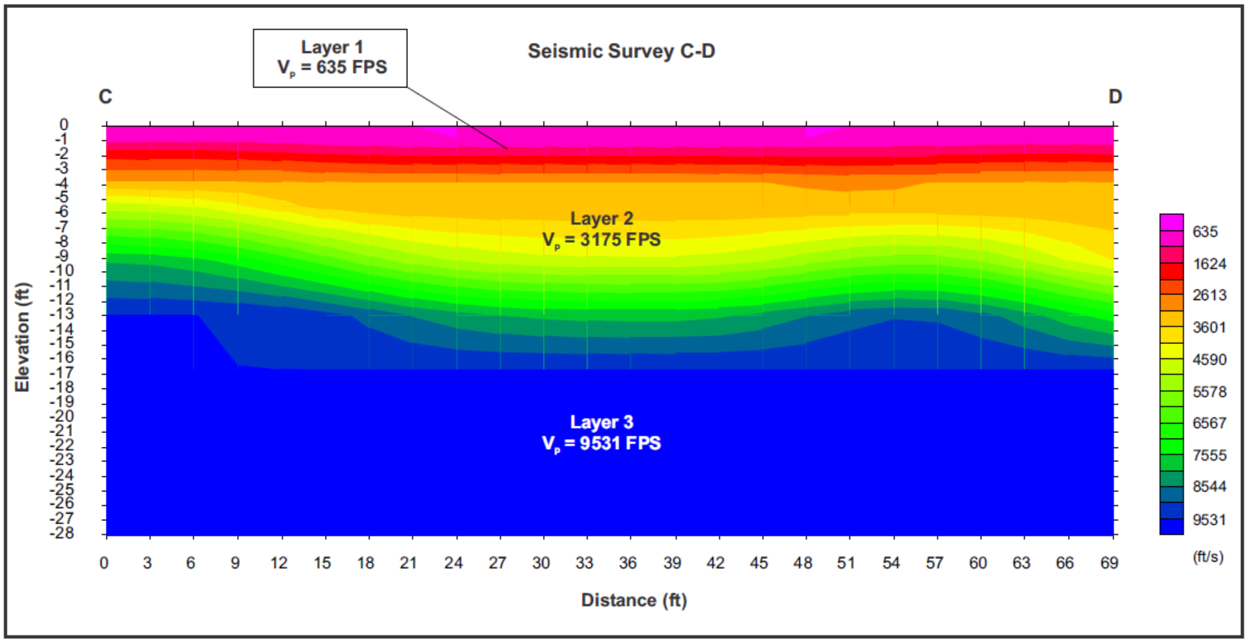


Figure 9: Tomographic cross section of Seismic Survey Line C-D

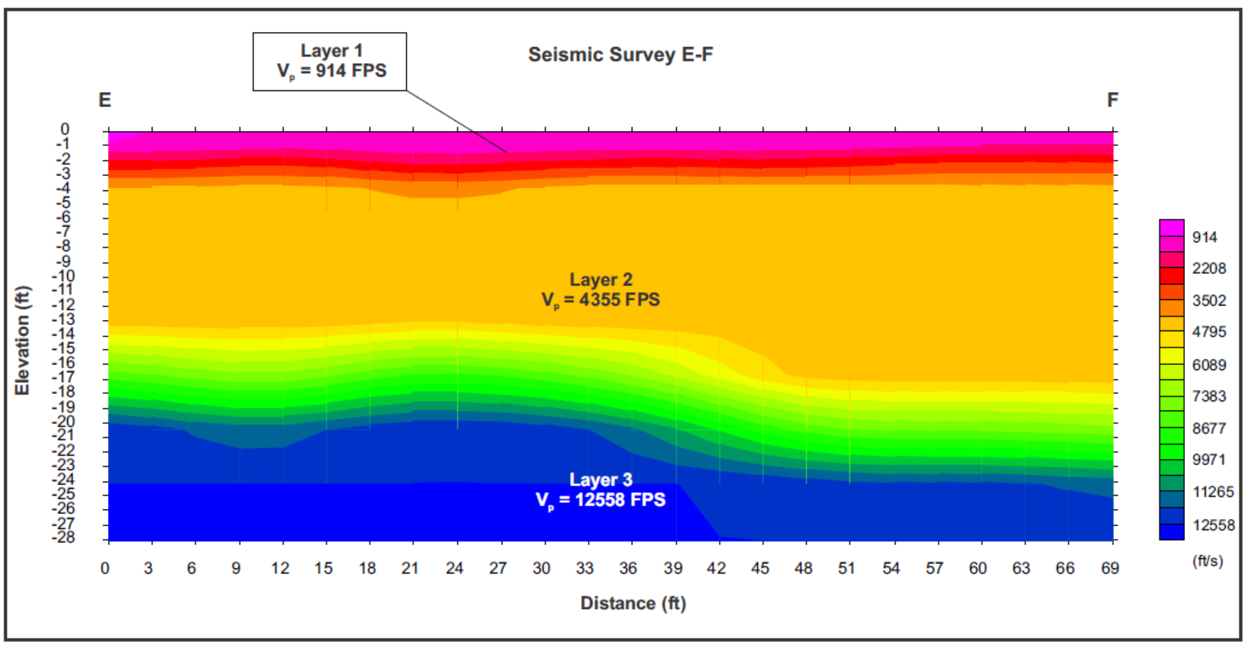


Figure 10: Tomographic cross section of Seismic Survey Line E-F

The relative position of Sedona and the Schnebly Hill Formation (part of the Supai Group) within the geologic staircase (stratigraphy) and the associated with the Mogollon Rim is shown in the figure below:

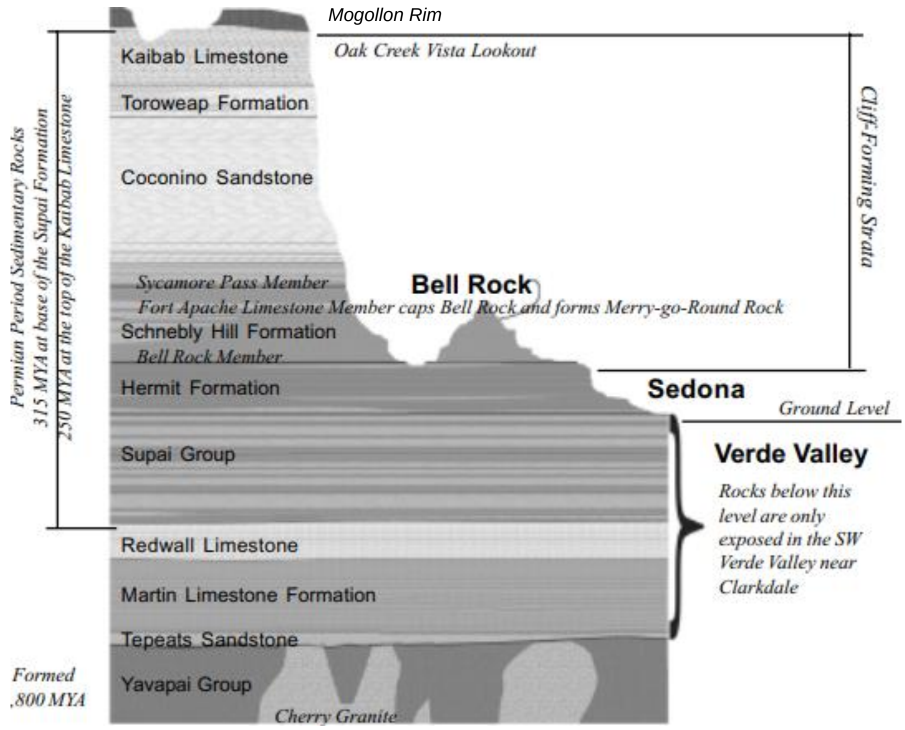


Figure 11: Idealized vertical stratigraphic section of the geologic formations in the Sedona area

4.2 Engineering Properties of the Site Soils

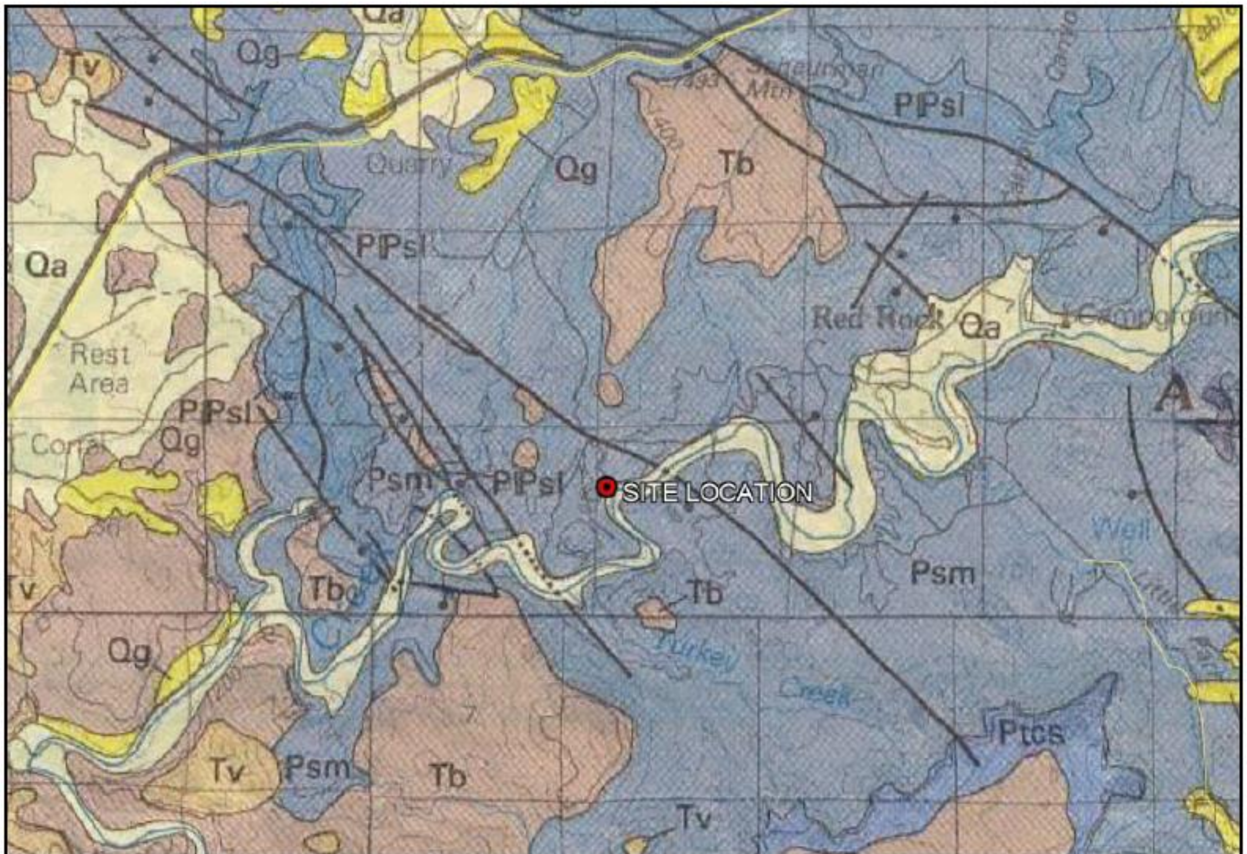
Expansive soils are soils that expand or swell and are typically known to have a shrink/swell potential. Cohesive soils, or clay soils, tend to shrink as they are dried, and swell as they become wetted. The clay content of the soil determines the extent of the shrink/swell potential. The surface soils encountered at the site are considered slightly cohesive based on the laboratory testing (i.e., measured plasticity index of 12). 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 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).

4.3 Local Geology

The local geology and our field investigation indicate that a thin layer of overburden soil (defined herein as Layer 1) overlies a rock mass comprised of the Schnebly Hill Formation (part of the

Supai Group) which includes siltstone, sandstone, claystone, mudstone (defined herein as Layers 2 and 3). Refer to the following Geologic Map which shows the geologic units, the site-specific composition and the proximity to the site.



Explanation:

Qa	Alluvium (Holocene)	Tb	Volcanic Rocks (Pliocene? and Miocene) Basaltic Volcanic Rocks Undiffentiated
Ql	Landslide Deposits (Holocene and Pleistocene)	Psm	Sedimentary Rocks - Middle part (Lower Permian) - Siltstone, sandstone, conglomerate, and minor dolomitic limestone[1]
Qg	Gravel (Holocene and Pleistocene)	PPsl	Sedimentary Rocks - Lower part (Lower Permian and Upper Pennsylvanian) - Mudstone, siltstone, sandstone, conglomerate, limestone, and minor dolomite[1]

[1]The Permian to Pennsylvanian sedimentary rocks (Psm and PPsl) are comprised of interbedded sandstone, siltstone, mudstone, claystone, shale, limestone, and dolomite, and are typically characterized by ledgy outcrops. Orange to reddish sandstone forms cliffs near Sedona. The unit includes the Supai Group and Hermit Shale. The Schnebly Hill Formation is part of the Supai Group which comprises the majority of the site. The Schnebly Hill Formation includes four members in the Sedona area and consist of, from youngest to oldest: The Sycamore Pass Member, Ft. Apache Member, Bell Rock Member, and the Rancho Rojo Member.

Figure 12: Geologic map of site and surrounding area

Geologic Map courtesy of Department of the Interior U.S. Geological Survey Miscellaneous Investigation Series Map I-1896, Geologic Map of the Sedona 30' x 60' Quadrangle, Yavapai and Coconino Counties, Arizona, by Gordon W. Weir, George E. Ulrich, and L. David Nealy (1989).

4.4 Groundwater

No groundwater was encountered during the course of this firm's site investigation. Groundwater is expected at a depth of approximately 221.5 feet according to recent well data in the area (GWSI Site ID: 344850111494801).

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:

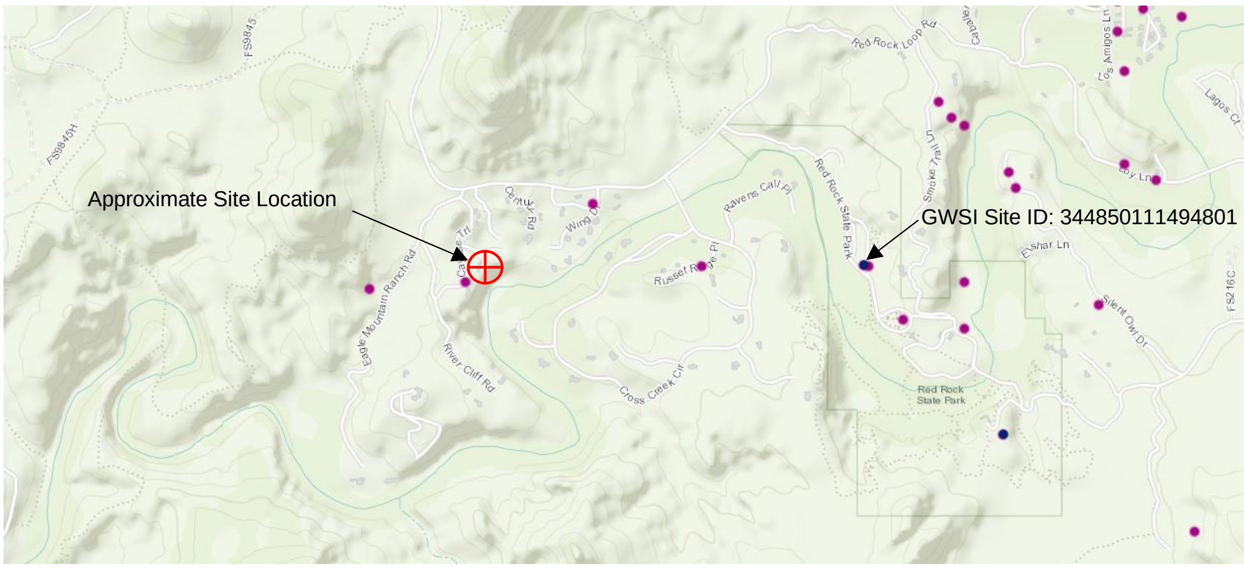


Figure 13: Groundwater Map

4.5 Frost Depth

The maximum depth of frost penetration is 1.25 feet indicating a minimum surface-level code foundation embedment depth of 2.0 feet for foundations on soil (site elevation approximately 3920 feet).

5.0 <u>RECOMMENDATIONS</u>

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. included in their "Handbook of Ripping."

Table 4: Excavating Conditions

Layer	Depth of Occurrence[1]	Seismic Wave Velocity (feet per second)	Remarks Relative to Rippability
1	Layer 1 occurs to depths ranging from 1.9 to 4.5 feet below the existing site surface	635 to 947	Conventional-Case 580 Trencher (special note given to over-size aggregate)[2]
2 (Hard Dig)	Layer 2 occurs below depths ranging from 1.9 to 4.5 feet and above highly variable depths ranging from 7.3 to 21.4 feet from the existing site grade	3175 to 4927	D10T, Caterpillar 235 (appropriately sized hoe ram attachment for backhoes)
3 (Very Hard Dig)	Layer 3 occurs below highly variable depths ranging from 7.3 to 21.4 feet from the existing site grade	9531 to 13695	Blasting techniques may be required to accomplish effective material removal[3]

[1]Calculated depth ranges 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]It must be noted that over-sized aggregate (particles greater than 3 inches) will occur within Layer 1 and should be anticipated during the excavation process. Over-sized particles must not be used as structural fill.

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

The subsurface soils (Layer 1) will be 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 B soil. Temporary excavations into Layer 1 soils are to be configured at a 1H:1V incline. Temporary excavations into Layer 2 are to be configured at a 1H:1.5V incline. Temporary excavations into Layer 3 are to be configured at a 1H:4.5V 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. Vann Engineering, Inc. does not assume any responsibility for construction site safety or the activities of the contractor.

5.2 Cut Slope Stability

The following tabulation presents this firm's analysis of safe cut slopes for the anticipated subsurface conditions. **However, it should be noted that the subsurface rock (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.

Table 5: Cut Slope Recommendations Not Exceeding 20 Feet in Height (Including Trenches)

Portion of Cut Slope	Temporary Cut Slope Ratio (Horizontal to Vertical)[a]	Permanent Cut Slope Ratio (Horizontal to Vertical)[a]
Layer 1	1:1	2.5:1
Layer 2	1:1.5	1:1
Layer 3	1:4.5	1:2.5

[a]This firm should be notified during construction in order to verify field conditions and inspect all cut slopes for structural features/discontinuities (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.

Because of the benched nature of the sedimentary rock at the site, the following rough cross section has been generated to illustrate the recommended benching pattern for all cut slopes. The general slopes presented above must be followed using a line through all respective slope toes.

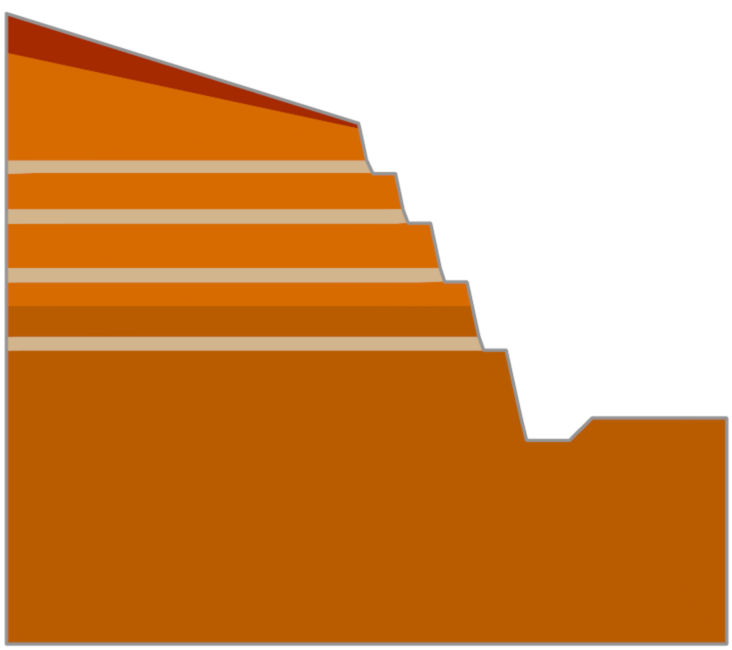


Figure 14: Cut Slope Illustration (20' Maximum Height)

20.0 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.

Items not included in this report are:

- Rock bolting/soil nailing to achieve a stable 1:5 (horizontal to vertical) cut slope.
- Slope protection in connection with the above in terms of Tecco mesh and shotcrete.
- Combination of rock bolts and a series of retaining walls.

Addressing the above items will encompass extensive additional effort and can be addressed in a subsequent analysis, if requested.

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, and pedestrian traffic should be directed away from, 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 6: Buffer Zones

Vertical Rock Cut-Slope Height (feet)	Horizontal Rock-Fall Impact Zone Distance (feet)
5	2.5
10	5

Vertical Rock Cut-Slope Height (feet)	Horizontal Rock-Fall Impact Zone Distance (feet)
15	7.5
20	10

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.

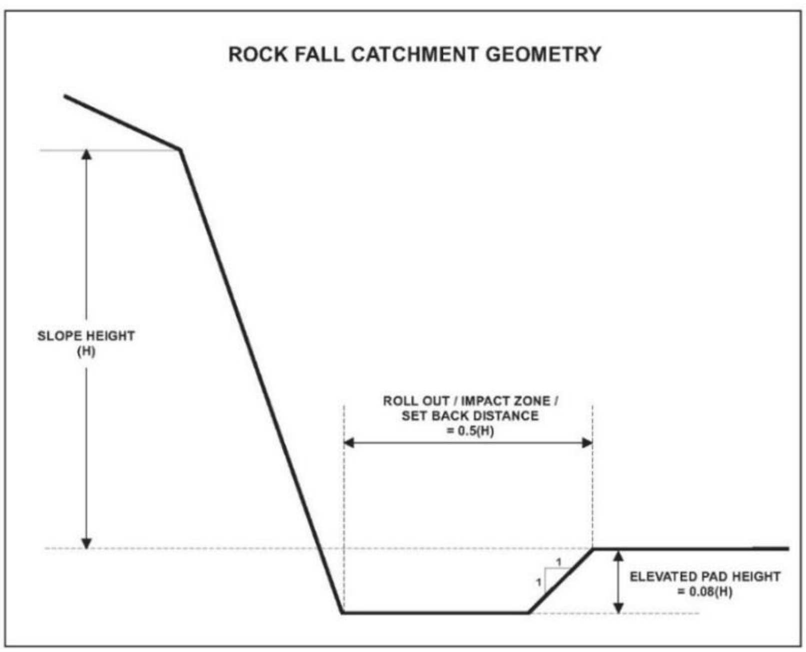


Figure 15: Rock Fall Catchment Geometry

5.3 Backfill Settlement

Retaining/basement 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/basement walls, or over utilities. Excessive settlement of loose backfills has caused damage to pavements, floor slabs, pedestrian walkways, planters, etc., which adjoin backfilled retaining/basement walls.

Deep compacted backfills will also tend to settle differently relative to retaining/basement 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/backfill 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.

Even with proper backfill compaction (well compacted – 95 percent minimum), the backfill will have the potential for about 1.2 inches of settlement (for 10.0 feet of total backfill) in the event of wetting by irrigation or broken conduits. With moderately compacted backfill (90 percent minimum), the magnitude of backfill settlement may approach 3.0 inches (for 10.0 feet of total backfill). Further, with poorly compacted backfill (85 percent minimum), the approximate magnitude of backfill settlement may reach as much as 6.0 inches (for 10.0 feet of total backfill). The preceding estimates for backfill settlement are those which may occur through settlement of the backfill alone, without any surcharge or other structural loading condition. Refer to the following table which reflects the anticipated settlement without any structural loads.

Table 7: Backfill Settlement

Backfill Types			Anticipated Settlement without any Structural Loads (in.)					
% Compaction	Description	% Estimated Strain	2.5 feet of backfill	5.0 feet of backfill	7.5 feet of backfill	10.0 feet of backfill	12.5 feet of backfill	15.0 feet of backfill
95-98	Very Well Compacted	**0.5**	0.15	0.3	0.45	0.6	0.75	0.9
95	Well Compacted	**1**	0.3	0.6	0.9	1.2	1.5	1.8
90	Moderately Compacted	**2.5**	0.75	1.5	2.25	3.0	3.75	4.5
85	Poorly Compacted	**5**	1.5	3.0	4.5	6.0	7.5	9.0
80	Very Poorly Compacted	**7.5**	2.25	4.5	6.75	9.0	11.25	13.5

Accordingly, it is recommended that where slabs are supported on grade over fill but are also tied to or connected to elements supported at basement 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.

Foundation stepping will be required to prevent any transitional foundation from bearing on fill or retaining/basement wall backfill soil. Specifically, this refers to a footing that will transition from the retaining/basement 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/Basement Wall Footing Transitions, Option A (Included in Section IV). If footings must bear on or in retaining/basement wall backfill, the recommendations included in Surface to Retaining/Basement Wall Footing Transitions, Options B and C, must be followed. Note: retaining/basement 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.

5.4 Site Preparation

It is recommended that all vegetation 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, at a minimum, the uppermost 8.0 inches of the native 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 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. **Please note that significant over-sized aggregate (greater than 3.0 inches) will exist within the subsurface and must not be used as structural fill.** All final compaction shall be as specified herein.

Special notes for conventional surface-level systems:

> It is necessary that a minimum of 1.5 feet of engineered fill lie beneath all conventional surface-level foundations for the structures in order to utilize the bearing capacity for engineered fill for design of foundation width. 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 (subexcavation cut surface below foundations) shall not require moisture processing and compaction. Subexcavation excavation or foundation excavations may be terminated upon contact with Layer 2 or Layer 3 material provided an adequate depth has been achieved (to be verified by a representative of this firm).

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. Refer to the transition lot detail presented herein.

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 pads 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.

Foundation stepping will be required to prevent any transitional foundation from bearing on fill or retaining/basement wall backfill soil. Specifically, this refers to a footing that will transition from the retaining/basement 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/Basement Wall Footing Transitions, Option A (Included in Section IV). If footings must bear on or in retaining/basement wall backfill, the recommendations included in Surface to Retaining/Basement Wall Footing Transitions, Options B and C, must be followed. Note: retaining/basement 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.

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 8: Compaction Requirements

Material	Building Area	Percent Compaction (ASTM D698)	Compaction Moisture Content Range (%)
On-site soils with $12 \leq PI < 15$	Below Foundation Level	95 min	optimum -1 to optimum +3
	Above Foundation Level[1]	92 - 97	optimum to optimum +4
On-site 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

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
Base course	Below Interior Concrete Slabs	95 min	-

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

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 9: 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.

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.

5.5 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). Assuming the average degree of compaction will approximate 97 percent of the standard maximum density, the approximate shrinkage of the reworked native undisturbed soils are as follows:

Table 10: Shrinkage

Material	Estimated Shrinkage (Based on ASTM D698A)
Native Undisturbed Soil (Layer 1)	17% ± 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.

5.7 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 7-16:

Table 11: ASCE 7-16 Section 20.3 Table 20.3-1 Site Classification

	Site Class	$\bar{V}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{S}_u$
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		
F	Soils Requiring Site Response Analysis in Accordance with Section 21.1	See Section 20.3.1		

The formula to determine the weighted average shear wave velocity is defined below:

$$\bar{V}_s = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$

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 weighted average is shown below.

$$\bar{V}_s = \frac{100\,ft}{\frac{4.5\,ft}{483\,fps} + \frac{16.9\,ft}{2408\,fps} + \frac{78.6ft}{6918\,fps}}$$

$$\bar{V}_s = 3610\,fps$$

By calculation of the shear wave, the weighted average shear wave velocity equals 3610 feet per second for the uppermost 100 feet. The 2018 IBC Site Class **B** may be utilized in the earthquake design of the proposed site.

5.8 Conventional Surface-Level and Basement-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. Several bearing scenarios are presented below. Should a bearing or site condition be encountered that is not included in the following table, the soil engineer shall be notified to generate supplemental recommendations. Refer to the Transition Lot Detail (as presented herein) presented herein for an illustration of the recommended subexcavation and recompaction effort.

It is recommended that all perimeter foundations and isolated exterior foundations bearing on 1.5 feet of engineered fill be embedded a minimum of 2.0 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 should be founded a minimum of 2.0 feet below finish floor level. Foundations bearing on native undisturbed soil (Layer 1) must be embedded a minimum depth of 3.5 feet for an allowable soil bearing capacity of 1500 psf. Where footings will bear on Layer 2, foundations must have a minimum footing thickness of 1.5 feet. Where footings will bear on or into Layer 3, foundations must have a minimum footing thickness of 1.0 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 in the design of shallow spread (column) and continuous (wall) foundations for the proposed structures.

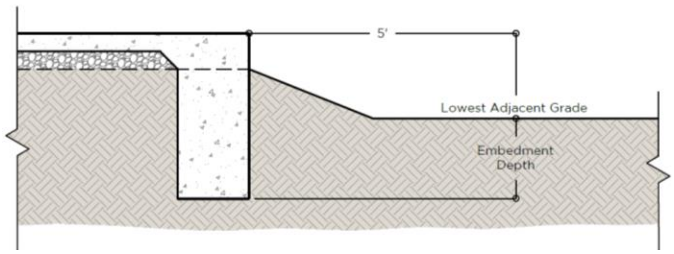
Table 12: Conventional Surface-Level and Basement-Level Spread Foundations

Foundation Embedment Depth[1]	Bearing Stratum[2]	Allowable Soil Bearing Capacity[3]
3.5 Feet	Native Undisturbed Soil (Layer 1)[4]	1500 PSF
2.0 Feet	1.5 Feet of Engineered Fill[5]	1500 PSF
Bearing at the surface of Layer 2, with a minimum footing thickness of 1.5 feet	Layer 2 occurs below depths ranging from 1.9 to 4.5 feet above highly variable depths ranging from 7.3 to 21.4 feet below the existing site grade	3000 PSF
Bearing at the surface of Layer 3, with a minimum footing thickness of 1.0 feet	Layer 3 occurs highly variable below depths ranging from 7.3 to 21.4 feet below the existing site grade	9000 PSF
Socketed 1.0 feet into Layer 3		10000 PSF (Limiting condition)

[1]Conditions for foundation embedment depth:

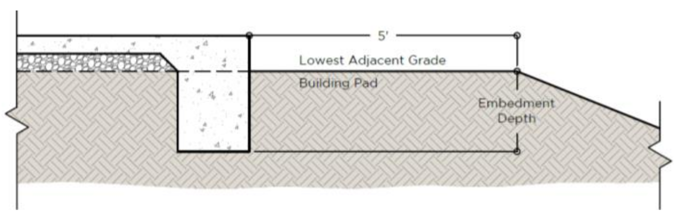
a) The depth below the lowest adjacent exterior pad grade within 5.0 feet of proposed exterior walls.

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 design (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 allowable soil bearing capacity value and associated allowable loads 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 of ½ inch 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 (Layer 1). For example, if ABC/cement slurry is used, 1.5 feet of the mixture should underlie a conventional foundation depth of 1.5 feet for an allowable soil bearing capacity of 1500 psf and ½-inch total and ¼-inch differential settlement. The preceding table shall govern the thickness of 2-sack ABC/cement slurry depending on the allowable soil bearing capacity and settlement criteria selected.

[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 for design of foundation width. 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 (subexcavation cut surface below foundations) shall not require moisture processing and compaction. Subexcavation excavation or foundation excavations may be terminated upon contact with Layer 2 or Layer 3 material provided an adequate depth has been achieved (to be verified by a representative of this firm).

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.

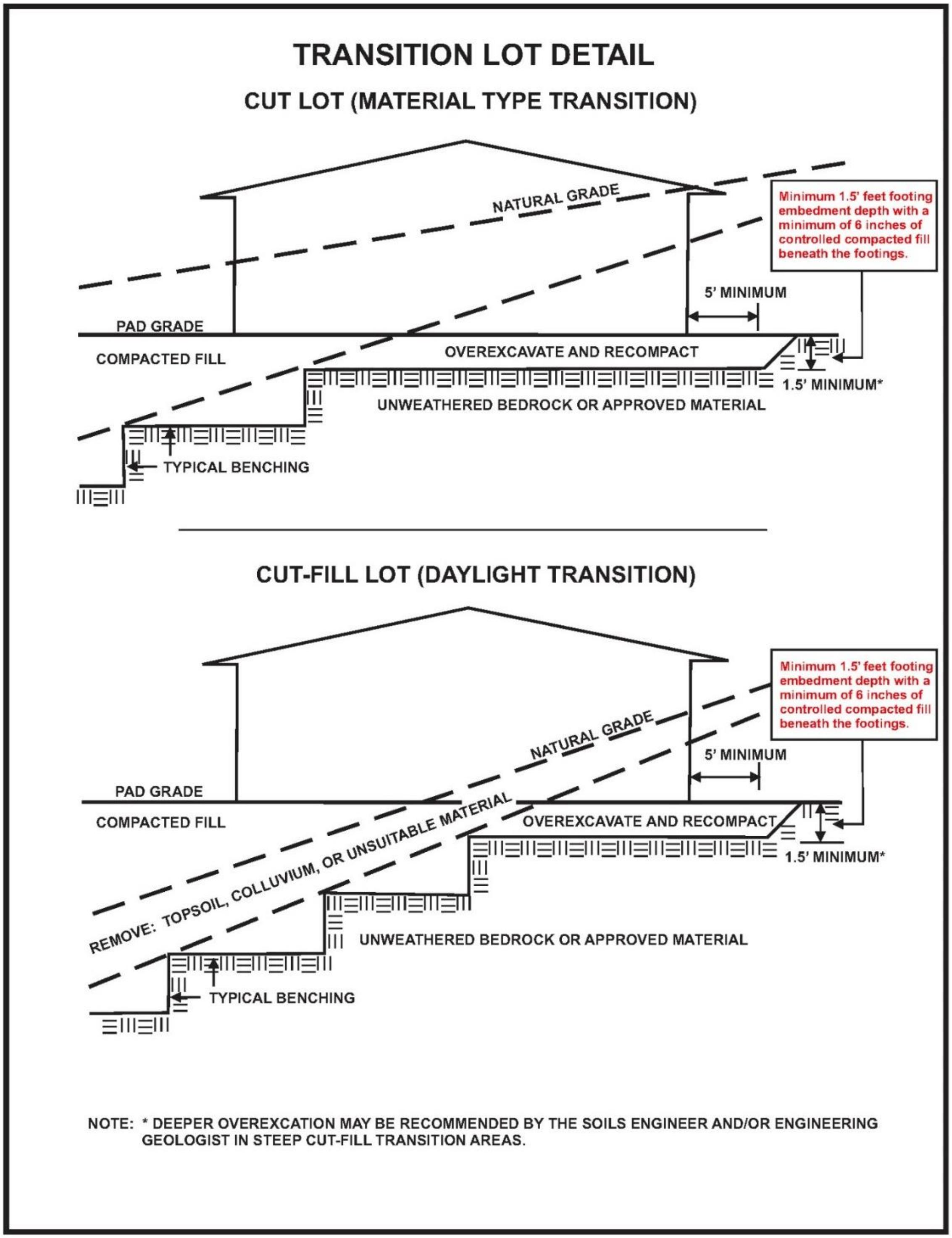


Figure 16: Transition Lot Detail

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 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/basement wall backfill soil. Specifically, this refers to a footing that will transition from the retaining/basement 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/Basement Wall Footing Transitions, Option A (Included in Section IV). If footings must bear on or in retaining/basement wall backfill, the recommendations included in Surface to Retaining/Basement Wall Footing Transitions, Options B and C, must be followed. Note: retaining/basement 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.

A minimum 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), engineered fill, Layer 2, and Layer 3:

Table 13: Lateral Stability

<table>
<tr><th>Parameter</th><th>Wall Type</th><th>Engineered Fill</th><th>Native Undisturbed Soil (Layer 1)</th><th>Layer 2[c]</th><th>Layer 3[c]</th></tr>
<tr><td>Active (K_a) Pressure[a]</td><td>Free-end</td><td colspan="4">34 psf/ft</td></tr>
<tr><td>At-Rest (K_o) Pressure[a]</td><td>Fixed-end[b]</td><td colspan="4">52 psf/ft</td></tr>
<tr><td rowspan="2">Passive (K_p) Resistance</td><td>Free-end/Fixed-end independent of base friction</td><td>358 psf/ft</td><td>291 psf/ft</td><td>546 psf/ft</td><td>888 psf/ft</td></tr>
<tr><td>Fixed-end in conjunction with base friction</td><td>240 psf/ft</td><td>195 psf/ft</td><td>366 psf/ft</td><td>595 psf/ft</td></tr>
<tr><td rowspan="2">Coefficient of Base Friction (μ)</td><td>Free-end/Fixed-end independent of passive resistance</td><td>0.62</td><td>0.53</td><td>0.87</td><td>0.97</td></tr>
<tr><td>Free or Fixed-end in conjunction with passive resistance</td><td>0.42</td><td>0.36</td><td>0.52</td><td>0.65</td></tr>
</table>

[a]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.

[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

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 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 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 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 designs 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.

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.

5.11 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 3 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 roof-line. 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, rain-water 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.

5.12 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 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.

Table 14: Foundation Design Alterations Due to Landscaping

Flowers & Shrub Planting Distance	Tree Planting Distance	Design Changes
5 feet	10 feet	-
4 feet[1]	9 feet	Increase footing embedment depth by 6.0 inches[2]
3 feet[1]	8 feet	Increase footing embedment depth by 12.0 inches[2]
2 feet[1]	7 feet	Increase footing embedment depth by 18.0 inches[2]

[1]Verification from the landscape architect that low water consumption plants are being installed must be submitted to this office for approval.

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

Ground cover plants with low water requirements may be acceptable for landscaping near foundations. Ground cover vegetation helps to reduce fluctuations in the soil moisture content. Limit the watering to the minimum needed to maintain the ground cover vegetation near foundations. 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 far away from foundations in irrigated areas, the chances of foundation damage are greatly reduced. If irrigation/sprinkler systems are to be used, we recommended installing the system all around the structure to provide uniform moisture throughout the year. 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.

5.13 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, which as a result of economic considerations, accept 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 previous foundation recommendations are not designed 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 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 6.0-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.

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

9013 NORTH 24TH AVENUE, SUITE 7, PHOENIX, ARIZONA 85021
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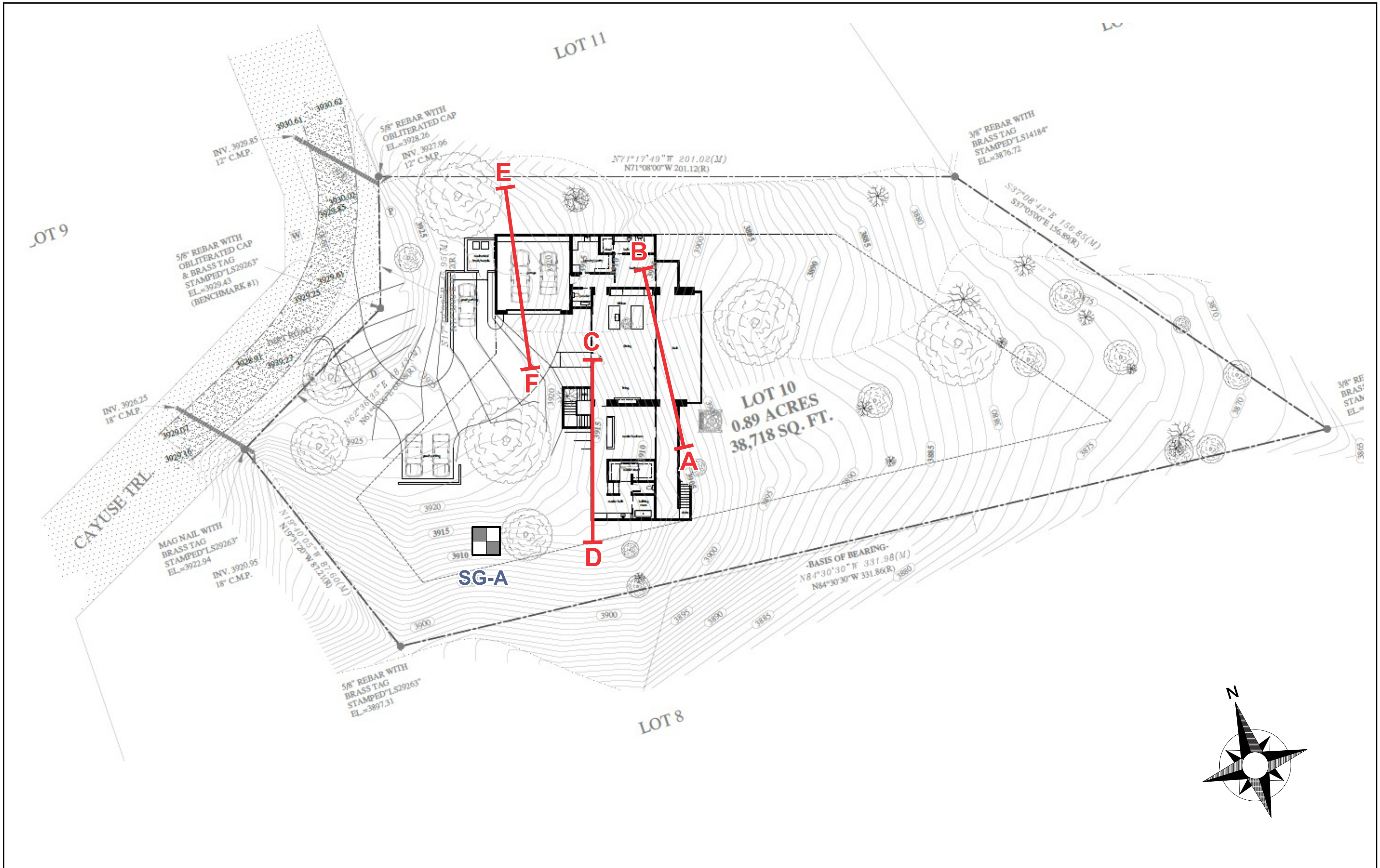


SITE PLAN | PROJECT 28171

PROPOSED HUMPHREY - BORGOGNI RESIDENCE
APN 408-19-009
LOT 10 - CUP OF GOLD ESTATES
90 CAYUSE TRAIL
SEDONA, ARIZONA

SEISMIC SURVEY LOCATION

SUBGRADE SAMPLE LOCATION

VELOCITY CLASSIFICATION DATA

Proposed Humphrey - Borgogni Residence
APN 408-19-009
Lot 10 - Cup of Gold Estates
90 Cayuse Trail
Sedona, Arizona

Average Velocity of Layer 1: 832 fps (635 to 947)

Average Velocity of Layer 2: 4152 fps (3175 to 4927)

Average Depth to Layer 2: 2.8 feet

Range: 1.9 to 4.5 feet

Average Velocity of Layer 3: 11928 fps (9531 to 13695)

Average Depth to Layer 3: 13.8 feet

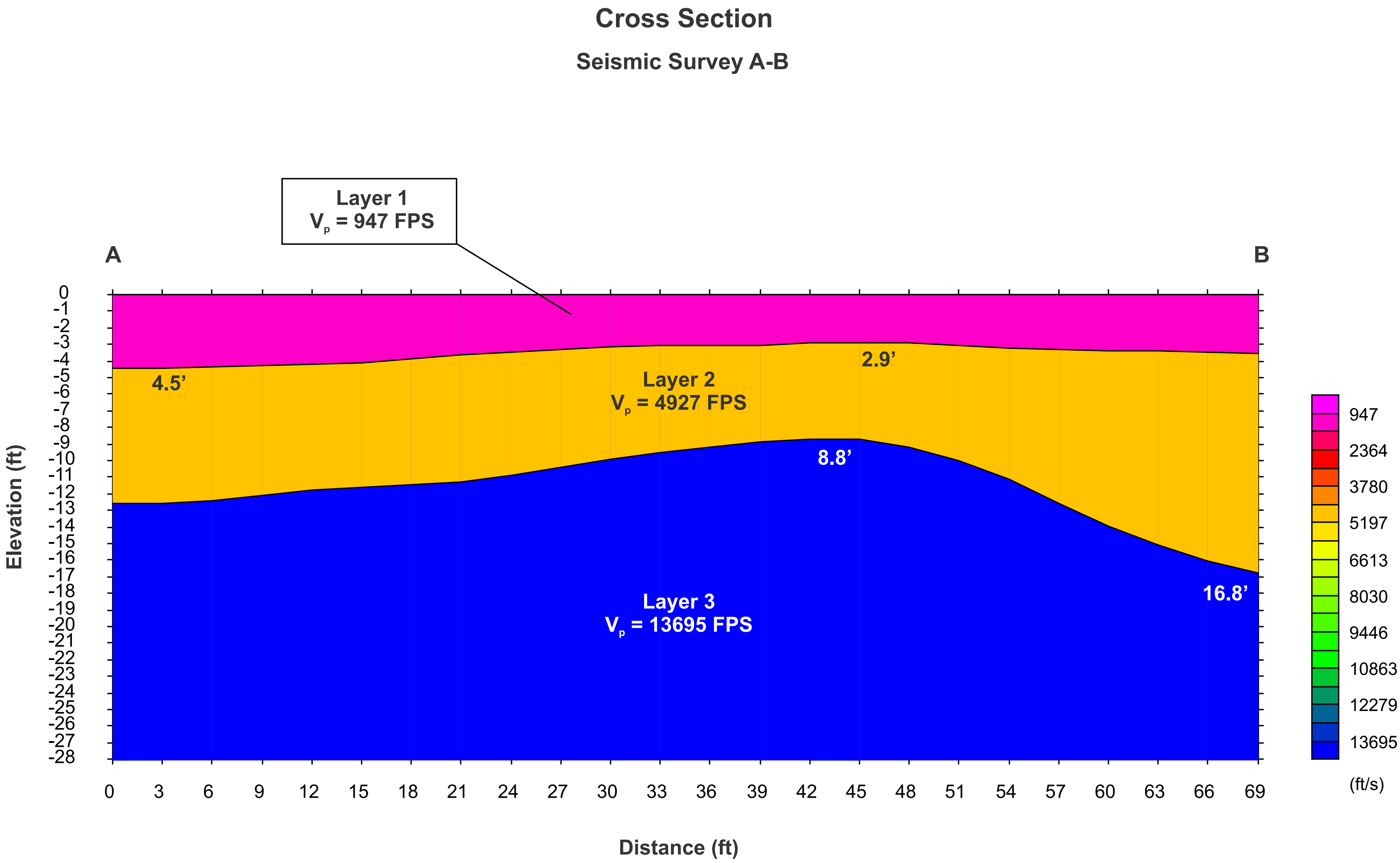
Range: 7.3 to 21.4 feet

Layer 1: Loose coarse-grained alluvium/residual soil comprised of silty sandy clayey gravel (GC)

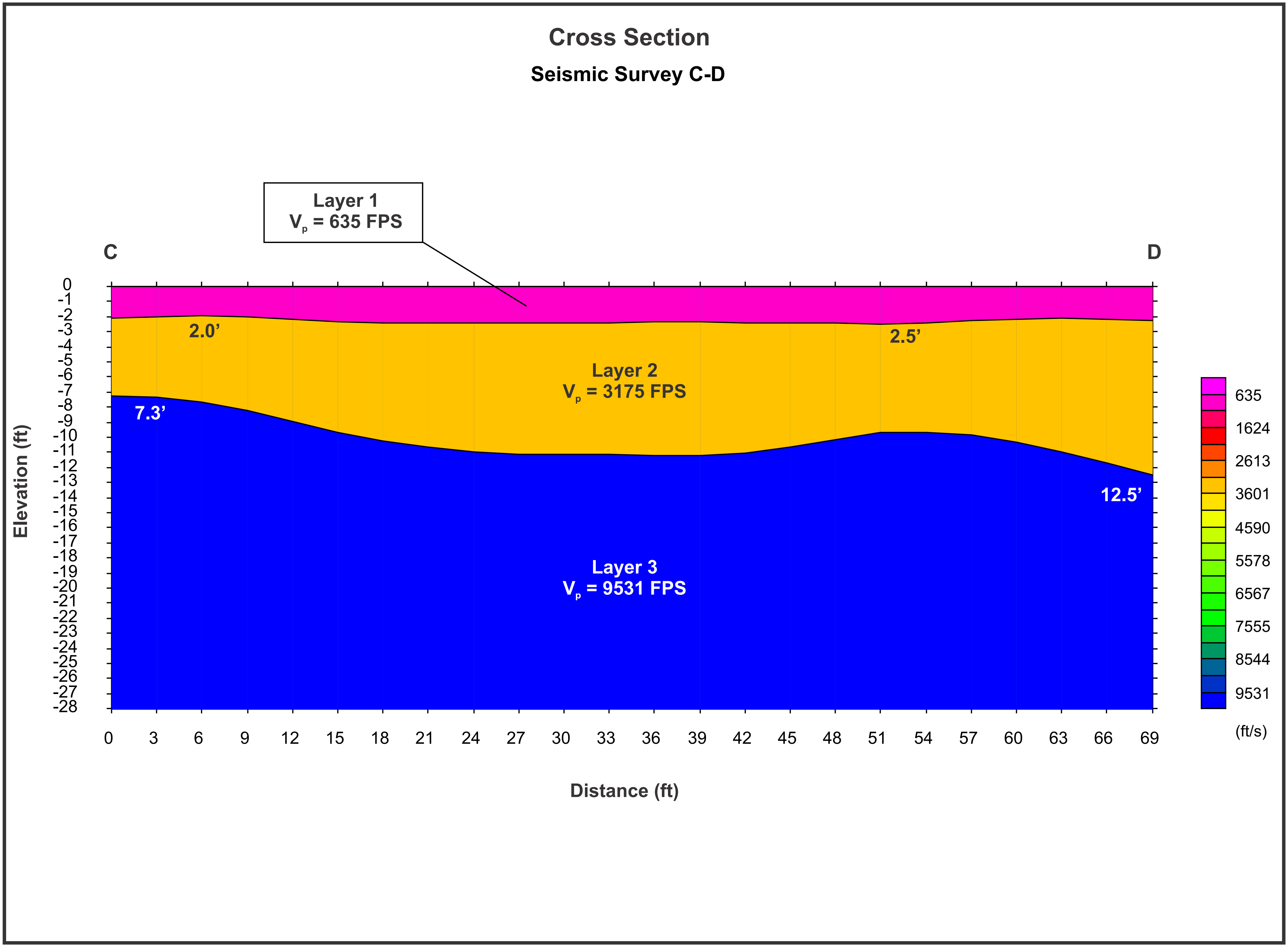
Layer 2: Highly to moderately weathered and fractured, poor, weak, Schnebly Hill Formation (part of the Supai Group) which includes siltstone, sandstone, claystone, mudstone

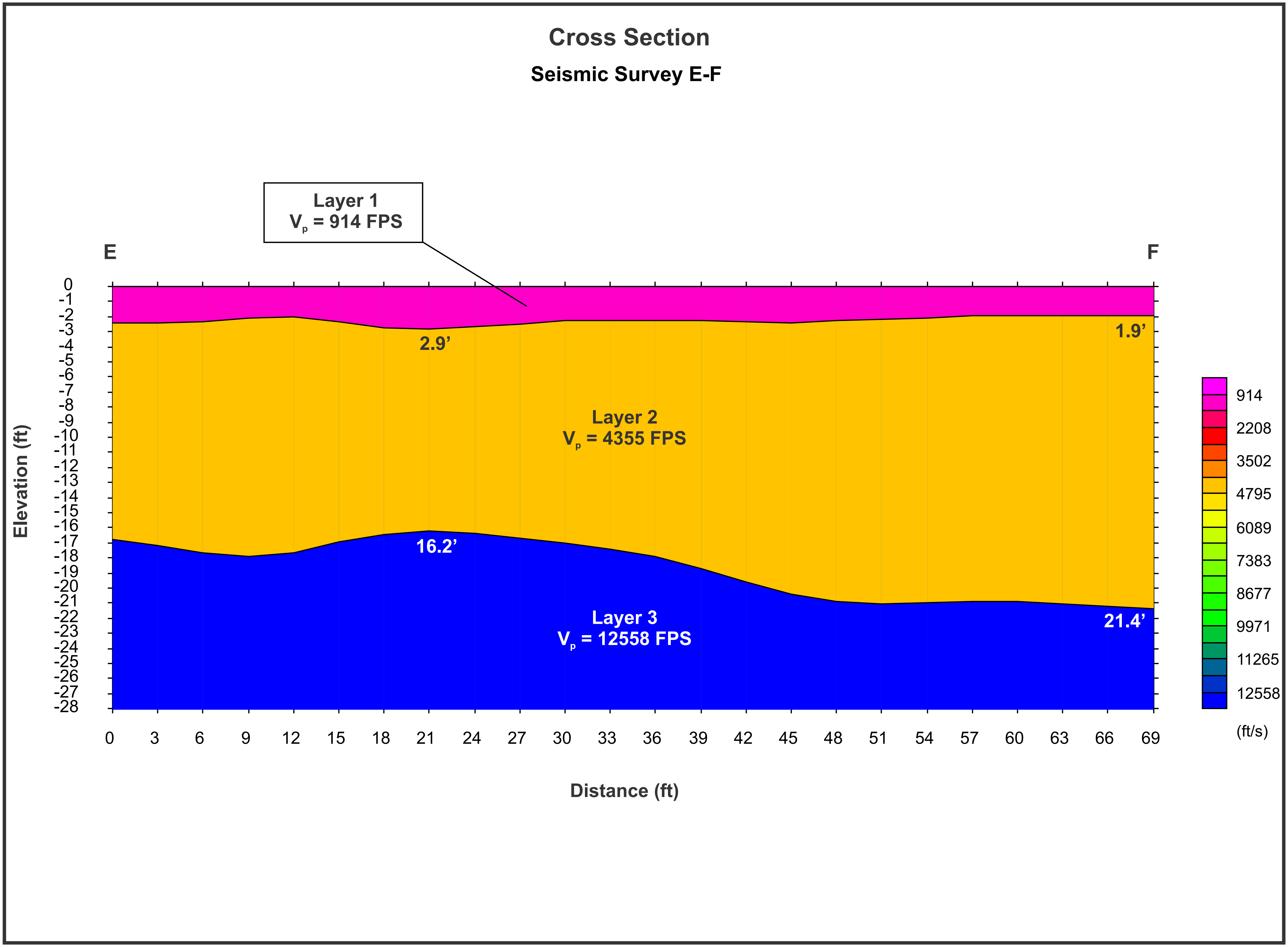
Layer 3: Slightly weathered and fractured,exellent, extremely strong Schnebly Hill Formation (part of the Supai Group) which includes siltstone, sandstone, claystone, mudstone

Line	Layer 1			Layer 2			Layer 3		
	Velocity	Depth (ft)		Velocity	Depth (ft)		Velocity	Depth (ft)	
A_B	947	-	-	4927	2.9	4.5	13695	8.8	16.8
C-D	635	-	-	3175	2.0	2.5	9531	7.3	12.5
E-F	914			4355	1.9	2.9	12558	16.2	21.4
Avgs	832	-		4152	2.8		11928	13.8	



Cross Section
Seismic Survey A-B
Layer 1
V_p = 947 FPS
A
B
4.5'
2.9'
Layer 2
V_p = 4927 FPS
8.8'
16.8'
Layer 3
V_p = 13695 FPS
Elevation (ft)
0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11 -12 -13 -14 -15 -16 -17 -18 -19 -20 -21 -22 -23 -24 -25 -26 -27 -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)
947
2364
3780
5197
6613
8030
9446
10863
12279
13695
(ft/s)

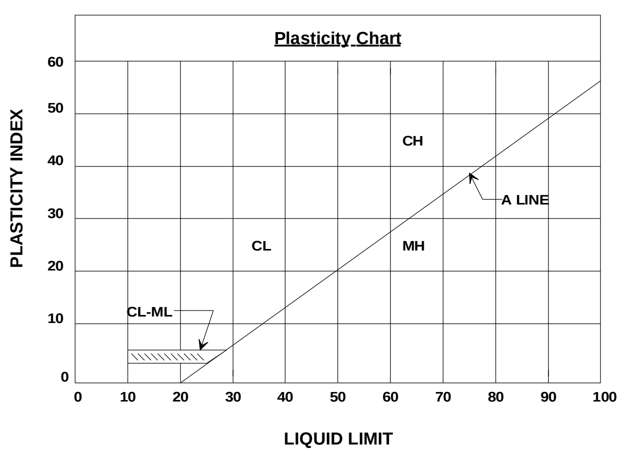

Cross Section
Seismic Survey C-D
Layer 1
V_p = 635 FPS
C
D
2.0'
2.5'
Layer 2
V_p = 3175 FPS
7.3'
12.5'
Layer 3
V_p = 9531 FPS
Elevation (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
-16
-17
-18
-19
-20
-21
-22
-23
-24
-25
-26
-27
-28
0
3
6
9
12
15
18
21
24
27
30
33
36
39
42
45
48
51
54
57
60
63
66
69
Distance (ft)
635
1624
2613
3601
4590
5578
6567
7555
8544
9531
(ft/s)


Cross Section
Seismic Survey E-F
Layer 1
V_p = 914 FPS
E
F
2.9'
1.9'
Layer 2
V_p = 4355 FPS
16.2'
21.4'
Layer 3
V_p = 12558 FPS
Elevation (ft)
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
-16
-17
-18
-19
-20
-21
-22
-23
-24
-25
-26
-27
-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)
914
2208
3502
4795
6089
7383
8677
9971
11265
12558
(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

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

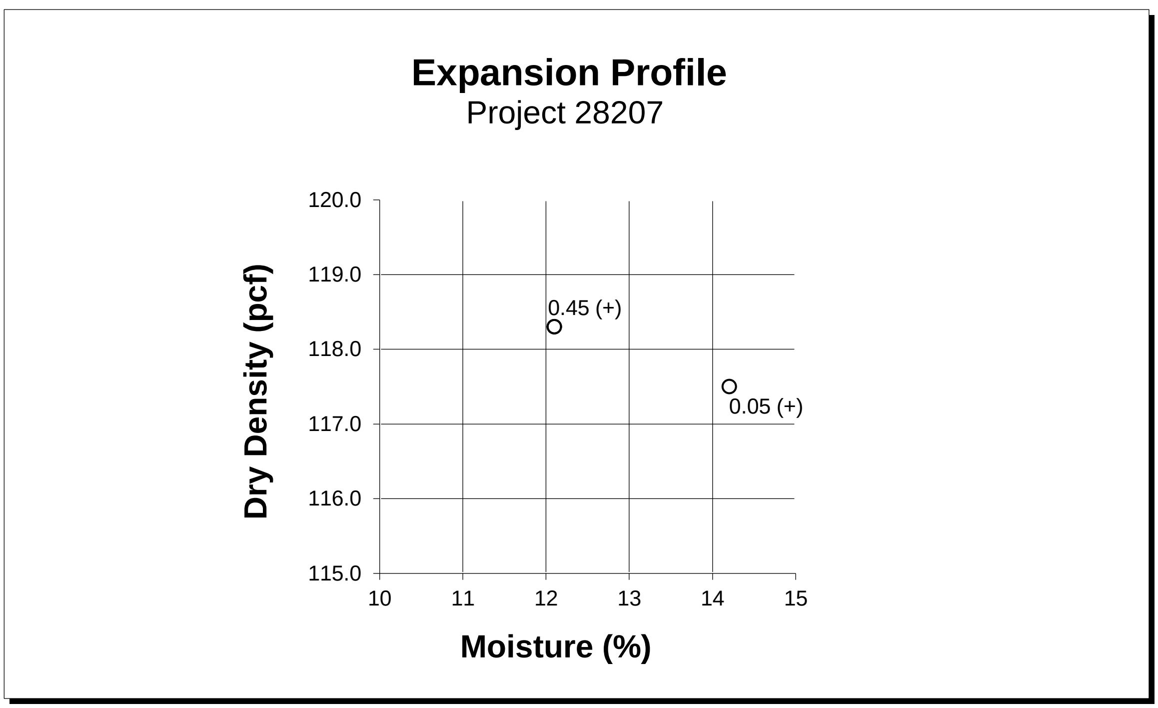
SECTION III

EXPANSION TEST DATA

PROPOSED HUMPHREY-BORGOGNI RESIDENCE
APN 408-19-009
LOT 10 - CUP OF GOLD ESTATES
90 CAYUSE TRAIL
SEDONA, ARIZONA

Sample Location	*Remolded Moisture Content (%)*	*Dry Density (PCF)*	*Volume Change After Saturation (%)*	*Adjusted Volume Change After Saturation (%)*
SG-A (0.0'-1.5')	14.2	117.5	0.08 (+)	0.05 (+)
SG-A (0.0'-1.5')	12.1	118.3	0.74 (+)	0.45 (+)

(+) denotes expansion
(-) denotes compression



CLASSIFICATION TEST DATA

PROPOSED HUMPHREY-BORGOGNI RESIDENCE
APN 408-19-009
LOT 10 – CUP OF GOLD ESTATES
90 CAYUSE TRAIL
SEDONA, ARIZONA

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’-1.5’)	-	100	91	60	48	42	-	36	29	12	GC	4.5

Project 28207
Vann Engineering, Inc. - Phoenix, Arizona

SECTION IV

9013 NORTH 24TH AVENUE, SUITE 7, PHOENIX, ARIZONA 85021
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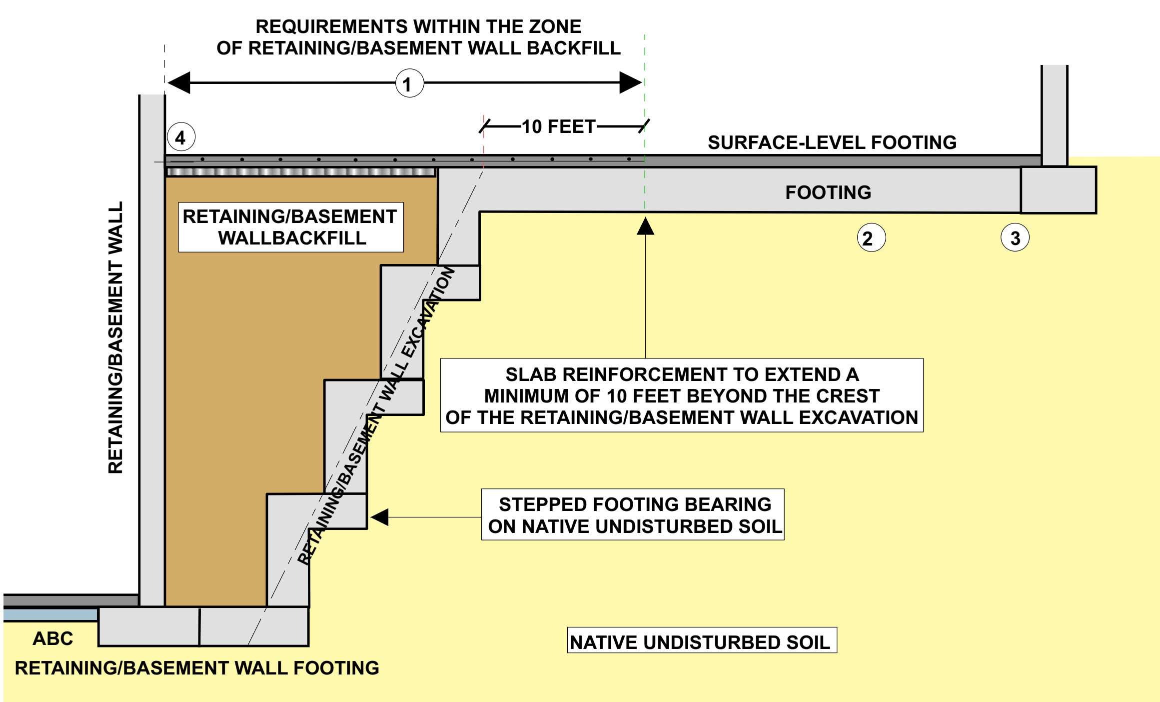
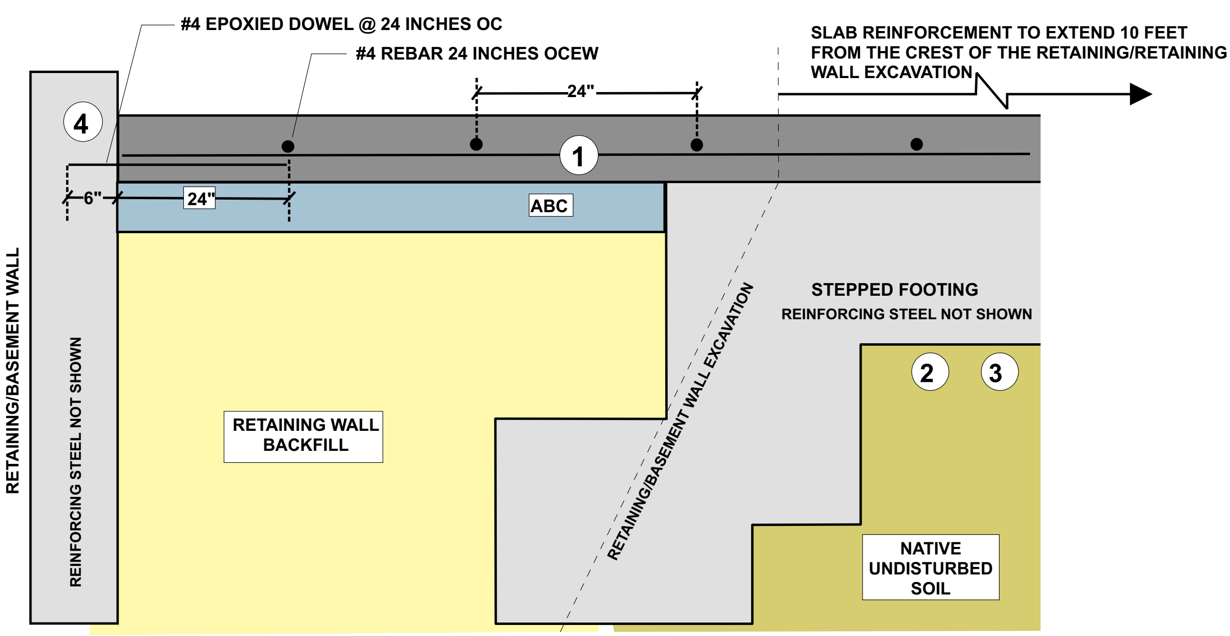
SURFACE TO RETAINING/BASEMENT WALL FOOTING TRANSITIONS

OPTION A: (CROSS SECTION)

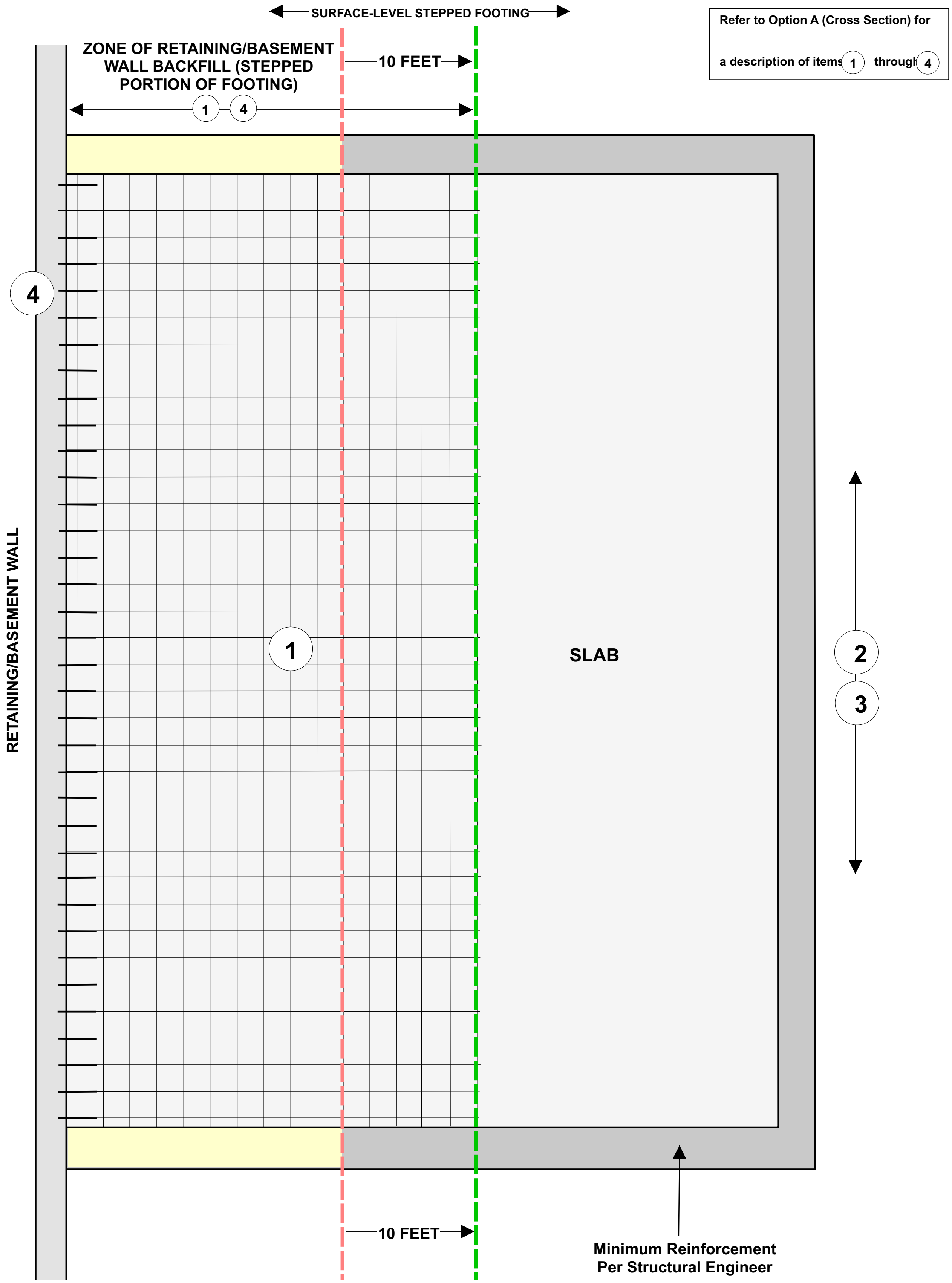


1. REINFORCE SLAB WITH #4 REBAR @ 24 INCHES OCEW, CHAIRED, 100 PERCENT TIED, AND CONNECTED TO THE FOOTING STEEL
2. REFER TO EARTHWORK SECTION FOR REQUIRED ZONE OF SCARIFICATION BENEATH SLABS, SIDEWALKS, PARKING AREAS, ETC.
3. REFER TO SURFACE-LEVEL FOUNDATION TABLES FOR MINIMUM FOOTING DEPTHS AND ASSOCIATED BEARING CAPACITIES (NOTE: CONTROLLED AND OR IMPORTED COMPACTED FILL MAY BE REQUIRED BELOW FOOTINGS)
4. #4 EPOXIED DOWEL @ 24 INCHES OC, MINIMUM 6 INCH EMBEDMENT INTO RETAINING/BASEMENT WALL (LAP AND TIE 24 INCHES TO THE SLAB STEEL)



- ALL REINFORCING STEEL AND DETAILS SHOWN ABOVE TO BE VERIFIED BY A REGISTERED STRUCTURAL ENGINEER
- ILLUSTRATIONS NOT TO SCALE
- REFER TO OPTION A (PLAN VIEW)

OPTION A: (PLAN VIEW)



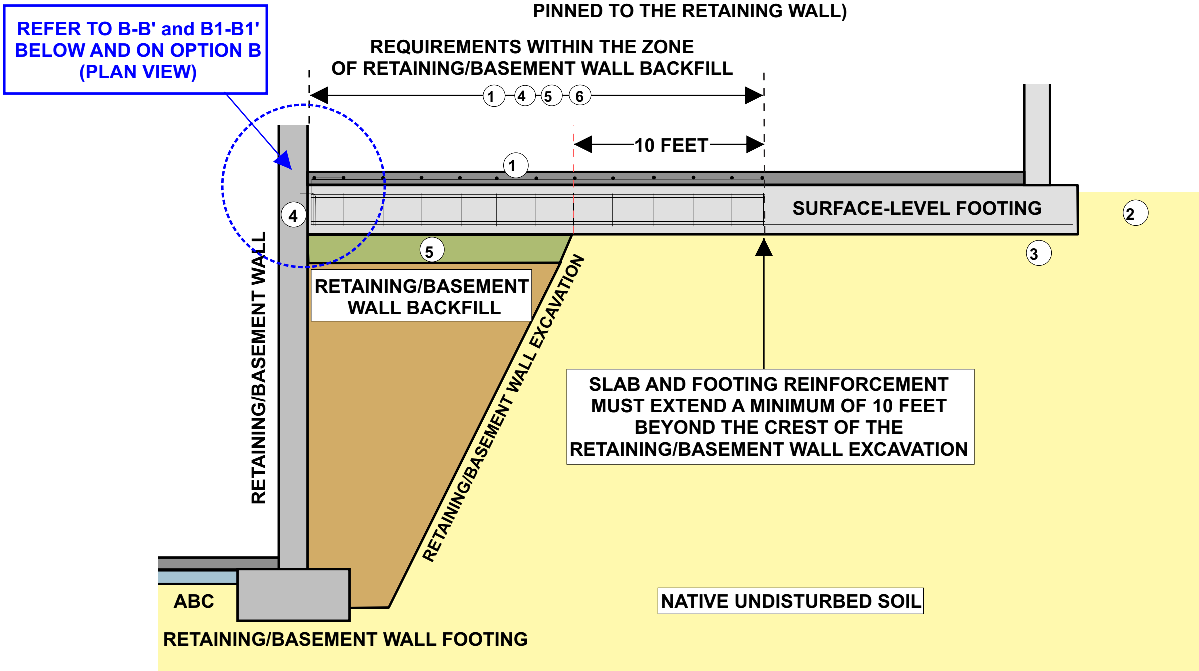
EXPLANATION

LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

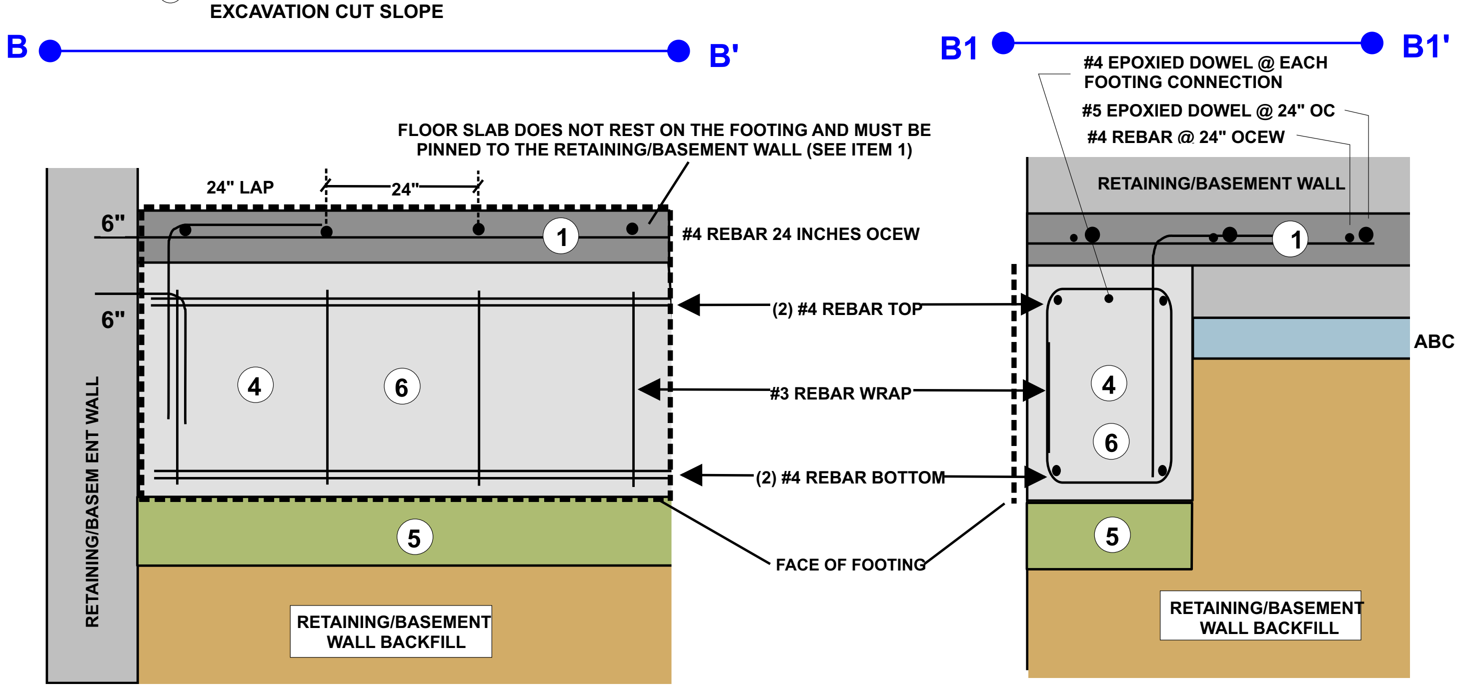
LINE DENOTING A POINT 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE

OPTION B: (CROSS SECTION)

SURFACE-LEVEL FOOTINGS BEARING ON RETAINING/BASEMENT WALL BACKFILL (MUST BE PINNED TO THE RETAINING WALL)

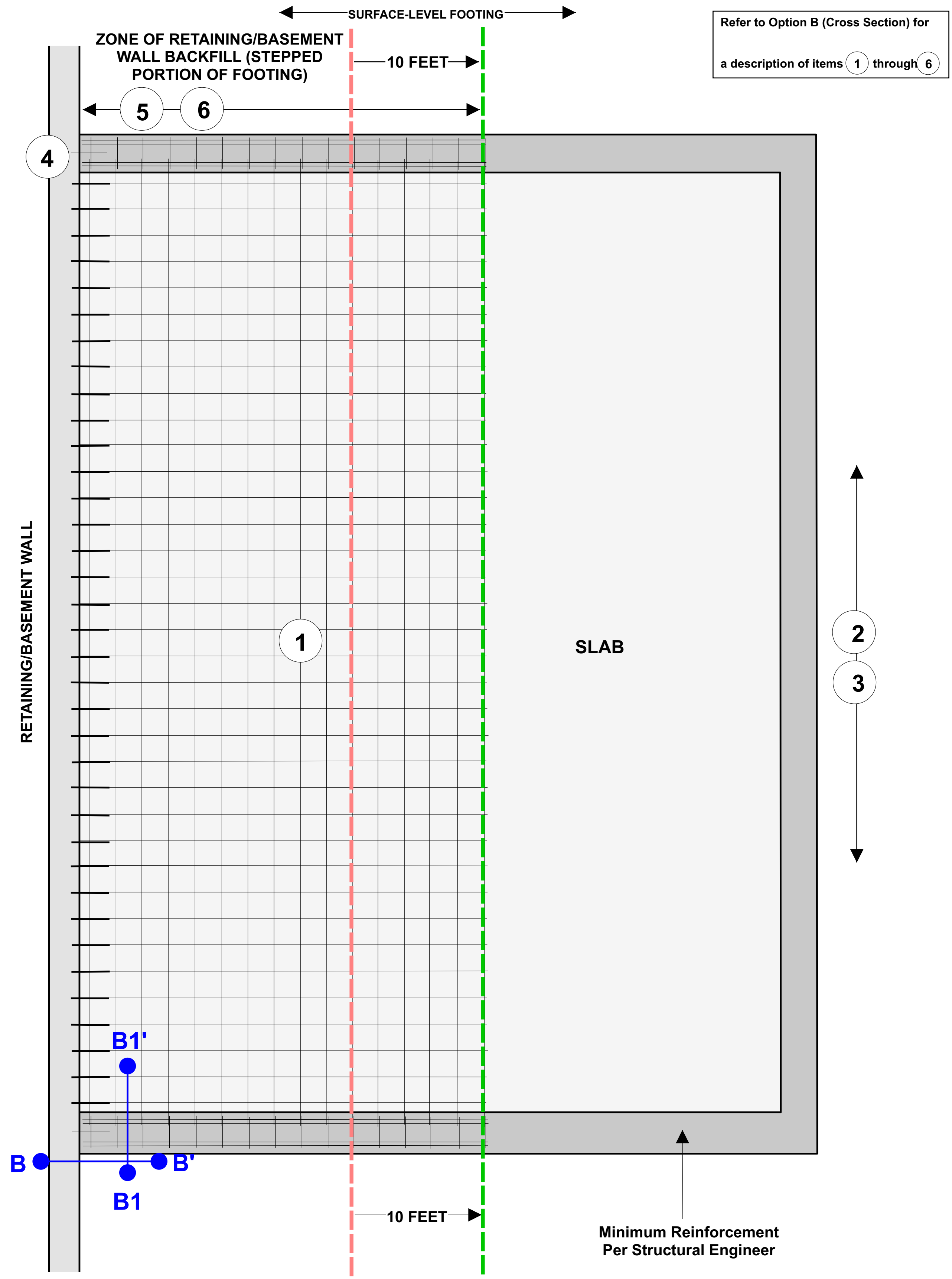


1. REINFORCE SLAB WITH #4 REBAR @ 24 INCHES OCEW, CHAIRED, 100 PERCENT TIED, AND CONNECTED TO THE FOOTING STEEL. FLOOR SLAB MUST BE TIED TO THE BASEMENT/RETAINING WALL WITH #5 EPOXIED DOWELS @ 24 INCHES OC
2. REFER TO EARTHWORK SECTION FOR REQUIRED ZONE OF SCARIFICATION BENEATH SLABS, SIDEWALKS, PARKING AREAS, ETC.
3. REFER TO SURFACE-LEVEL FOUNDATION TABLES FOR MINIMUM FOOTING DEPTHS AND ASSOCIATED BEARING CAPACITIES (NOTE: CONTROLLED AND OR IMPORTED COMPACTED FILL MAY BE REQUIRED BELOW FOOTINGS)
4. DOUBLE REINFORCE FOOTINGS (2 #4 REBAR TOP, 2 #4 REBAR BOTTOM, 1 #3 WRAP @ 24 INCHES OC) AND TIE WITH #4 EPOXIED PINS INTO THE BASEMENT WALL @ EACH FOOTING CONNECTION (6 INCH MINIMUM EMBEDMENT)
5. HAND-TAMP (COMPACT) THE BOTTOM 6 INCHES OF THE FOOTING EXCAVATION, WITHIN THE ZONE OF RETAINING/BASEMENT WALL BACKFILL, TO A MINIMUM OF 95% OF THE MAXIMUM ASTM D698 DRY DENSITY
6. DOUBLE WIDTH OR DOUBLE DEPTH OF FOOTING; COMMENCING 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE



- ALL REINFORCING STEEL AND DETAILS SHOWN ABOVE TO BE VERIFIED BY A REGISTERED STRUCTURAL ENGINEER
- ILLUSTRATIONS NOT TO SCALE
- REFER TO OPTION B (PLAN VIEW)

OPTION B: (PLAN VIEW)



EXPLANATION

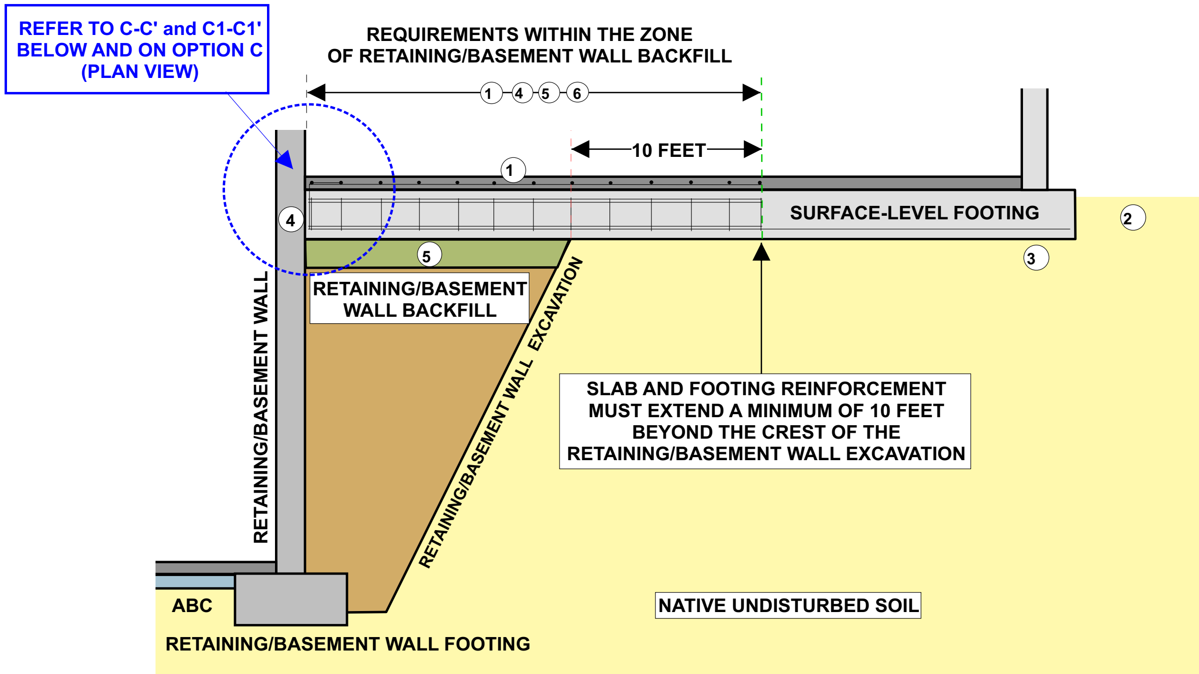
LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

LINE DENOTING A POINT 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE

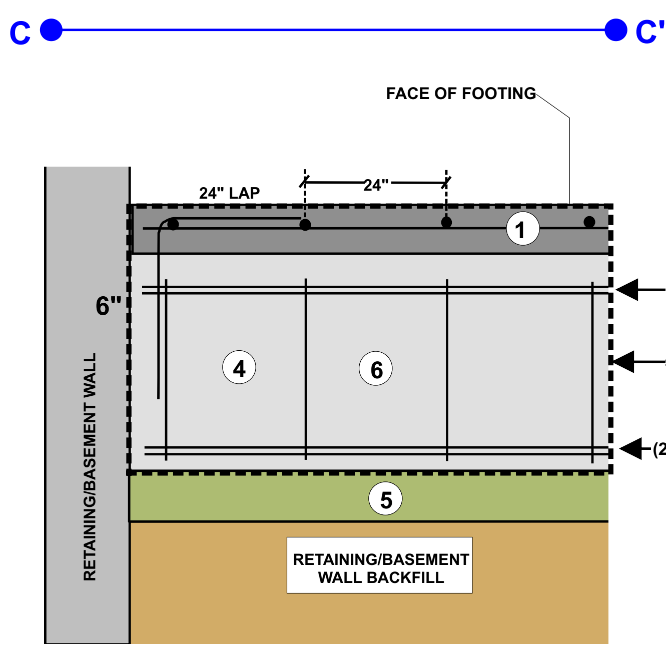
CROSS SECTIONAL VIEW - REFER TO OPTION B (CROSS SECTION)

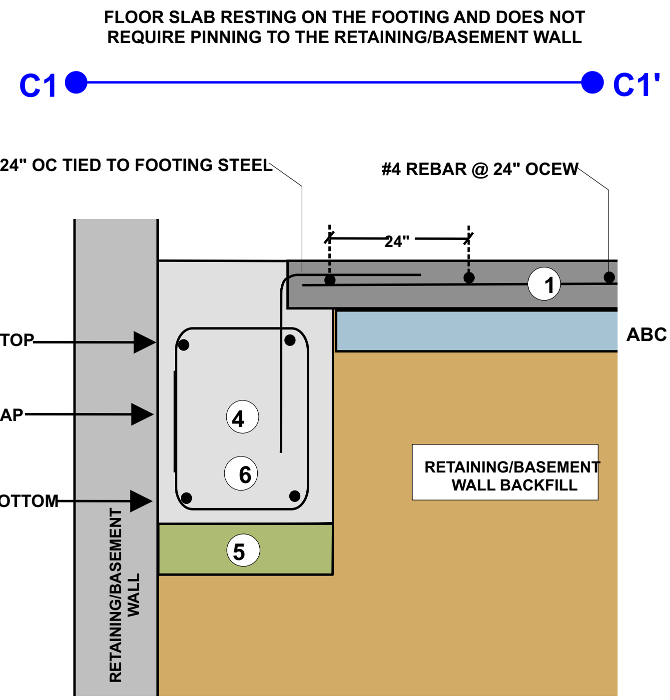
OPTION C: (CROSS SECTION)

SURFACE-LEVEL FOOTINGS BEARING ON RETAINING/BASEMENT WALL BACKFILL

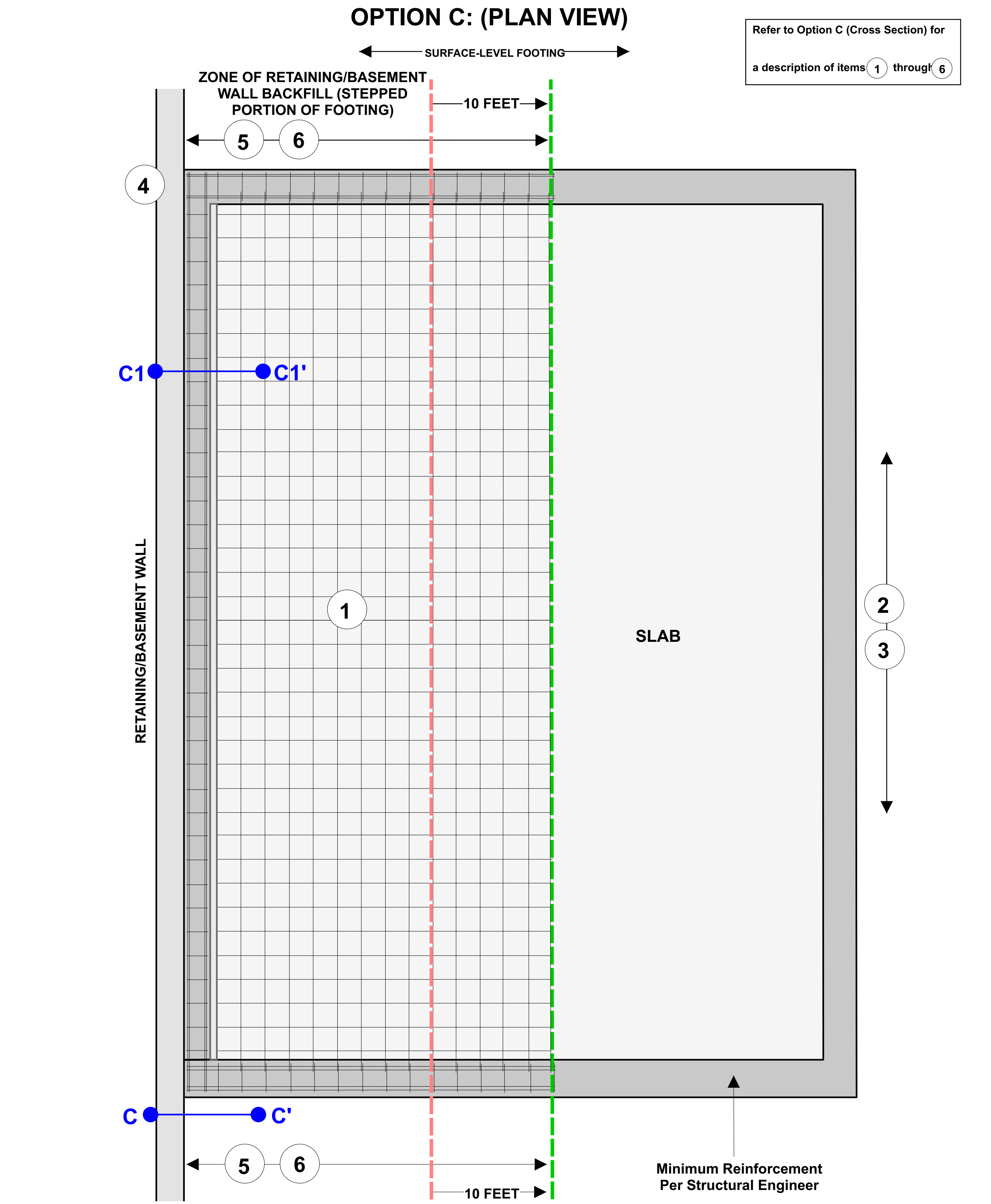


1. REINFORCE SLAB WITH #4 REBAR @ 24 INCHES OCEW, CHAIRED, 100 PERCENT TIED, AND CONNECTED TO THE FOOTING STEEL.
2. REFER TO EARTHWORK SECTION FOR REQUIRED ZONE OF SCARIFICATION BENEATH SLABS, SIDEWALKS, PARKING AREAS, ETC.
3. REFER TO SURFACE-LEVEL FOUNDATION TABLES FOR MINIMUM FOOTING DEPTHS AND ASSOCIATED BEARING CAPACITIES (NOTE: CONTROLLED AND OR IMPORTED COMPACTED FILL MAY BE REQUIRED BELOW FOOTINGS)
4. DOUBLE REINFORCE FOOTINGS (2 #4 REBAR TOP, 2 #4 REBAR BOTTOM, 1 #3 WRAP @ 24 INCHES OC)
5. HAND-TAMP (COMPACT) THE BOTTOM 6 INCHES OF THE FOOTING EXCAVATION, WITHIN THE ZONE OF RETAINING WALL BACKFILL, TO A MINIMUM OF 95% OF THE MAXIMUM ASTM D698 DRY DENSITY
6. TRIPLE WIDTH OR DOUBLE DEPTH OF FOOTING; COMMENCING 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE





- ALL REINFORCING STEEL AND DETAILS SHOWN ABOVE TO BE VERIFIED BY A REGISTERED STRUCTURAL ENGINEER
- ILLUSTRATIONS NOT TO SCALE
- REFER TO OPTION C (PLAN VIEW)



EXPLANATION

LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

LINE DENOTING A POINT 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE

CROSS SECTIONAL VIEW - REFER TO OPTION C (CROSS SECTION)

VANN ENGINEERING INC

Geotechnical Engineering Enviromental Consulting Construction Testing & Inspection

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