VANN ENGINEERING INC

Geotechnical Investigation Report

PROPOSED SCHERR RESIDENCE
APN 168-75-029
5416 EAST DESERT JEWEL DRIVE
PARADISE VALLEY, ARIZONA 85253

Prepared for:

Mr. Jason Scherr
6843 Lost Garden Terrace
Parkland, Florida 33076

November 21, 2017

Project 25548

1980 – 2017
37
YEARS OF EXCELLENCE
Vann Engineering Inc.

VANN ENGINEERING INC
GEOTECHNICAL ENGINEERING ▪ ENVIRONMENTAL CONSULTING ▪ CONSTRUCTION TESTING & OBSERVATION

November 21, 2017 Project 25548

Mr. Jason Scherr
6843 Lost Garden Terrace
Parkland, Florida 33076

RE: GEOTECHNICAL INVESTIGATION REPORT
PROPOSED SCHERR RESIDENCE
APN 168-75-029
5416 EAST DESERT JEWEL DRIVE
PARADISE VALLEY, ARIZONA 85253

Mr. Scherr:

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. As an additional service, this firm would be pleased to review the project plans and structural notes for conformance to the intent of this report. We trust that this report will assist you in the design and construction of the proposed project. Vann Engineering, Inc. appreciates the opportunity to provide our services on this project and looks forward to working with you during construction and on future projects. **This firm possesses the capability of performing testing and inspection services during the course of construction. Such services include, but are not limited to, compaction testing as related to fill control, foundation inspections and concrete sampling. Please notify this firm if a proposal for these services is desired.** Should any questions arise concerning the content of this report, please feel free to contact this office as soon as possible.

Respectfully submitted,

VANN ENGINEERING, INC.

Mark Smelser, BS
Project Geologist

Registered Professional Engineer (Geological)
CERTIFICATE NO. 15208
JEFFRY D. VANN
Date Signed 11/21/17
ARIZONA, U.S.A.
Expires 09/30/19

Jeffry D. Vann, MS, PE, F.ASCE
Principal Engineer

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SECTION I

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1.0 INTRODUCTION

This document presents the results of a geotechnical investigation report conducted by Vann Engineering, Inc. for the:

PROPOSED SCHERR RESIDENCE
APN 168-75-029
5416 EAST DESERT JEWEL DRIVE
PARADISE VALLEY, ARIZONA 85253

The services performed provide an evaluation at selected locations of the subsurface soil conditions throughout the zone of significant foundation influence. The following aerial photograph (Figure 1) shows the site conditions.

Figure 1: Aerial photograph of site and immediate vicinity

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 below.

Table 1: Design Loads

Foundation Type	Maximum Column Load (KIPS)	Maximum Wall Load (KLF)
Conventional, shallow, lightly loaded surface-level spread foundations with total and differential settlements limited to ½ inch and ¼ inch, respectively.	100	5.0

Anticipated structural loads in excess of 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
- Explanation of applicable geologic hazards
- Recommendations for shallow surface level and deep-cut basement-level spread foundations; allowable bearing capacity based on a settlement analysis of ½ inch total settlement and ¼ inch differential settlement
- Recommendations for fixed and free-end retaining walls
- General excavation conditions
- Lateral stability analysis including active pressure, passive pressure and base friction
- Recommendations for site grading
- Recommendations for cut slope stability
- Recommendations for drainage and slab support
- 2012 IBC site classification

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

1.3 Authorization

The obtaining of data from the site and the preparation of this geotechnical investigation report have been carried out according to this firm's proposal (**VE17GT0808K dated 8/8/17)**, authorized by **Jason Scherr**, to proceed with the work. Our efforts and report are limited to the scope and limitations set forth in the proposal.

1.4 Standard of Care

Since our investigation is based upon review of background data, observation of site materials, and engineering analysis, the conclusions and recommendations are professional opinions. Our professional services have been performed using that degree and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities.

These opinions have been derived in accordance with current standards of practice and no other warranty, express or implied, is made. The limitations of this report and geotechnical issues which further explain the limitations of the information contained in this report are listed at 7.0.

2.0 PROJECT DESCRIPTION

2.1 Proposed Development

Vann Engineering, Inc. understands that a new custom home is proposed for construction at the above-mentioned site, with a planned basement level.

2.2 Site Description

The subject site consists of hillside terrain that slopes down to the west. The site is moderately vegetated. Cobble to small boulder-sized particles were observed scattered across the site surface. Numerous rock outcrops were observed scattered across the site, including a large outcrop at the southeastern corner of the site.

Please refer to the following photographs which show the nature of the site at the time of the field investigation.

Figure 2: General site conditions showing rock outcrops

Figure 3: General site conditions showing large rock outcrop at the southeast corner 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) 24-channel 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 length of each seismic survey was 72.0 feet, thereby allowing an examination of the subsurface to a depth of 28.0 feet below the existing site grade.

Information pertaining to the subsurface profile was obtained through analysis of seismic refraction data and geological observations of the site. Seismic wave velocities, representative of the various strata, are listed in Section I of this report. Note: Changes in the calculated velocity indicate strata breaks or distinct changes within the same stratum. The important concept to remember with this method is that it is predominantly effective where velocities increase from layer to layer, moving downward from the surface. Analytical methods are used by this firm for determining the depth to the various layers, even in the most complex multi-layer situations. However, when a denser harder soil or rock layer overlies a weaker or less dense soil or rock layer, the weaker or less dense layer is masked and not detected by the seismograph. Thus, the Cross Sections presented herein may not reveal a possible weaker underlying layer, within or below the depicted layers. If a weaker layer is encountered during the excavation efforts, this office should be contacted immediately for further recommendations.

Generally, the depth of a seismic survey investigation is approximately equal to one-third the length of the survey. For example, if it is desired to examine the substrata to a depth of 20.0 feet, the survey should extend a distance of 60.0 feet. However, seismic survey exploration depths, as mentioned above and depicted on the Cross Sections presented herein, are calculated by using a computer program (SeisImager 2D) that generates cross sections of the subsurface geology at each seismic survey location. Further, total exploration depths, as stated above, of the seismic survey study may vary from one survey line to the next. Furthermore, the calculated depths are dependent on the program’s ability to interpret the subsurface layering, and are based primarily on the penetration and refraction of the seismic wave into and through the subsurface stratum. Thus, the actual seismic survey exploration depths were 28.0 feet below the existing grade, regardless of the length of the survey lines.

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

3.2 Laboratory Testing

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

Table 2: Laboratory Testing

Test	Sample(s)	Purpose
Sieve Analysis / Atterberg Limits	Native subgrade soils (1)	Soil classification
Moisture Only	Native subgrade soils (1)	Determination of in-situ moisture content

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 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	Velocity Range (FPS)	Average Velocity (FPS)	Average Depth[1, 2]	Classification	USCS
1	1092 to 2130	1722	Above 2.3 feet	Gravelly silty clayey sand	SC-SM
2	4287 to 5405	4832	Below 2.3 feet and above 7.9 (Note: Layer 2 outcrops occur at the surface at several locations across the site)	Highly to moderately weathered and fractured quartz-muscovite schist	-
3	9279 to 10231	9716	Below 7.9 feet	Slightly weathered and fractured quartz-muscovite schist	-

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

[2]The depth to Layers 2 approaches zero moving towards visible outcrops.

Refer to the cross sections in Section II 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.

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 soils encountered at the site are considered cohesionless (plasticity index of 5) based on the laboratory testing. Based on field and laboratory test data, this firm has determined that the potential for soil expansion is low for the native soils.

Collapsible soils are typically comprised of silt and sand size grains with small amounts of clay. The collapse potential of a soil depends on the in-situ density, depth of the deposit and the extent of a porous structure. When loading is applied to collapsible soils, originating from the weight of the structure, along with wetting, settlement occurs. Wetting sources are most commonly associated with landscape irrigation, inadequate surface drainage, utility line leakage, proximity of retention basins and water features to a structure, and long-term ponding next to the structure. Based on laboratory seismic refraction survey test data, the soils encountered at the site are considered to have a low potential for collapse and excessive differential soil movement.

5.0 RECOMMENDATIONS

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

5.1 Excavating Conditions

Excavations greater than 4.0 feet should be sloped or braced as required to provide personnel safety and satisfy local safety code regulations. The following table summarizes the seismic wave velocity and possible rippability conditions for the various layers. The rippability conditions are based on the seismic P-wave velocities and data utilized by Caterpillar Inc. included in their "Handbook of Ripping."

Table 4: Excavating Conditions

Layer	Depth Interval[1, 2]	Seismic Wave Velocity (feet per second)	Remarks Relative to Rippability
1	Above 2.3 feet	1092 to 2130	Conventional-Case 580 Trencher
2	Below 2.3 feet and above 7.9 feet	4287 to 5405	D10N, Caterpillar 235 with an appropriate sized hydraulic ram hoe attachment to accomplish effective material removal
3	Below 7.9	Above 6000	Blasting techniques may be required to accomplish effective removal[3]

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

[2]The depth to Layers 2 approaches zero moving towards visible outcrops.

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

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

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 Slopes Not Exceeding 20 Feet in Height

Portion of Cut Slope	Temporary Cut Slope Ratio (Horizontal to Vertical) "During the life of construction"	Permanent Cut Slope Ratio (Horizontal to Vertical)
Layer 1	2:1	2:1
Layer 2	1:2**	1:1.5* 1:2**
Layer 3	1:4.5*	1:2.5* 1:4.5**

Table 6: Cut Slopes Not Exceeding 30 Feet in Height

Portion of Cut Slope	Temporary Cut Slope Ratio (Horizontal to Vertical) "During the life of construction"	Permanent Cut Slope Ratio (Horizontal to Vertical)
Layer 1	2:1	2:1
Layer 2	1:1.5**	1:1* 1:1.5**
Layer 3	1:4*	1:2* 1:4**

*Maximum safe cut slope ratio (horizontal to vertical) without completion of a Markland stability analysis being performed on Layers 2 and 3 rock masses during the cutting operation (*i.e. A Markland stability analysis shall be required to verify the existence of fractured or foliated/parted rock and their respective orientations, and that if the conditions will pose a potential safety risk from unstable conditions*).

**Maximum safe cut slope ratio (horizontal to vertical) with the completion of a Markland stability analysis or other slope stability analysis during construction that will verify the integrity of the rock mass and ensure slope stability.

Thirty (30.0) feet is recommended as the maximum cut slope height, using the appropriate cut slope ratios for the corresponding height limitation.

Should the above presented cut slope recommendations not work with the site's geometry, a series of retaining walls would need to be designed and constructed, or stabilization of a steeper cut slope that is bolted.

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.

This firm should be notified during construction to verify field conditions and inspect all cut slopes for structural features (e.g. shear zones, foliation/parting, fractures, joint orientations and slabbing) contained within the rock mass that could lead to slope instability and eventual slope failure. If conditions relative to the integrity and stability of the rock mass are observed during the site excavation and are noted during a site inspection, this firm may alter the above-recommended cut slopes to adhere to a more stable condition.

Therefore, it is critical that all cut slope excavations be inspected at a point where; if unstable conditions are identified, that mitigation measures can be implemented before large scale cuts have been performed or slope failure occurs (i.e. inspecting and potentially modifying the cut slope recommendations, or possibly recommending the use of rock anchors, rock netting, or retaining walls for slope stability, when the cut is no greater than 10.0 feet in height). Note: Altered recommendations or mitigation measures shall be based on the results obtained from a Markland stability analysis, which is not part of the scope of work for this report.

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

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

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

Table 7: Horizontal Rock-Fall Impact Zone

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

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

Any cut into the natural slope will result in the removal of the toe for the up-slope portion, resulting in the potential movement of up-slope boulders riding on the surface. In addition to cut operations, vibrations from heavy equipment can induce a seismic-like component to a cut or natural slope which may reduce the overall slope stability and decrease the factor of safety against sliding to 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. Therefore, future slope movements should be anticipated 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.

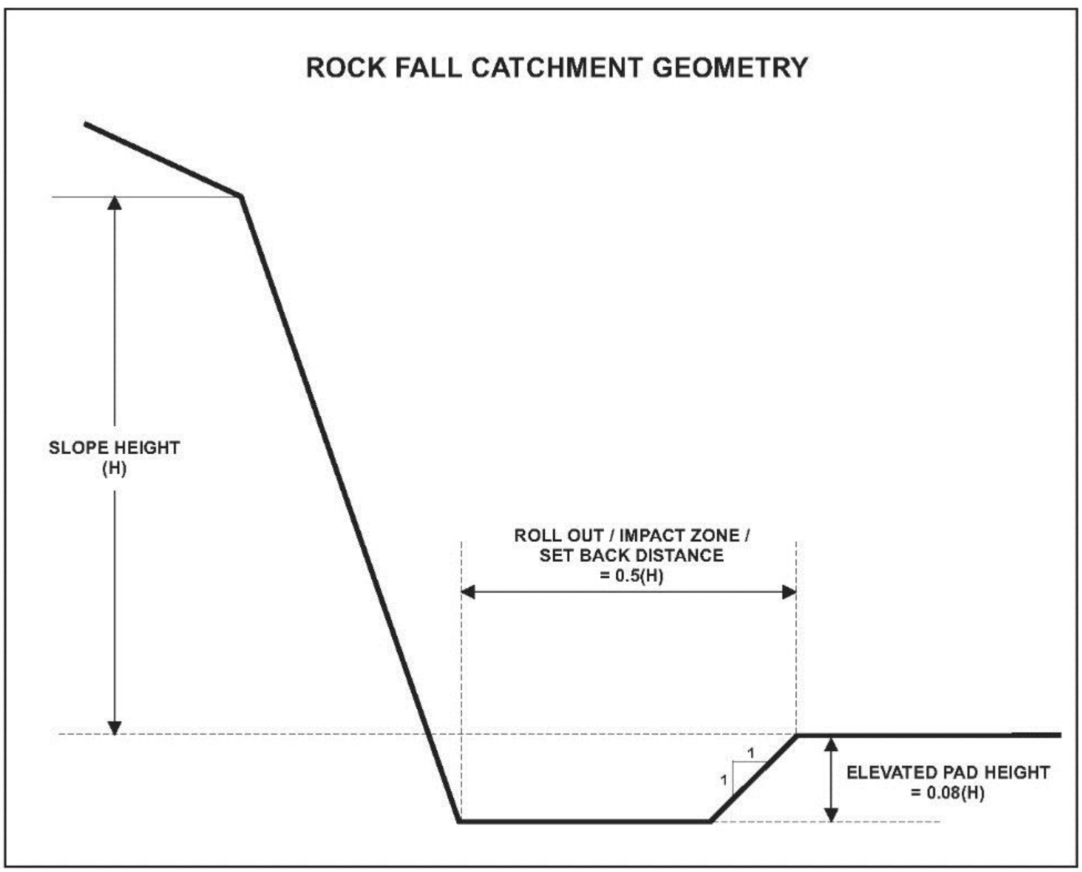


Figure 4: Rock Fall Catchment Geometry

5.3 Backfill Settlement

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

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

Flooding has also been experienced in below grade areas due to breakage of utility lines embedded in loose retaining/basement wall backfills, and from infiltration of surface water (irrigation and/or rainfall) through loose retaining/basement 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 8: Backfill Settlement

Backfill Types			Anticipated Settlement without any Structural Loads (in.)													
% Compaction	Description	% Estimated Strain	2.5 feet of Backfill	5 feet of Backfill	7.5 feet of Backfill	10 feet of Backfill	12.5 feet of Backfill	15 feet of Backfill	16 feet of Backfill	17.5 feet of Backfill	20 feet of Backfill	22.5 feet of Backfill	25 feet of Backfill	26 feet of Backfill	27.5 feet of Backfill	30 feet of Backfill
95-98	Very Well Compacted	0.5	0.15	0.3	0.45	0.6	0.75	0.9	1.0	1.05	1.2	1.35	1.5	1.6	1.65	1.8
95	Well Compacted	1	0.3	0.6	0.9	1.2	1.5	1.8	1.9	2.1	2.4	2.7	3.0	3.1	3.3	3.6
90	Moderately Compacted	2.5	0.75	1.5	2.25	3.0	3.75	4.5	4.8	5.25	6.0	6.75	7.5	7.8	8.25	9.0
85	Poorly Compacted	5	1.5	3.0	4.5	6.0	7.5	9.0	9.5	10.5	12.0	13.5	15.0	15.5	16.5	18.0
80	Very Poorly Compacted	7.5	2.25	4.5	6.75	9.0	11.25	13.5	14.3	15.75	18.0	20.25	22.5	23.3	24.75	27.0

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

5.4 Site Preparation

The following recommendations are presented as a guide in the compilation of construction specifications. The recommendations are not comprehensive contract documents and should not be utilized as such. Although underground facilities such as septic tanks, cesspools, basements, and dry wells were not encountered, such features may be encountered during construction. These features should be demolished or abandoned in accordance with the recommendations of the geotechnical engineer. Such measures may include backfill with 2-sack ABC/cement slurry.

It is recommended that all vegetation and deleterious materials be removed at the commencement of site grading activities. Following the removal of the above listed items, the uppermost 8.0 inches of the native soils must be reworked to establish a stable condition. All final compaction shall be as specified herein. The scarification and compaction requirement applies to cut situations as well as fill situations.

Any site cut material may be reused as structural supporting fill provided that it is free of all vegetation, deleterious matter, the <u>maximum particle size is 3 inches</u>, and a suitable percentage of fines will be generated to ensure a stable mixture.

Complete removal and cleaning of any undesirable materials and proper backfilling of 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 <u>wash areas</u>, trench areas, or sloped topography should involve <u>horizontal</u> layers placed in 6-inch lifts; such that each successive lift is benched into the native site soils a minimum lateral distance of <u>5.0 feet</u>.

Any tree removal efforts to accommodate the new structures 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).

It is the understanding of this firm that various utility trenches may traverse the completed pad. The backfill of all utility trenches, if not in conformance with this report, may adversely impact the integrity of the completed pad. 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 imported 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 9: Compaction Requirements

Material	Building Area	Percent Compaction (ASTM D698)	Compaction Moisture Content Range
On-site soils used as subbase fill or backfill for structural support with PI < 12	Below Foundation Level	95 min	Optimum -2 to optimum +2
	Above Foundation Level[1]	95 min	Optimum -2 to optimum +2
Imported Subbase fill or backfill for structural support	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.

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 10: Imported Fill Requirements

Soil Parameter	Requirement
Plasticity Index:	14 (Maximum)
Particle Size:	3.0 inches (Maximum)
Passing 3-inch Sieve	70-100 %
Acceptable Passing #4 Sieve	50-100 %
Passing #200 Sieve:	60 percent (Maximum)
Expansion Potential*:	1.5 % (Maximum)
Sulfates:	0.19 % (Maximum)

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

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 2:1 (horizontal: vertical) ratio if the 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 on-site soils are as follows:

Table 11: Shrinkage

Material	Estimated Shrinkage (Based on ASTM D698A)
Native Undisturbed	13% ± 3

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

5.7 Site Classification

This project is not located over any known active faults or fault associated disturbed zones. A 2012 IBC Site Classification of **A** may be utilized in the earthquake design of the proposed structure.

5.8 Surface Level Spread Foundations

It is recommended that all perimeter foundations and isolated exterior foundations bearing on native undisturbed soil (Layer 1) or engineered fill 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 native undisturbed soil (Layer 1) or engineered fill should be founded a minimum of 1.5 feet below finish floor level. Shallower foundations, i.e. 1.0 feet thick may be possible where footings shall bear on or into Layers 2 and 3.

In order to minimize the adverse effects of differential settlement, it is recommended that all of the foundations bear on the same or very similar stratum, i.e. situations should be avoided where a portion of the footings are bearing on Layer 2 or stronger and a portion is bearing on engineered fill or Layer 1. If the downslope footings for the structure are to bear on engineered fill or Layer 1, it is recommended to place some degree of recompacted soil beneath the upslope footings as well, i.e. minimum of 6.0 inches, such that the resultant bearing condition is somewhat equivalent across the site. A Transition Lot Overexcavation diagram has been provided in Section IV to aid in construction efforts associated with the aforementioned.

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

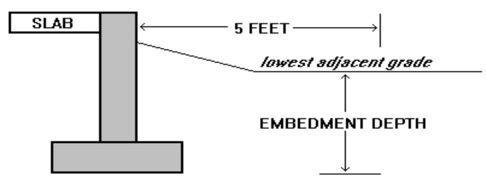
Table 12: Conventional Surface-Level Foundations

Foundation Embedment Depth[1]	Bearing Stratum[2]	Allowable Soil Bearing Capacity[3]
1.5 Feet	Native undisturbed soil (Layer 1) or engineered fill	2000 PSF

[1]Conditions for foundation embedment depth:

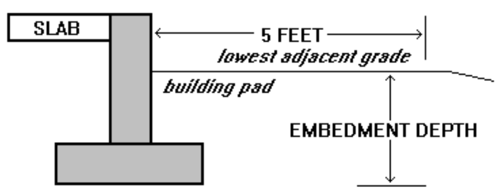
a) The depth below the lowest adjacent exterior pad grade within 5.0 feet of proposed exterior walls;

Condition A



b) The depth below finish 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 particular 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.

Table 13: Surface-Level Foundations Bearing on or into Layer 2 and 3

Foundation Embedment Depth (ft) - as defined herein	Depth of Occurrence Below Existing Grade	Bearing Layer	Allowable Soil Bearing Capacity
Bearing at the surface of Layer 2, with a minimum footing thickness of 1.0 feet	Below 2.3 feet and above 7.9 feet	Layer 2	4000 PSF

Foundation Embedment Depth (ft) - as defined herein	Depth of Occurrence Below Existing Grade	Bearing Layer	Allowable Soil Bearing Capacity
Bearing at the surface of Layer 3, with a minimum footing thickness of 1.0 feet	Above 7.9 feet	Layer 3	9000 PSF
Socketed 1.0 feet into Layer 3	Above 7.9 feet	Layer 3	10000 PSF (Limiting condition)

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 design live loads. The allowable bearing may be increased by a factor of 1.33 for resistance to wind loads and/or temporary eccentric loading.

Building foundations to be constructed in close proximity to retention basins (within 5 feet) should be embedded 1.0 feet deeper than the stated depths in the preceding bearing capacity tables.

Shallow foundations that are adjacent to lower foundation areas must be stepped down so that their base is below the lower backfill materials, and below a line projected upward from the nearest lower foundation edge at a 45-degree angle.

It is recommended that continuous footings and stem walls are reinforced and bearing walls be constructed with frequent joints to better distribute stresses in the event of localized settlements. Similarly, all masonry walls should be provided with both vertical and horizontal reinforcement. It is recommended that the footing excavations be inspected to ensure that they are free of loose soil which may have blown or sloughed into the excavations. It will also be necessary for the geotechnical engineer to verify that the footing embedment depths and bearing stratum adhere to the recommendations presented above.

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 Level 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 Level 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.

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 14: Lateral Stability

Parameter	Wall Type	Native Undisturbed Soil (Layer 1) and Engineered Fill	Layer 2[c]	Layer 3[c]
Active (K_a) Pressure[a]	Free-end	34 psf/ft		
At-Rest (K_o) Pressure[a]	Fixed-end[b]	52 psf/ft		
Passive (K_p) Resistance	Free-end/Fixed-end independent of base friction	358 psf/ft	593 psf/ft	888 psf/ft
	Fixed-end in conjunction with base friction	240 psf/ft	398 psf/ft	595 psf/ft
Coefficient of Base Friction (μ)	Free-end/Fixed-end independent of passive resistance	0.62	0.81	0.97
	Free or Fixed-end in conjunction with passive resistance	0.42	0.52	0.65

[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

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

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 with a typical thickness of 4.0 inches. The aggregate base material should conform to the requirements of local practice.

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

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

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

5.11 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 2 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 5 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 15: 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 depth by 6.0 inches[2]
3 feet[1]	7 feet	Increase footing depth by 12.0 inches[2]
2 feet[1]	7 feet	Increase footing 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 well 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.

5.13 Foundations and Risks

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

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

6.0 ADDITIONAL SERVICES

As an additional service, this firm would be pleased to review the project plans and structural notes for conformance to the intent of this report. Vann Engineering, Inc. should be retained to provide documentation that the recommendations set forth are met. These include, but are not limited to documentation of site clearing activities, verification of fill suitability and compaction, and inspection of footing excavations. Relative to field density testing, a minimum of 1 field density test should be taken for every 2500 square feet of building area, per 6-inch layer of compacted fill.

This firm possesses the capability of performing testing and inspection services during the course of construction. Such services include, but are not limited to, compaction testing as related to fill control, foundation inspections and concrete sampling. Please notify this firm if a proposal for these services is desired.

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

A. Proof rolling and fill subgrade conditions
B. Suitability of borrow materials
C. Fill control for building pads (verification of overexcavation depths and lateral extents, compaction testing, and the general monitoring of fill placement)
D. Foundation observations (compliance with the General Structural Notes, depths, bearing strata, etc.)
E. Basement, structural or retaining wall backfill testing
F. Backfilling and compaction of excavations (e.g. Utility trench backfill)
G. Special inspections as dictated by the local municipality

H. Concrete sampling and testing for footings, stem walls and floor slabs
I. Subgrade testing for proposed pavement areas
J. ABC testing for proposed pavement areas
K. Asphaltic concrete testing for proposed pavement areas
L. Subgrade preparation for on-site sidewalk areas
M. Grout sampling and testing, where applicable
N. Mortar sampling and testing, where applicable
O. Compliance with the geotechnical recommendations

7.0 LIMITATIONS

This report is not intended as a bidding document, and any contractor reviewing this report must draw their own conclusions regarding specific construction techniques to be used on this project. The scope of services carried out by this firm does not include an evaluation pertaining to environmental issues. If these services are required by the lender, we would be most pleased to discuss the varying degrees of environmental site assessments.

This report is issued with the understanding that it is the responsibility of the owner to see that its provisions are carried out or brought to the attention of those concerned. In the event that any changes to the proposed project are planned, the conclusions and recommendations contained in this report shall be reviewed and the report shall be modified or supplemented as necessary. Prior to construction, we recommend the following:

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

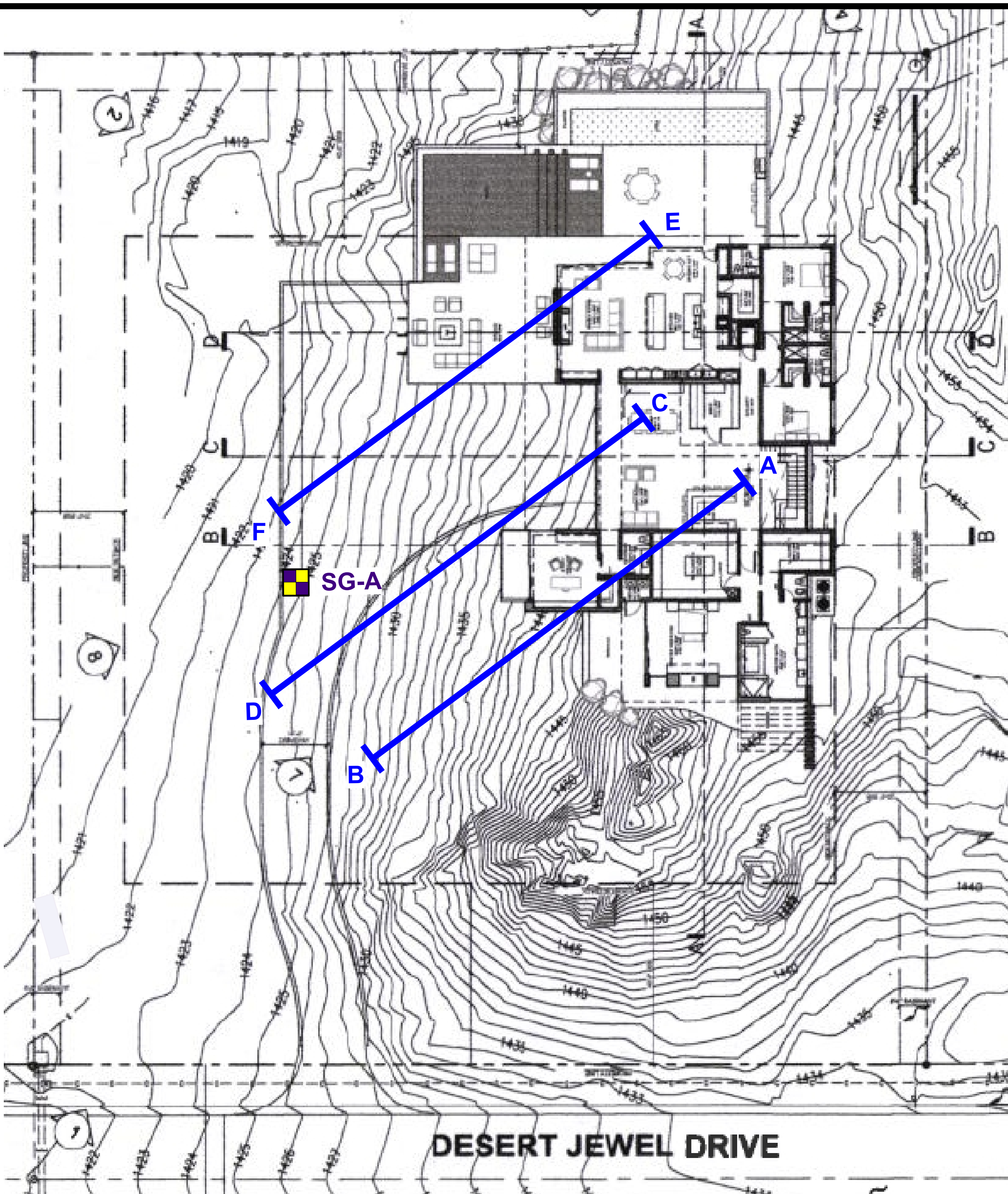
Relative to this firm's involvement with the project during the course of construction, we offer the following recommendations:

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

DEFINITION OF TERMINOLOGY

Allowable Soil Bearing Capacity Allowable Foundation Pressure	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
Aggregate Base Course (ABC)	A sand and gravel mixture of specified gradation, used for slab and pavement support.
Backfill	A specified material placed and compacted in a confined area.
Base Course	A layer of specified material placed on a subgrade or subbase.
Base Course Grade	Top of base course.
Bench	A horizontal surface in a sloped deposit.
Caisson	A concrete foundation element cased in a circular excavation, which may have an enlarged base. Sometimes referred to as a cast-in-place pier.
Concrete Slabs-on-Grade	A concrete surface layer cast directly upon a base, subbase, or subgrade.
Controlled Compacted Fill	Engineered Fill. Specific material placed and compacted to specified density and/or moisture conditions under observation of a representative of a soil engineer.
Differential Settlement	Unequal settlement between or within foundation elements of a structure.
Existing Fill	Materials deposited through the action of man prior to exploration of the site.
Expansive Potential	The potential of a soil to increase in volume due to the absorption of moisture.
Fill	Materials deposited by the action of man.
Finish Grade	The final grade created as a part of the project.
Heave	Upward movement due to expansion or frost action.
Native Grade	The naturally occurring ground surface.
Native Soil	Naturally occurring on-site soil.
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



LEGEND:

SEISMIC TEST LOCATIONS

SUBGRADE SAMPLE LOCATION

N

SCALE: N.T.S.	SITE PLAN	PREPARED BY: MS
DATE:11/20/2017		

PROPOSED SCHERR RESIDENCE
APN 168-75-029
5416 EAST DESERT JEWEL DRIVE
PARADISE VALLEY, ARIZONA

PROJECT 25548

VANN ENGINEERING INC

PROJECT 25548

Average Velocity of Layer 1 = 1722 fps (Range 1092 to 2130)

Average Velocity of Layer 2 = 4832 fps (Range 4287 to 5405)

Average Depth to Layer 2 = 2.3 feet

Range: 0.8 ft to 4.9 ft

Average Velocity of Layer 3 = 9716 fps (Range 9279 to 10231)

Average Depth to Layer 3 = 7.9 feet

Range: 4.6 ft to 12.4 ft

LAYER 1: GRAVELLY SILTY CLAYEY SAND (SC-SM)

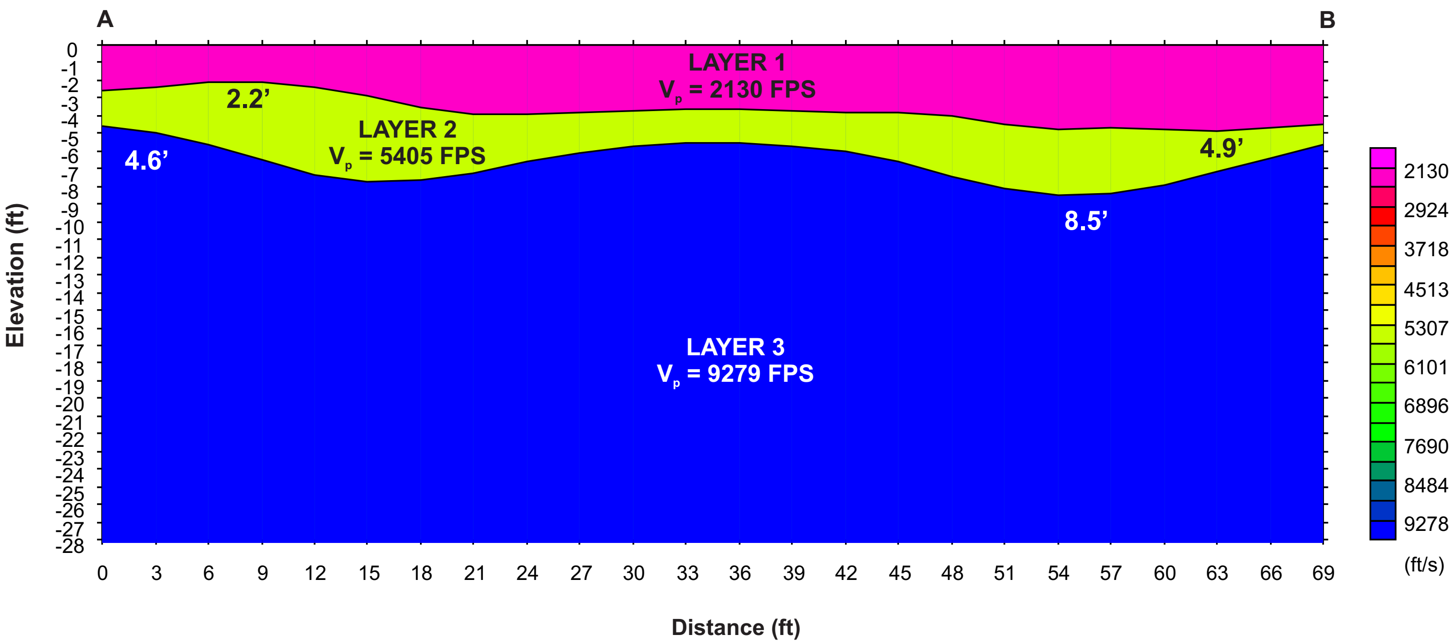
LAYER 2: HIGHLY TO MODERATELY WEATHERED AND FRACTURED, POOR, WEAK QUARTZ-MUSCOVITE SCHIST

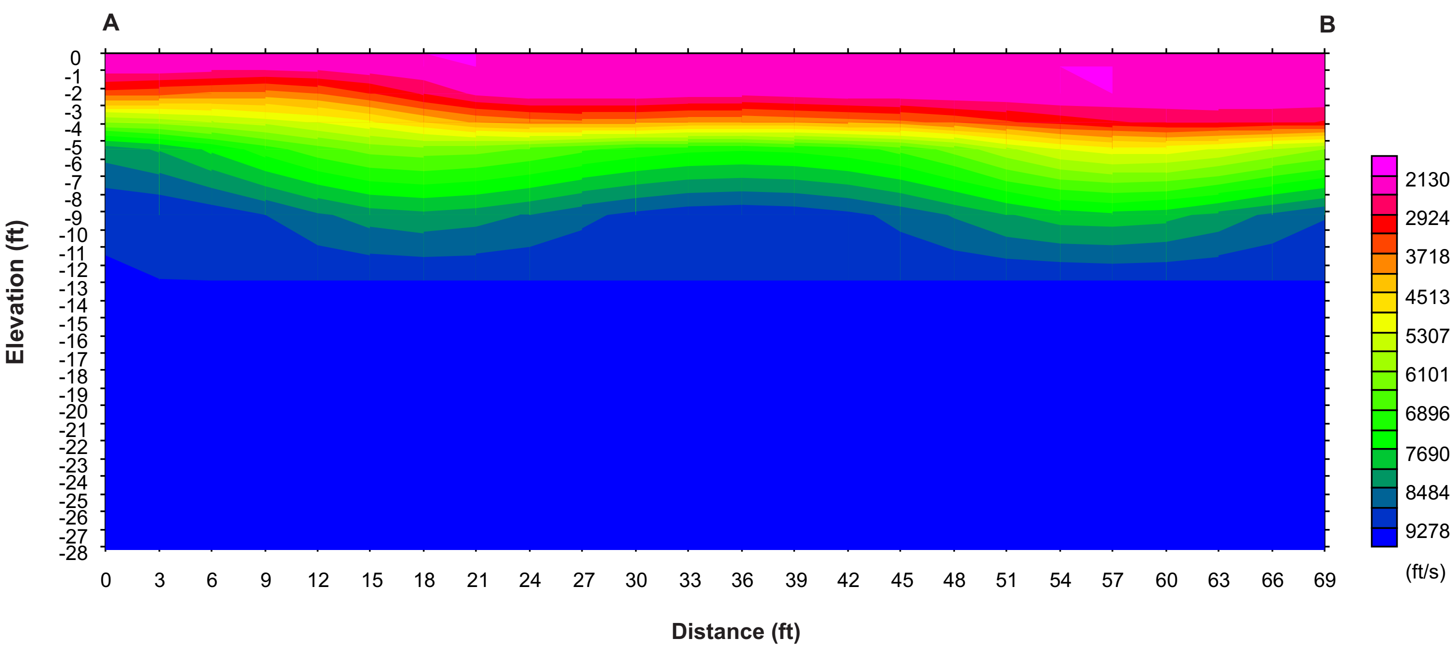
LAYER 3: SLIGHTLY WEATHERED AND FRACTURED, VERY GOOD, VERY STRONG QUARTZ-MUSCOVITE SCHIST

Line	Layer 1		Layer 2			Layer 3		
	Velocity (F)	Depth	Velocity (F)	Depth		Velocity (F)	Depth	
A-B	2130	-	5405	2.2	4.9	9279	4.6	8.5
C-D	1943	-	4805	0.8	2.7	9639	6.3	12.4
E-F	1092	-	4287	1.5	1.8	10231	6.6	9.0
		-						
		-						
Averages	1722	-	4832	2.3		9716	7.9	

CROSS SECTION

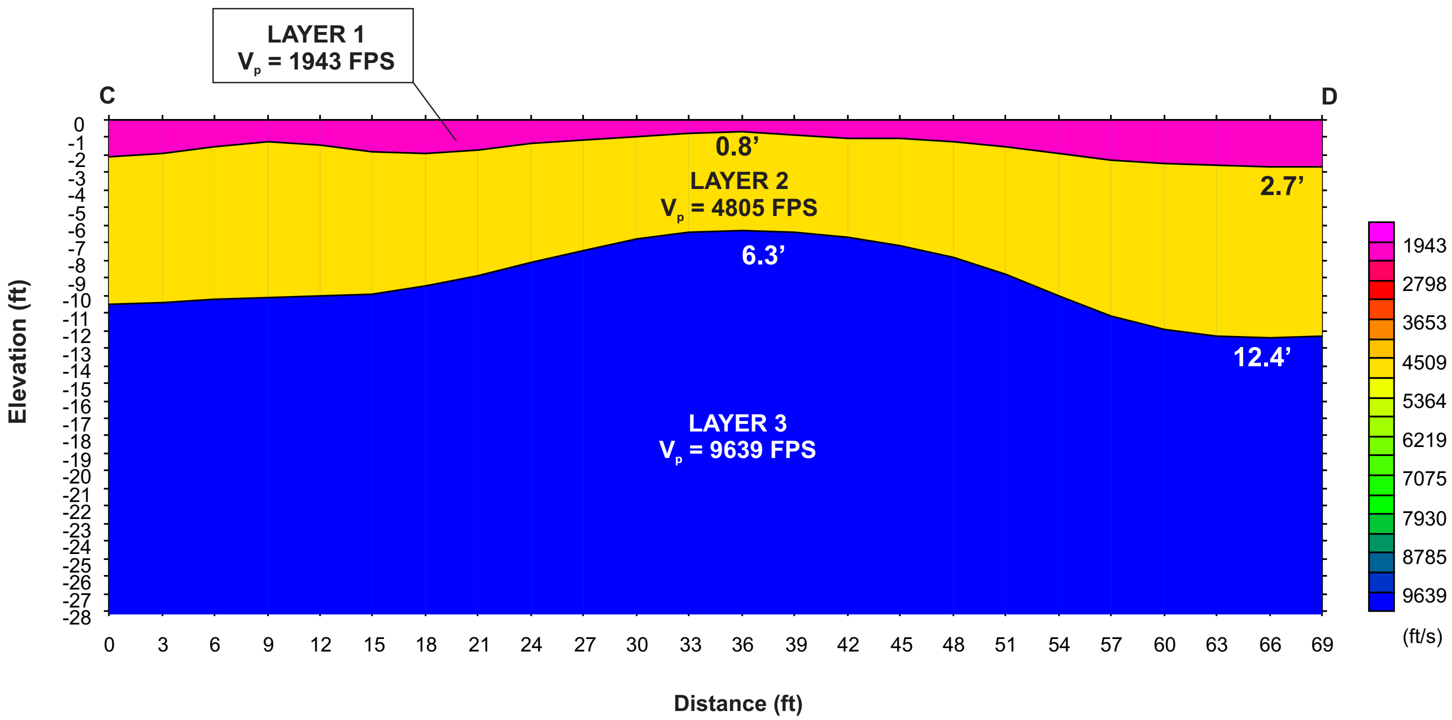
SEISMIC SURVEY A-B

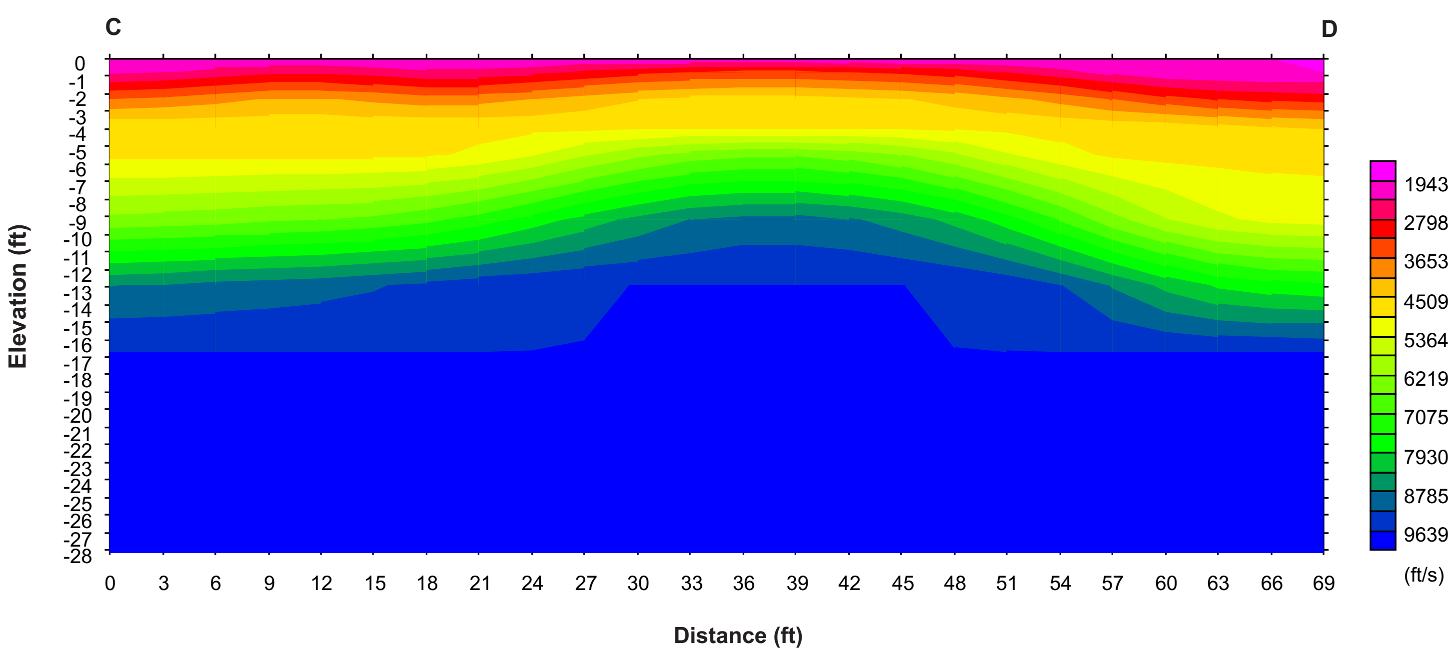




CROSS SECTION

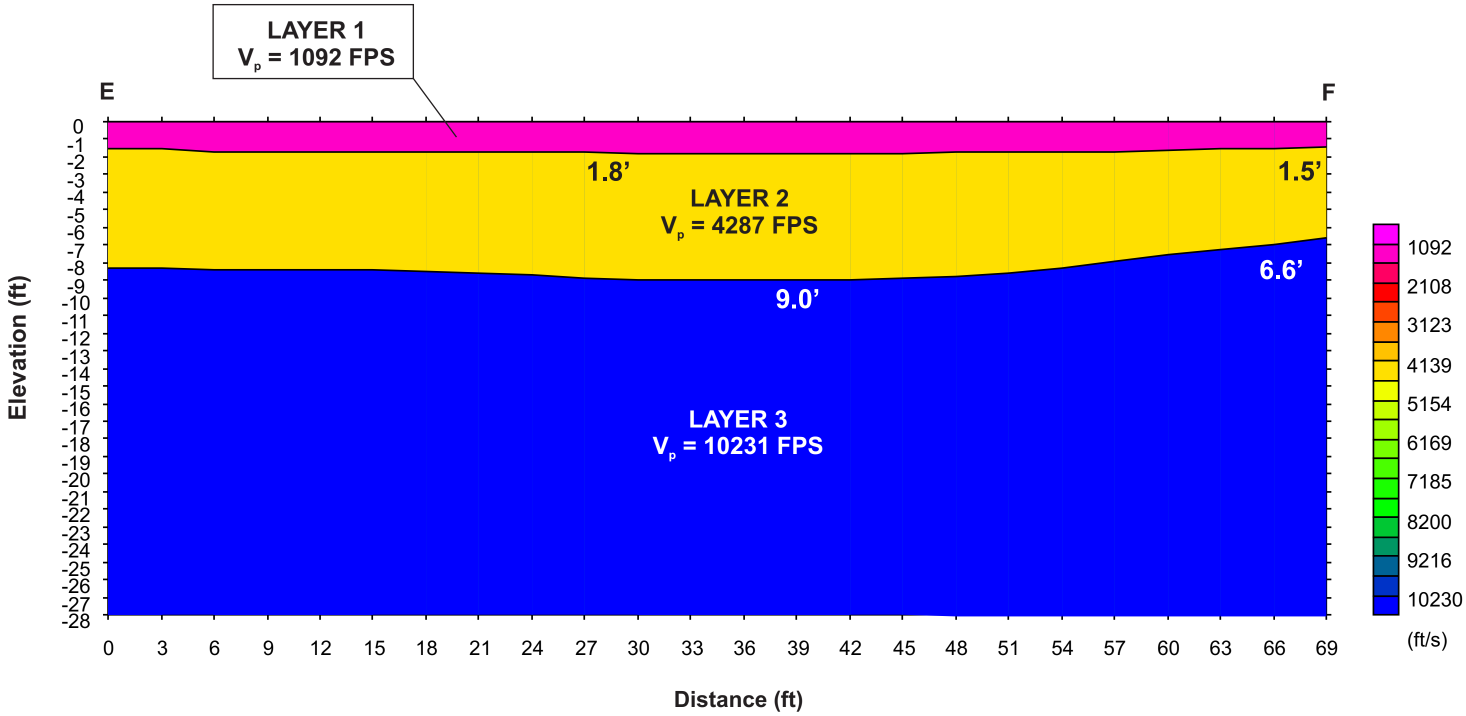
SEISMIC SURVEY C-D

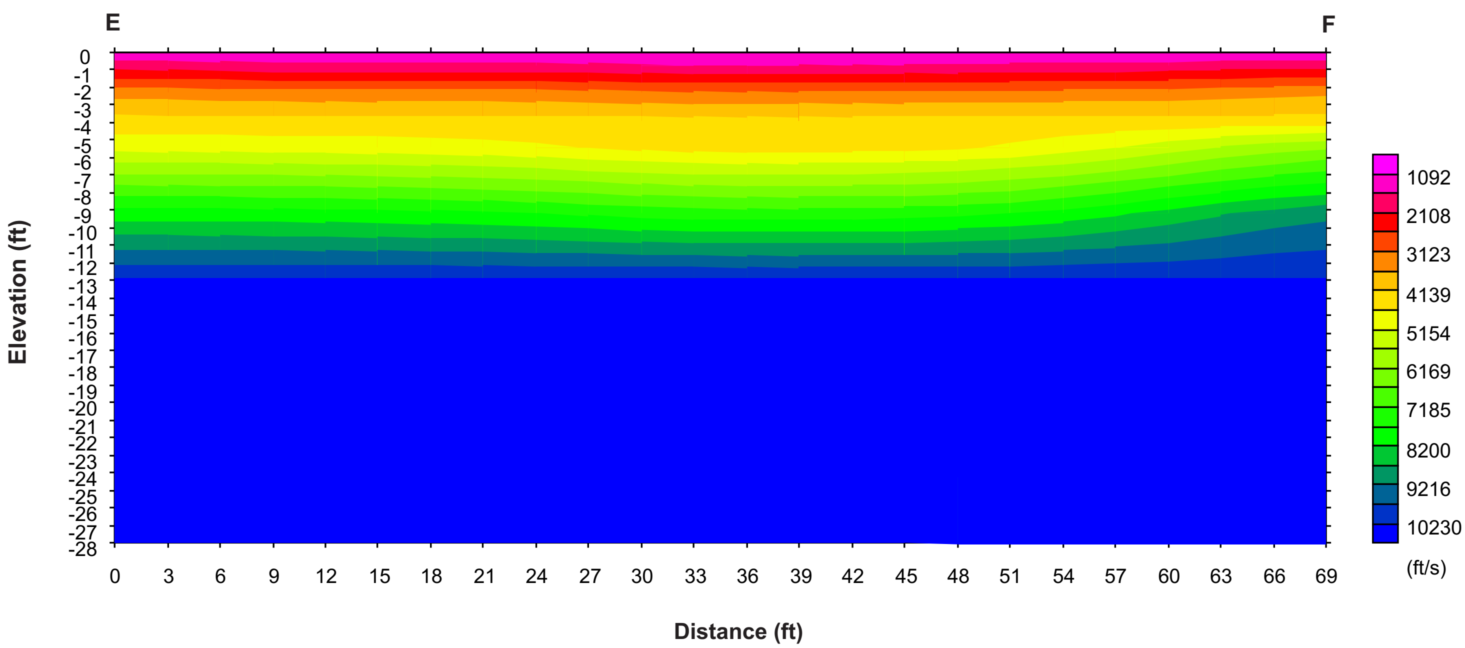




CROSS SECTION

SEISMIC SURVEY E-F

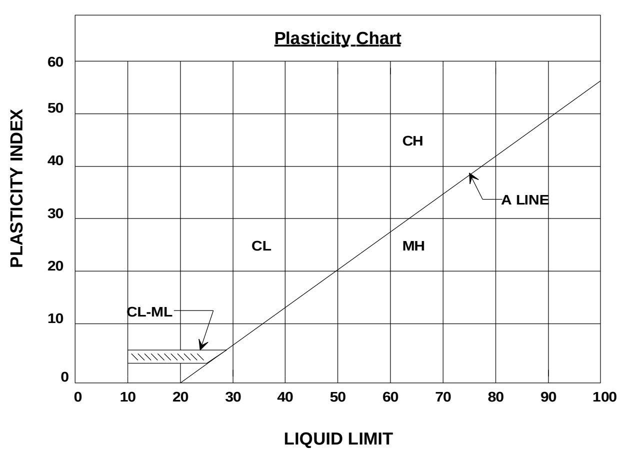




LEGEND

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

CLASSIFICATION TEST DATA

PROPOSED SCHERR RESIDENCE
APN 168-75-029
5416 EAST DESERT JEWEL DRIVE
PARADISE VALLEY, ARIZONA 85253

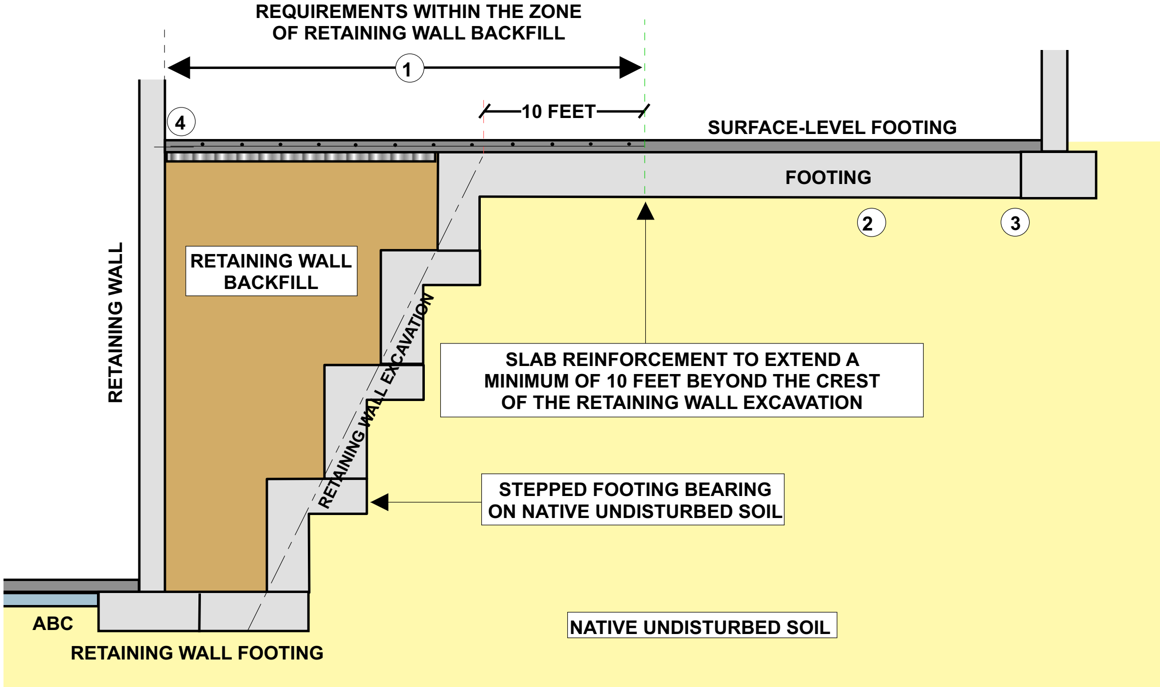
Sample Location	***Sieve Analysis (% Passing Sieve Size)***								***Atterberg Limits***			***Moisture Content %***
	3”	***2”***	***1”***	***#4***	***#10***	***#40***	***#100***	***#200***	***LL***	***PI***	***USCS***	
SG-A (1.0’-2.0’)	-	100	96	75	66	53	-	31	24	5	SC-SM	1.0

SECTION IV

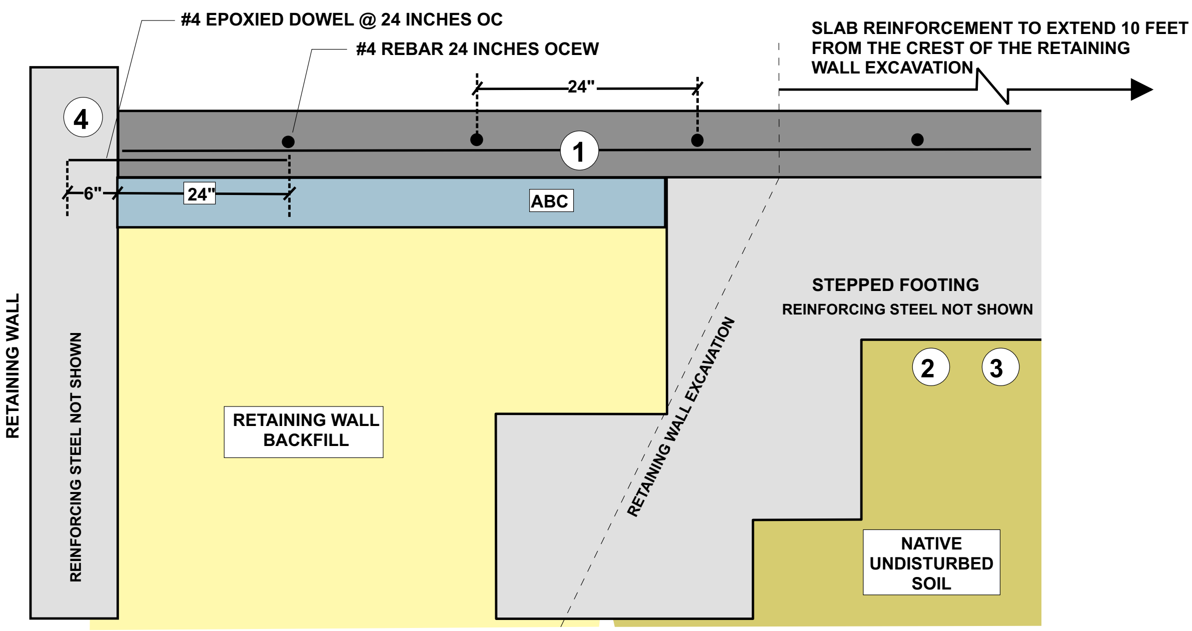
SURFACE TO RETAINING WALL FOOTING TRANSITIONS

OPTION A: (CROSS SECTION)

SURFACE-LEVEL FOOTINGS
BEARING ON NATIVE UNDISTURBED SOIL
STEPPED TO MEET RETAINING WALL FOOTINGS

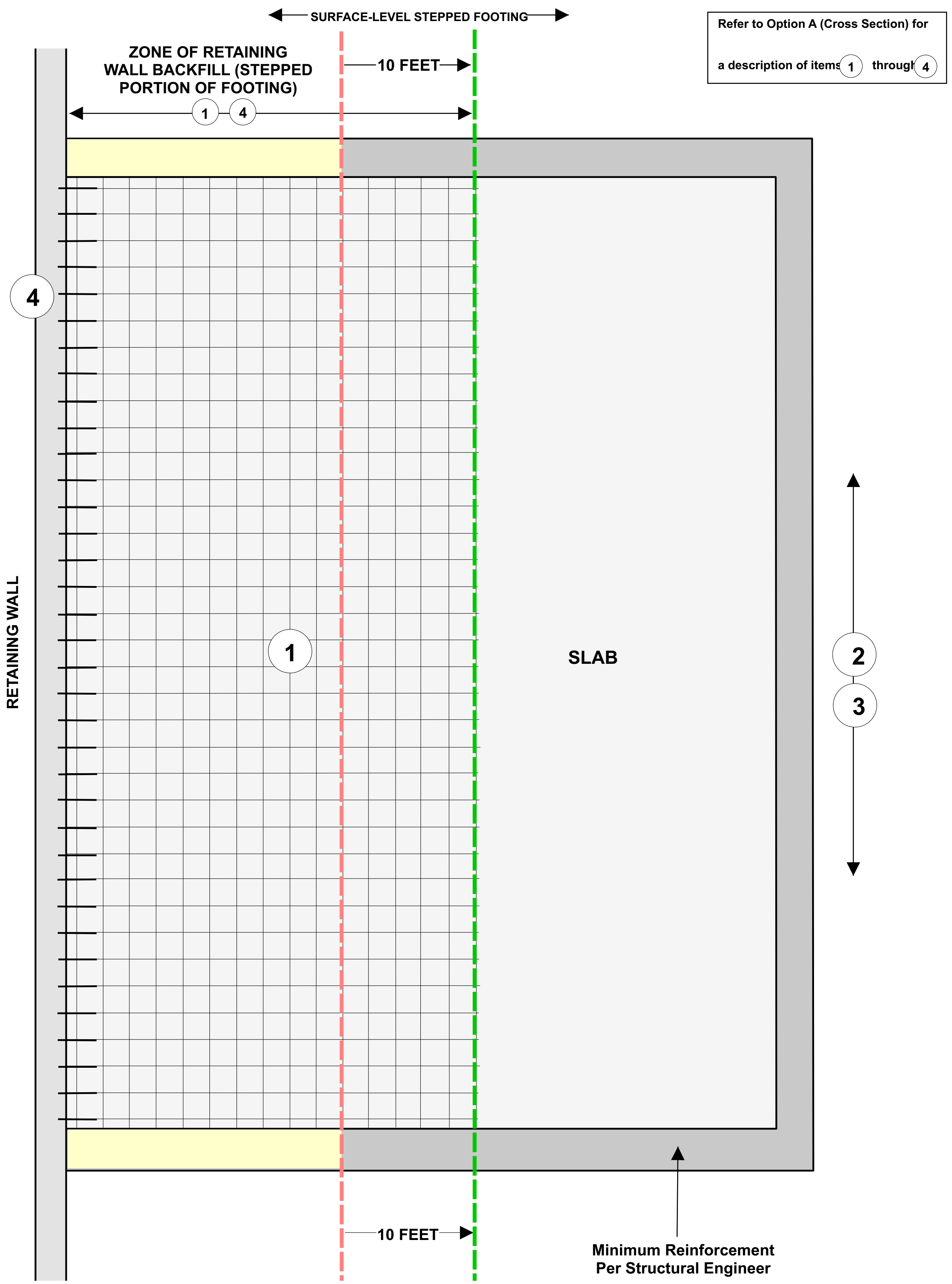


1. REINFORCE SLAB WITH #4 REBAR @ 24 INCHES OCEW, CHAIRED, 100 PERCENT TIED, AND CONNECTED TO THE FOOTING STEEL
2. REFER TO EARTHWORK SECTION FOR REQUIRED ZONE OF SCARIFICATION BENEATH SLABS, SIDEWALKS, PARKING AREAS, ETC.
3. REFER TO SURFACE-LEVEL FOUNDATION TABLES FOR MINIMUM FOOTING DEPTHS AND ASSOCIATED 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 WALL (LAP AND TIE 24 INCHES TO THE SLAB STEEL)



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

OPTION A: (PLAN VIEW)



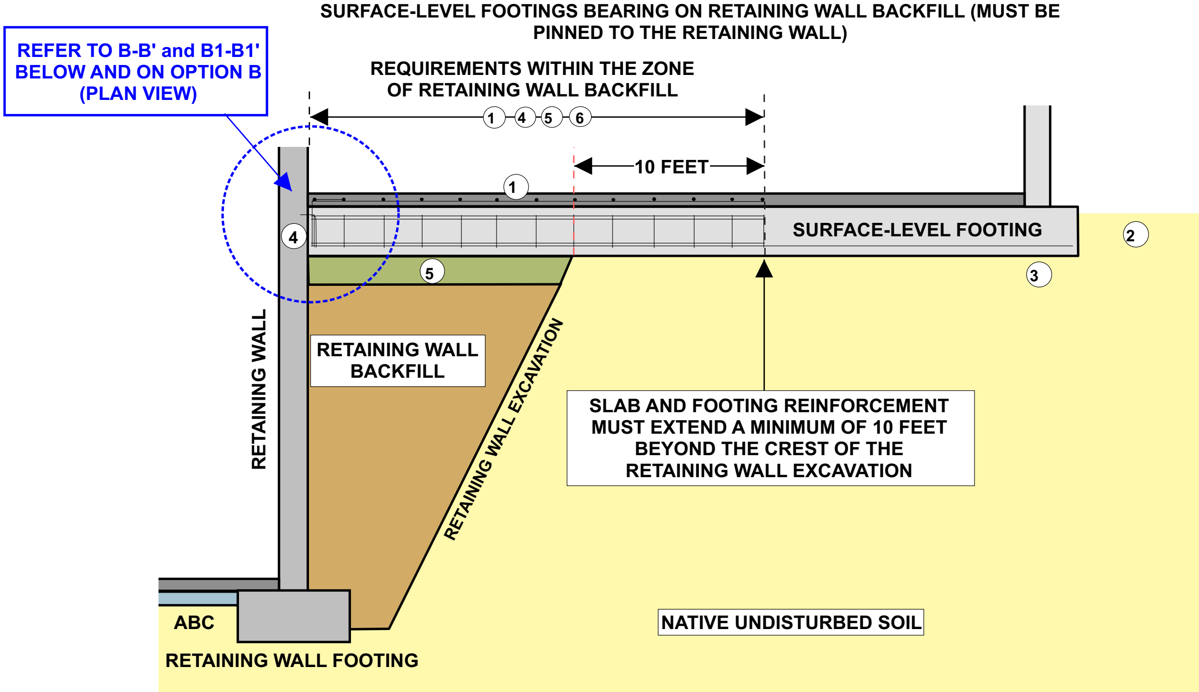
EXPLANATION

LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

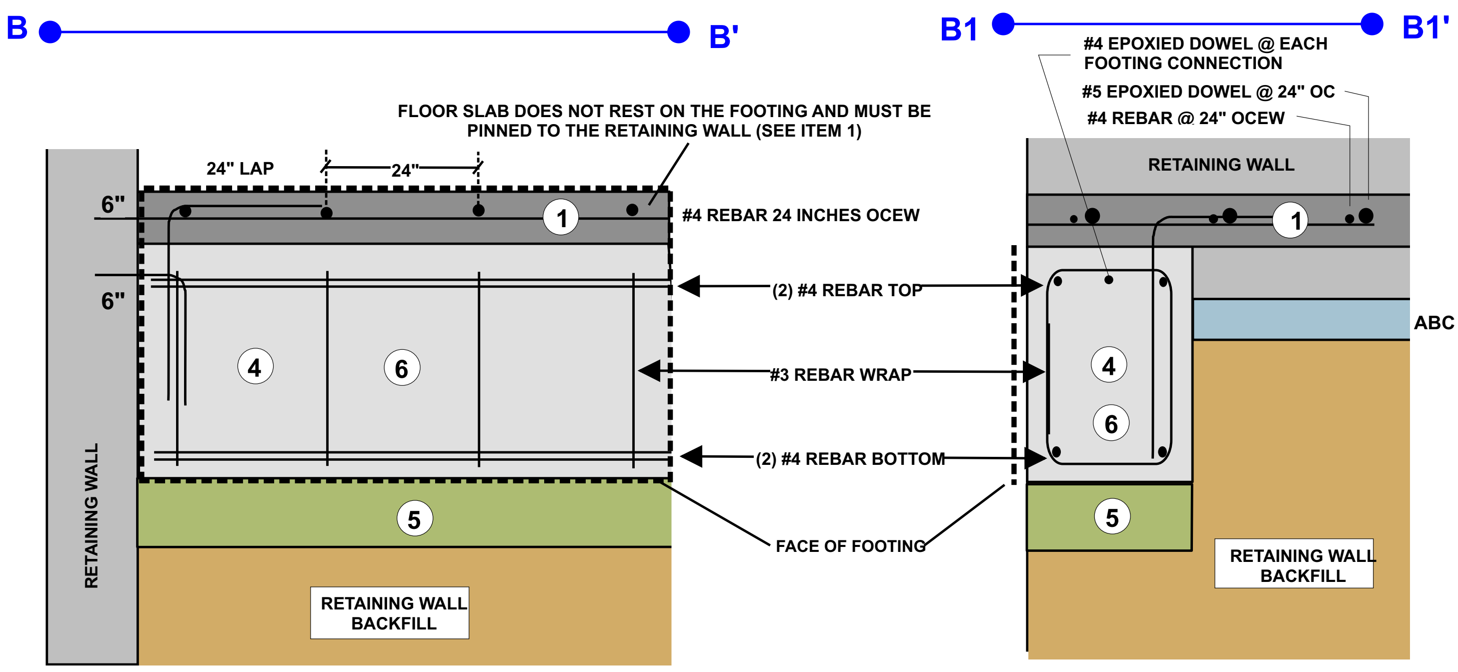
LINE DENOTING A POINT 10 FEET BEYOND THE CREST OF THE EXCAVATION CUT SLOPE

OPTION B: (CROSS SECTION)

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

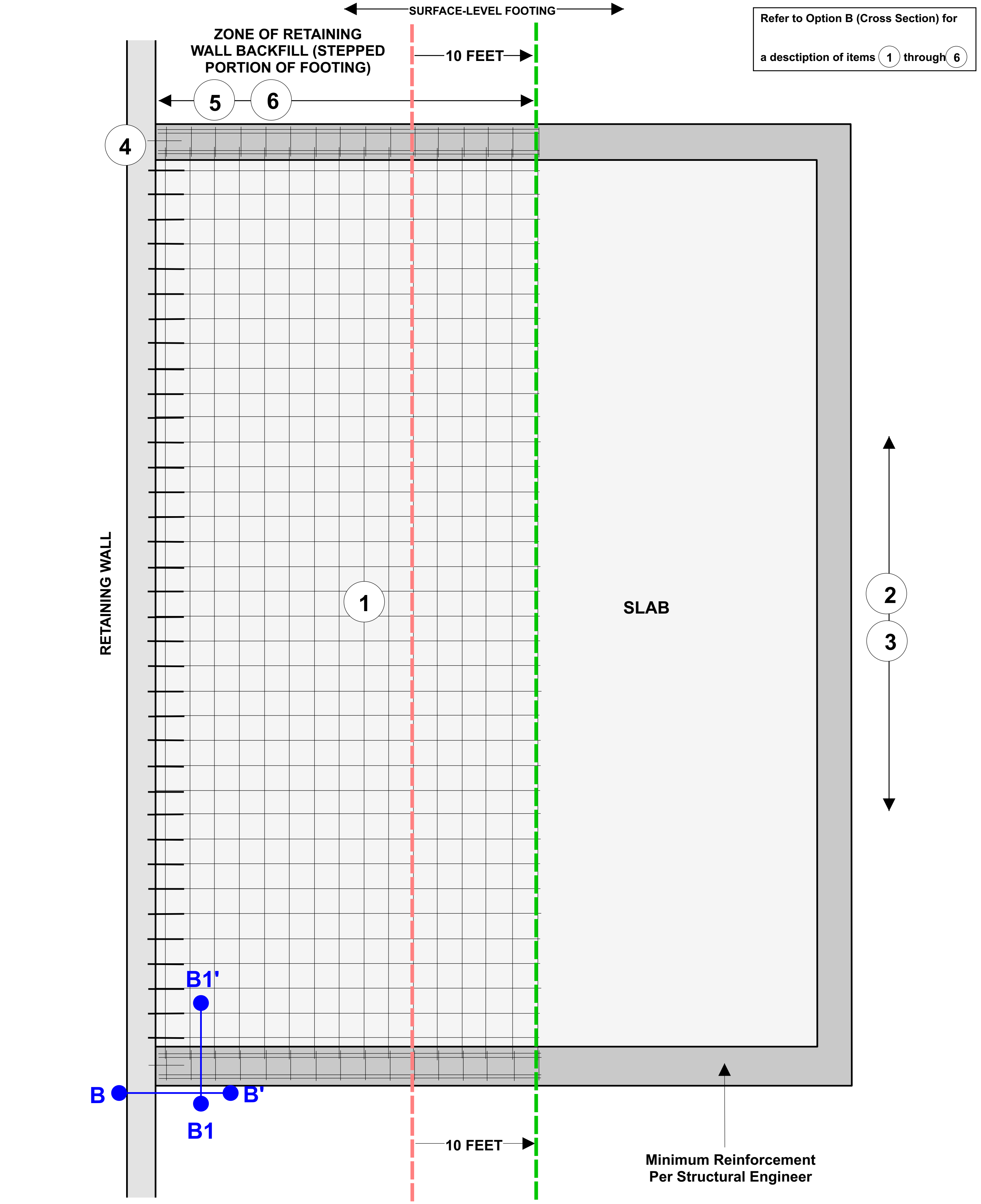


1. REINFORCE SLAB WITH #4 REBAR @ 24 INCHES OCEW, CHAIRED, 100 PERCENT TIED, AND CONNECTED TO THE FOOTING STEEL. FLOOR SLAB MUST BE TIED TO THE BASEMENT/RETAINING WALL WITH #5 EPOXIED DOWELS @ 24 INCHES OC
2. REFER TO EARTHWORK SECTION FOR REQUIRED ZONE OF SCARIFICATION BENEATH SLABS, SIDEWALKS, PARKING 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 WALL BACKFILL, TO A MINIMUM OF 95% OF THE MAXIMIM 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 STRUCTUAL ENGINEER
- ILLUSTRATIONS NOT TO SCALE
- REFER TO OPTION B (PLAN VIEW)

OPTION B: (PLAN VIEW)



EXPLANATION

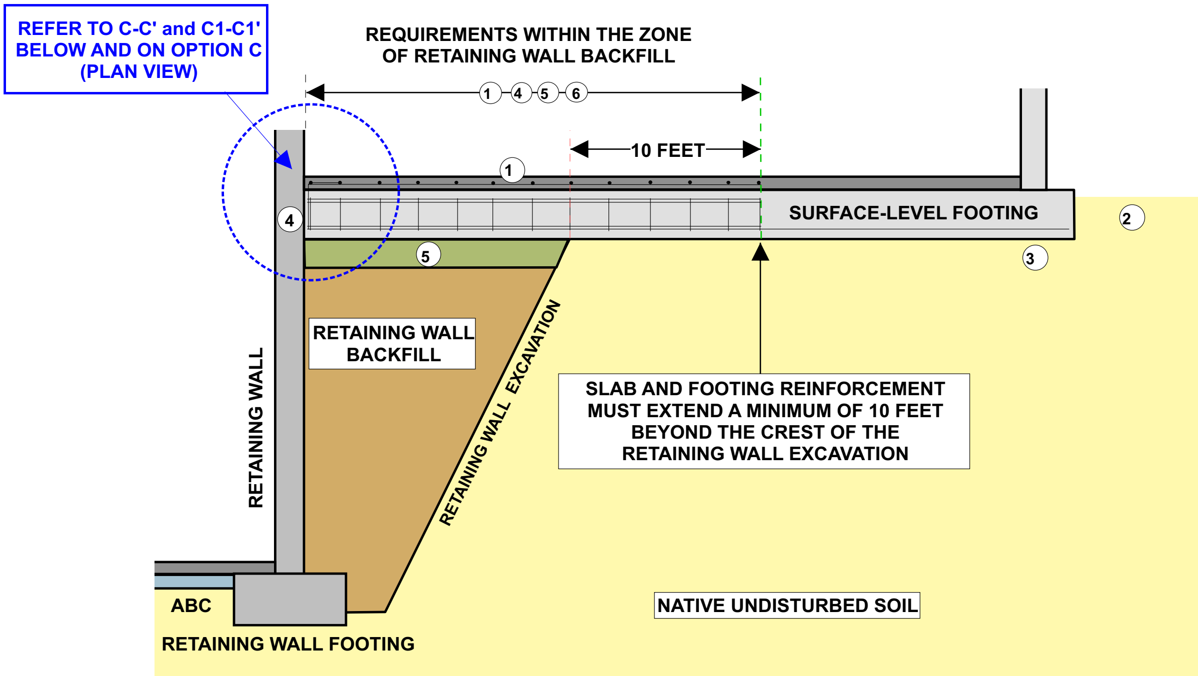
LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

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

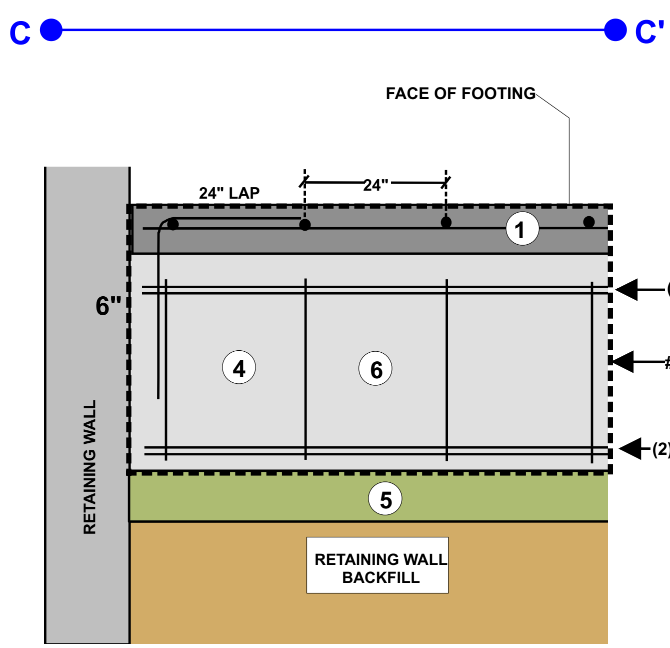
CROSS SECTIONAL VIEW - REFER TO OPTION B (CROSS SECTION)

OPTION C: (CROSS SECTION)

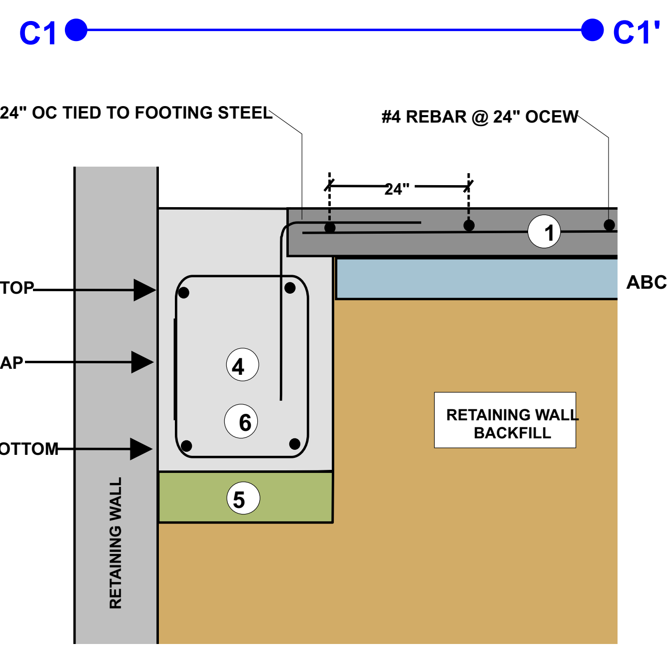
SURFACE-LEVEL FOOTINGS BEARING ON RETAINING WALL BACKFILL



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

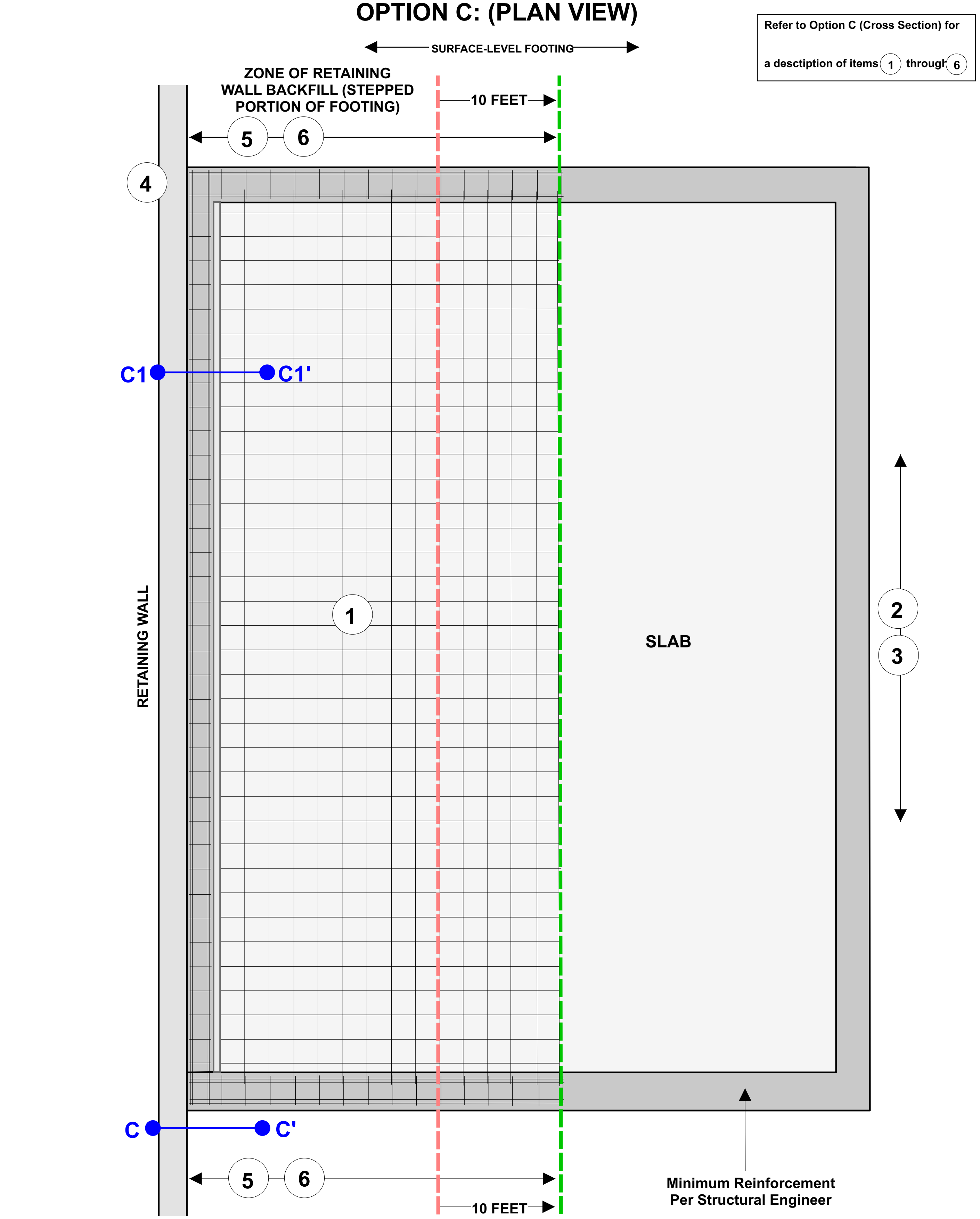


FLOOR SLAB RESTING ON THE FOOTING AND DOES NOT REQUIRE PINNING TO THE RETAINING WALL



- 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)

OPTION C: (PLAN VIEW)



EXPLANATION

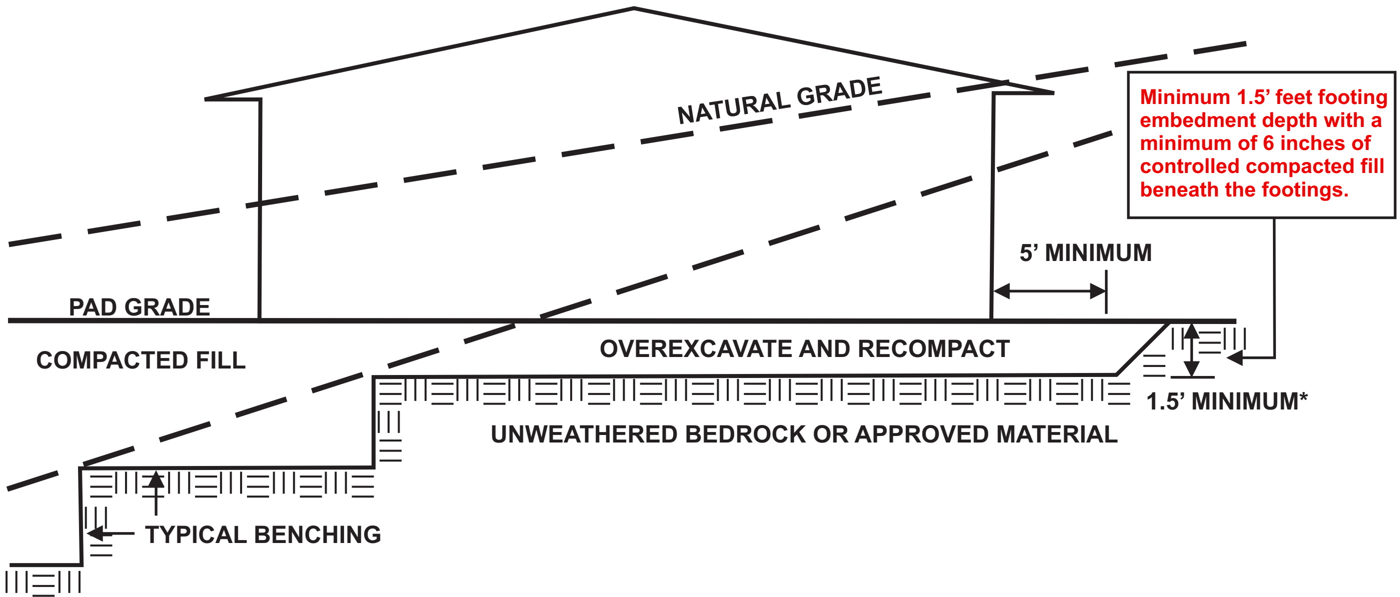
LINE DENOTING THE CREST OF THE EXCAVATION CUT SLOPE

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

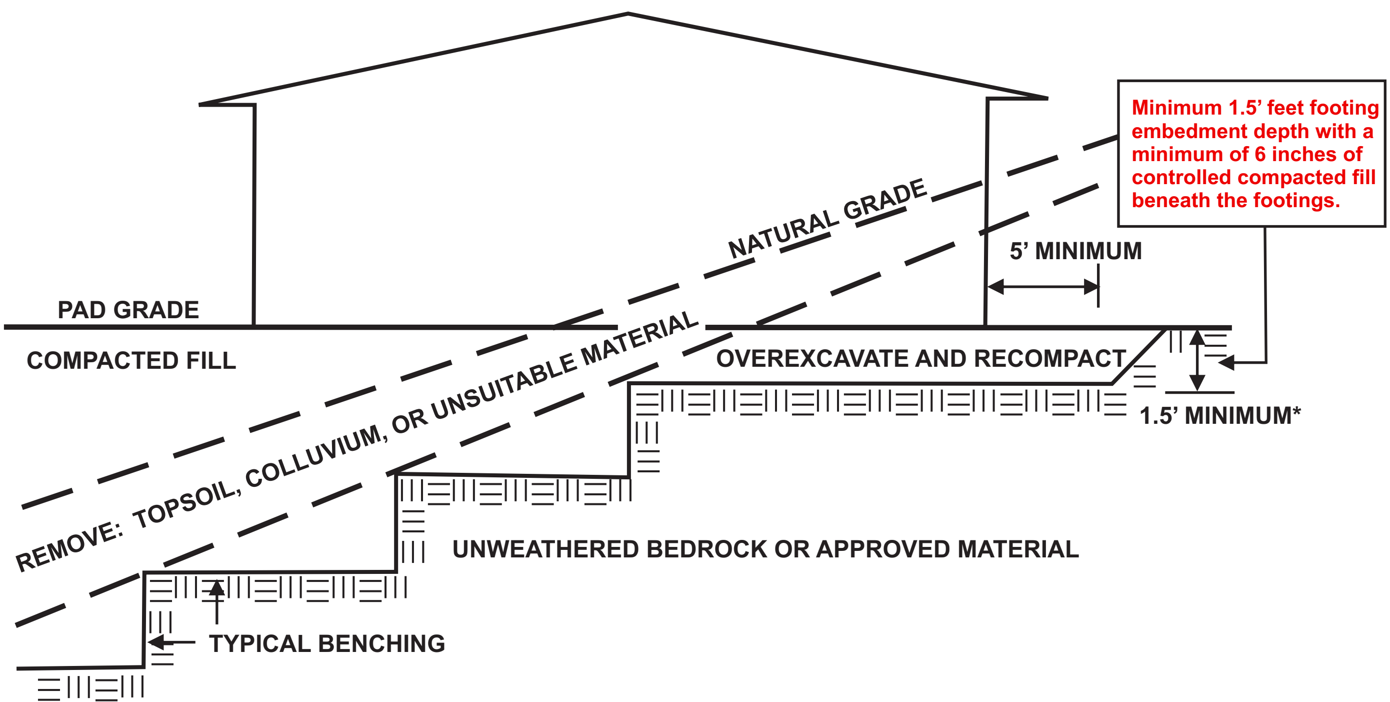
CROSS SECTIONAL VIEW - REFER TO OPTION C (CROSS SECTION)

TRANSITION LOT DETAIL

CUT LOT (MATERIAL TYPE TRANSITION)



CUT-FILL LOT (DAYLIGHT TRANSITION)



NOTE: * DEEPER OVEREXCATION MAY BE RECOMMENDED BY THE SOILS ENGINEER AND/OR ENGINEERING GEOLOGIST IN STEEP CUT-FILL TRANSITION AREAS.

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