Appendix E

Geotechnical Site Evaluation Report for Cancer Center Site

Geotechnical Site Evaluation Report
HCA Medical Office Building
Southeast Corner of Rolling Oaks
and
Los Padres Drives
400 East Rolling Oaks Drive
APN 681-0-180-265

prepared for:

HCA
One Park Plaza, Bldg II-E
Nashville, TN 37203

GORIAN
& ASSOCIATES, INC.

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Attachments:

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April 24, 2020

HCA
One Park Plaza, Bldg II-E
Nashville, TN 37203

Work Order: 64-0-0-100

Attention: Mr. Zach Wideman
Design and Construction

Subject: **Geotechnical Site Evaluation Report, HCA Medical Office Building, Southeast Corner of Rolling Oaks and Los Padres Drives (400 East Rolling Oaks Drive, APN 681-0-180-265), Thousand Oaks, California.**

1. INTRODUCTION

The following report contains the results of our geotechnical site evaluation addressing design and construction of a medical building for HCA at 400 East Rolling Oaks Drive in the southeast corner of Rolling Oaks and Los Padres Drives in Thousand Oaks, California (see Figure 1). The project will consist of grading a building pad for construction of a two-story medical building totaling 59,000 gross square feet of space. Details regarding the project were obtained from an information packet by Perkins + Will with the project layout shown on Plate 1 based on a conceptual grading plan by Kimley Horn. Parking will be provided in a surface parking lot around the building.

The site was previously developed for a child day care center. Grading for this center created three relatively level pads of which the building was in the center pad. The new building will require regrading of the site to accommodate the larger footprint of the medical building. The required grading will encounter a relatively thin layer of soil over Miocene-age bedrock of the Conejo Volcanics.

Field exploration for the project consisted of eight borings that was supplemented with laboratory testing to determine mechanical properties of the earth units. Based on our site evaluation, the site is suitable for the proposed construction from a geotechnical standpoint provided recommendations presented herein are implemented in the project design and construction. Descriptions of the site and geologic units along with our conclusions and recommendations are presented within the text of this report.

Testing for onsite stormwater infiltration was performed during the field exploration phase of this site evaluation. However, the testing indicated onsite stormwater infiltration was not feasible for the project.

2. PROPOSED DEVELOPMENT

HCA is proposing construction of a 59,000 gross square foot two-story medical building on the site of a previous child care center. The building will be OSHPD 3 structure for outpatients only and therefore, under the jurisdiction of the city of Thousand Oaks. The building will be comprised of three seismically independent structures consisting of a two-story medical building, one-story linear accelerator, and entry canopy. The main building and entry are anticipated to be of metal framing, whereas the accelerator will be of reinforced concrete.

The building will be roughly centered within the site as shown on Plate 1. Due to the increase in grade (surface elevation) toward the south, the building will be stepped into the hillside terrain. Therefore, the southern parking lot will be roughly at the elevation of the second floor of the building. Whereas, the northern parking lot will be at roughly at the elevation of the first floor. This grade change will require retaining walls in the eastern and southern walls of the building as shown in cross sections A-A' and B-B' on Plate 2 along with cuts to create a level building pad. Street access to the southern parking will be from Los Padres Drive and access to the northern parking lot will be from Rolling Oaks Drive. The two parking areas will be connected by a gently sloped parking area long the east side of the building. Additional retaining walls will be needed along the west and east sides and northeast boundaries of the project. Fill slopes anticipated along Rolling Oaks and Los Padres Drives. The project will also include on-site structures, such as fences/walls, light poles, bollards and a 297-space surface parking lot.

3. SCOPE OF GEOTECHNICAL SERVICES

Gorian and Associates, Inc. conducted the site evaluation outlined in our Proposal Number: 6886-10 dated March 10, 2020 to evaluate the geotechnical site conditions affecting design and construction of the HCA Medical Building project. All phases of the evaluation were conducted by or under the supervision of a State registered geotechnical engineer and certified engineering geologist.

3.1. ARCHIVAL REVIEW

Pertinent site geotechnical and geologic information in our files was reviewed and incorporated into this site evaluation.

3.2. SITE EVALUATION / FIELD EXPLORATION3

Subsurface exploration was performed using a subcontracted supplied and operated truck mounted 8-inch diameter hollow stem auger drill rig to excavate a total of eight borings on the site to observe and sample the subsurface conditions. Four borings were geotechnical in nature and four borings were drilled for stormwater infiltration testing. The borings were extended to depths of 7 to 14 feet with all borings extended to bedrock. Refusal to advance the borings was noted in borings B-2 and B-4 through B-8.

The field exploration activities were observed by a geologist from this office, who logged the underlying materials extracted from the excavations. Bulk soil samples were obtained from borings B-4 through B-7 and Standard Penetration Tests (SPT) were performed in the borings intended for stormwater infiltration testing. Relatively undisturbed samples were not recoverable within the bedrock and shallow soil cover.

At the conclusion of logging and sampling, the borings were backfilled with spoils from the boring cuttings. However, the backfill may settle over time and the site representative should fill any depression that may occur, as necessary.

3.3. STORMWATER INFILTRATION TESTING

Four locations (two deep and two shallow) were proposed to be tested for stormwater infiltration. Ventura County requirements for storm water infiltration testing involve performing at least two infiltration tests; one at the proposed bottom of the infiltration BMP and a second test 11 feet below the bottom of the infiltration BMP. For infiltration testing, hollow-stem auger borings were excavated at four locations within the areas of the proposed BMPs. Borings B-1 and B-3 were extended to 8 feet 4 inches and 9 feet below the existing ground surface respectively for the shallow testing. Borings B-2 and B-4 were intended to be used for the deep testing; however, refusal conditions were encountered at depths of 7 feet (B-

2) and 14 feet (B-4). These borings were terminated in highly indurated volcanic bedrock and deep testing was not conducted.

At the conclusion of logging and soil sampling, the two hollow-stem borings (B-1 and B-3) were converted to infiltration rate test wells by placing a 2-inch diameter pipe in each boring subsequent to the placement of 1 foot of medium bentonite chips in the bottom of the boring. The lower 5 feet of pipe was slotted (0.02). The annular space between the slotted pipe and the wall of the excavation was backfilled using clean #3 sand. The upper portion annular space was sealed off with bentonite chips and soil.

The test zone will be pre-soaked by filling to the top of each casing with water. The water will be allowed to pre-soak for a maximum period of 24 hours. However, after the presoak period water remained in the borings indicating a lack of infiltration into the subgrade.

3.4. GEOTECHNICAL LABORATORY TESTING

A program of laboratory testing was performed to evaluate geotechnical properties of selected soil samples obtained during the subsurface exploration. Testing included compaction characteristics, shear strength parameters, and expansion potential. Corrosion potential testing was performed for this report by an independent corrosion engineer.

3.5. SITE EVALUATION ENGINEERING ANALYSES AND REPORT PREPARATION

The results of our laboratory testing, in conjunction with our field findings are the basis for our engineering analyses. We have prepared geotechnical recommendations for design and construction of the proposed project. In addition, the results of our laboratory testing, in conjunction with our field findings are the basis for our evaluation of the potential for onsite stormwater infiltration. The following will be provided in this report

1. A Geotechnical Map (Plate 1) showing the site and location of the exploratory excavations along with Geotechnical Cross Sections A-A' and B-B' (Plate 2) through proposed area of construction.
2. Logs of Subsurface Data providing a description of the encountered subsurface strata and observed groundwater conditions (Appendix A).
3. A description of the laboratory testing program, including test results (Appendix B).
4. Discussion and geotechnical recommendations regarding:
   a) Geologic hazards including seismic setting of the site and faulting;
   b) Groundwater conditions if encountered:
   c) Seismic design criteria for new buildings;
   d) Soil collapse and expansion potential;
   e) Site preparation and remedial grading;
   f) Conventional foundation design and construction;
   g) Estimated settlements;
   h) Retaining wall design and construction;
   i) Pavement (for multiple traffic indices) and hardscape design recommendations;
   j) Stormwater infiltration; and
   k) Soil chemistry analysis, by subcontract.

4. EXISTING SITE AND CONDITIONS

The 4.84 acres parcel was previously developed for a child daycare center. Rough grading of the site resulted in three terraces within the graded area. The lower terrace supports an asphalt (AC) parking lot accessed from Rolling Oaks Drive. The middle terrace supported the main building and swimming pool.

The foundation and slab of this building remain after removal of the building. Also, portions of the prior pool remain. The upper terrace supports a playfield and sports court. The grade differences between the terraces are supported by either graded slopes or retaining walls. Numerous items from the daycare center remain onsite consisting of walks, walls, fences, and trees.

The site was graded by performing cuts into the ridgeline that ascends to the south from the upper pad. Compacted fill (Gorian, 1973) was placed onsite as indicated on Plate 1. Cuts were made during grading into the underlying bedrock consisting of Conejo Volcanics. The fill was derived from the cuts and consists predominately of clayey sand with either volcanic clasts or gravels.

5. REGIONAL GEOLOGY

The site is within the Conejo Valley basin area of Ventura County. The Conejo Valley basin is a non-structural basin bounded on the south and west by the western Santa Monica Mountains and on the north and east by highlands formed of the Conejo Volcanics (see Regional Geologic Map, Figure 2). The basin is part of the Transverse Ranges Province, a series of sub parallel east to west trending ridgelines and valleys. This province is tectonically characterized by active compression in a north south direction with associated east to west trending reverse/thrust faulting, folding, and normal faulting.

The site is on the northwestern edge of the northern flank of the Santa Monica Mountains and is underlain by fill and alluvial soils mantling bedrock at depth within the flat portion of the property and bedrock mantled by minor topsoil/colluvial soils on the hillsides. The source of the alluvial materials is generally attributed to the erosion of the Santa Monica Mountains, immediately south of the site. Bedrock underlying the alluvium at depth and exposed on the hillside is comprised of Quaternary-age bedrock of the Conejo Volcanics (following the nomenclature of Dibblee, 1992).

6. SITE GEOLOGY

The site is underlain by Miocene-age volcanic bedrock referred to as the Conejo Volcanics mantled locally with Quaternary-age older alluvium and artificial fill deposits. General descriptions of these units, sans topsoil, are presented below and in the attached Logs of Subsurface Data (Appendix A). The approximate spatial relationships are shown on the attached Geotechnical Map (Plate 1).

6.1. CONEJO VOLCANICS

Miocene-age bedrock of the Conejo Volcanics underlies the site and was encountered in all eight borings at depths ranging from 1 foot (B-5 and B-8) to 8.5 feet (B-3) below the existing ground surface. As encountered, the volcanic bedrock generally consists of red to yellowish brown to brown to dark gray to dark gray mottled with red and green fine-grained basalt in a damp and indurated condition. Some manganese oxide staining was noted on fracture surfaces. All borings were terminated in bedrock when the drilling reaching refusal conditions.

6.2. OLDER ALLUVIUM

Quaternary-age older alluvium locally mantles the bedrock on the site and was encountered in boring B-1 at a depth from 4 to 7.5 feet below the existing ground surface. As encountered, the older alluvium generally consists of pale brown sandy clay with a few fine gravels in a damp and hard condition.

6.3. ARTIFICIAL FILL

Artificial fill deposits were encountered in all eight borings and ranges in thickness from 1 foot (B-5 and B-8) to 8.5 feet (B-3). Artificial fill soils are soil deposits generated by man. The approximate areas of compacted fill (Gorian, 1973) placed onsite for the construction of the child daycare center are indicated on Plate 1. As encountered, the artificial fill generally consists of brown silty fine sand to brown to yellowish brown to gray clayey fine to coarse sand with some fine to coarse gravels in a damp to moist and loose to dense condition. Locally the artificial fill consists of brown sandy silty clay (B-5 and B-7) in a wet and soft condition.

6.4. GROUNDWATER

Groundwater was not encountered during the subsurface exploration program to the maximum depth drilled of 14 feet below the ground surface. In addition, the *Seismic Hazard Zone Report for the Thousand Oaks 7.5-minute Quadrangle, Ventura County, California* (CDMG 2002) does not indicate a high groundwater level in this area. However, seepage was encountered during the construction of the section of Rolling Oaks Drive east of the intersection with Los Padres Drive (Gorian, 1974). Seepage can occur within fractures within the Conejo Volcanic Bedrock. As in any groundwater situation, groundwater level fluctuations should be anticipated during the life of the project.

6.5. LANDSLIDES

No landslides are present within or near the site nor are any shown on regional geologic maps.

7. FAULTING AND SEISMICITY

The Conejo Valley/Santa Monica Mountains area is in a seismically active region prone to occasional damaging earthquakes. The destructive power of earthquakes can be grouped into fault-rupture, ground shaking (strong motion), and secondary effects of ground shaking such as tsunami, liquefaction, settlement, landslides, etc. The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined pre-existing active or potentially active faults. No doubt there are and will be exceptions to this, because it is not possible to predict the precise location of a new fault where none existed before (CDMG, 1975).

No active or potentially active faults are known to cross or be in close vicinity to the site. No faults are known to cross the site or adjacent vicinity and the site is currently not within an Alquist-Priolo Earthquake Fault Zone as defined by the State Geologist (CGS 2018). The closest active fault is the Simi Santa Rosa Fault Zone which lies to the north of the site. The potential for ground rupture on-site due to faulting during the time period of concern is considered remote.

Nevertheless, the property will be subjected to ground motion from occasional earthquakes in the region. Significant earthquakes have occurred within a 40-mile radius of the site within the last 25 years. Such as the 1994 Northridge earthquake that produced strong ground motions within Thousand Oaks. Significant earthquakes will likely occur in this area within the life expectancy of the proposed project and the site will experience strong ground shaking from these events.

Based on the United States Geological Survey (USGS) interactive web application, *https://earthquake.usgs.gov/hazards/interactive/* probabilistic seismic hazard analyses (PSHA) predict the Design Basis Earthquake for a 475-year return period (10% chance of being exceeded in 50 years) peak horizontal ground acceleration will be on the order of 0.38g for the bedrock conditions on site. The mean magnitude from this PSHA is 6.7 (Mw) with a mean distance of 17.8 km from the property.

The Design Basis Earthquake for a 2475-year return period (2% chance of being exceeded in 50 years) peak horizontal ground acceleration will be on the order of 0.68g for the bedrock conditions. The mean magnitude from this PSHA is 6.8 (Mw) with a mean distance of 13.4 km from the property.

Secondary effects of strong ground motion include tsunami, seiche, liquefaction, seismic settlement, earthquake triggered landslides, and flooding from dam failures. Tsunamis are impulsively generated water waves that can cause damage to ocean shoreline areas. A seiche is an oscillation wave within an enclosed body of water. The site is not near the ocean or adjacent a body of water and, therefore, is not subject to tsunami and seiche hazards. Furthermore, the site is not in the vicinity of a dam failure inundation zone. Earthquake induced landslides, liquefaction, and seismic settlement affecting the proposed site development are discussed below.

8. LIQUEFACTION AND SEISMICALLY INDUCED LANDSLIDE HAZARD

The proposed development is not within an area shown to have a potential for liquefaction on the State's Seismic Hazard Zones Map (CDMG, 2000). The bedrock and alluvium underlying the site are not considered susceptible to liquefaction or seismic induced settlement.

Areas prone to seismically induced landslides are slopes with steep gradients covered with weakly indurated bedrock, loose weak soils, or debris from previous landslides. These soil conditions combined with strong ground shaking caused by an earthquake can cause the cohesive strength of soils to weaken and move down slope under the force of gravity. Site grading is not anticipated to create significant slopes that will fall within the range of conditions considered susceptible to seismic slope instability as discussed above.

9. ONSITE STORMWATER INFILTRATION

As previously indicated, testing was performed for onsite stormwater infiltration. Shallow test wells were constructed in the northern portion of the site. Deeper infiltration test wells were not constructed due to the encountered bedrock in which refusal to advance the auger occurred. Water was introduced into the wells to presoak the soils prior to performing infiltration testing. However, the presoak water did not fully dissipate into the surrounding soils indicating a lack of infiltration. Therefore, onsite stormwater infiltrator is not considered feasible for the site due to the presence of the underlying bedrock and soils that are not suitable for stormwater infiltration.

10. CONCLUSIONS AND RECOMMENDATIONS

10.1. GENERAL

The site at 400 East Rolling Oaks Drive was evaluated from a geotechnical standpoint for the proposed medical office building. The construction described herein is feasible provided the following geotechnical recommendations are incorporated into the design and construction of the project. Use of this report constitutes the owner and parties using this report have fully read and understand the contents of this report. Construction including site preparation, grading, and fill placement should be per applicable building codes.

10.2. GEOLOGIC CONDITIONS

The site is underlain by shallow bedrock as indicated in the logs of borings. The bedrock is hard (indurated) and stable, therefore, some difficult excavation should be anticipated during site preparation operations. In Gorian, 1973, it is indicated that the fill was generated from cuts onsite and the cut material was predominantly volcanic basalt with a clayey sand matrix readily rippable with a D-9 dozer. Also, indicated is that the rocky material was easily broken down to eight-inch maximum size under the compactive effort applied by a 5 x 5 sheepsfoot roller. However, in a bedrock site, the rock hardness can very due to weathering and depth with the hardness generally increasing with depth.

10.3. SITE PREPARATION OPTIONS

As shown on Plate 1, the building will be centered roughly within the site. The building pad will be graded mostly with shallow cuts to the south. More detailed information regarding the grading within the building pad will be available when a fine grading plan is available for review. However, currently it appears there are two limited areas of fill within the building area, on the west there is an area of prior fill and on the east an area of proposed fill. Therefore, there are two options regarding preparation of the building pad. The first is to grade the site as shown without removal of the daylight (contact between fill and cut) from the building pad. This will require all footings be embedded or extended into the underlying bedrock. This may require deepening of footings above that planned based on encountered field conditions. The second option is to undercut the pad to a minimum of 3 feet below the footings to remove the daylight line. Either option is suitable from a geotechnical standpoint. The daylight line would not need to be removed for slab support. Grading of the site should be reviewed when fine grading plans area available.

10.4. SEISMIC DESIGN PARAMETERS

As previously discussed, active faults identified by the State are not onsite nor is the site within an Alquist-Priolo Earthquake Fault Zone. Nevertheless, the site is within a seismically active region prone to occasional damaging earthquakes.

Structures within the site may be designed using procedures for seismic design presented in ASCE/SEI 7-16. Mapped acceleration parameters are initially determined for sites having a shear wave velocity of 2,500 feet per second (Section C11.4.4). The S_s and S_1 values are adjusted to obtain the maximum considered earthquake (MCE) spectral acceleration values for the site based on its site class of C. The seismic design parameters for the site's coordinates (latitude 34.1737 N and longitude 118.8692 W) were obtained from the USGS web based spectral acceleration response maps and calculator: (http://earthquake.usgs.gov/hazards/designmaps/). The parameters are presented below. The complete Design Maps Detailed Report is attached hereto in Appendix C.

Risk Category II and III

SEISMIC PARAMETER	VALUE PER CBC
Short Period Mapped Acceleration (S_s)	1.45g
Long Period Mapped Acceleration (S_1)	0.52g
Site Class Definition	C
Site Coefficient (F_a)	1.2
Site Coefficient (F_v)	1.48
$S_{MS} = F_aS_s$	1.74g
$S_{M1} = F_vS_1$	0.77g
$S_{DS} = 2/3S_{MS}$	1.16g
$S_{D1} = 2/3S_{M1}$	0.51g
PGA_M	0.61g

The purpose of the building code earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage nor maintain function. Therefore, values provided in the building code should be considered minimum design values and should be used with the understanding site acceleration could be higher than addressed by code-based parameters. Cracking of walls and possible structural damage should be anticipated in a significant seismic event.

10.5. GROUNDWATER

As previously indicated, groundwater was not encountered during the subsurface exploration program to the maximum depth drilled of 14 feet below the ground surface. Therefore, groundwater is not anticipated to be encountered during the project construction. However, seasonal seepage can occur within fractures of the Conejo Volcanic bedrock. As in any groundwater situation, groundwater level fluctuations should be anticipated during the life of the project.

10.6. SITE PREPARATION AND GRADING

10.6.1. General

As previously discussed, the site may be graded with or without undercutting of the pad. Recommendations are provided below for grading of the site along with undercutting recommendations is this option is selected for the site.

The following sections contain geotechnical recommendations concerning site preparation and grading. All aspects of grading should be per the City of Thousand Oaks Building Code unless superseded by recommendations herein.

10.6.2. Existing Utilities

Existing utilities are present in the street with laterals to the lot. Therefore, protection of existing utilities to remain will be necessary during remedial grading and care should be taken to avoid surcharging them with proposed construction or building loads.

10.6.3. Site Clearing

The site should be cleared of unnecessary improvements, vegetation, and debris prior to beginning remedial removal operations. Material generated during site clearing should be removed from the site prior to starting earthwork. The removal should include soils disturbed during the removal process.

10.6.4. Tree Removal

Tree removal will be necessary within the proposed area of construction. A two to three cubic soil loss should be anticipated with each root ball removed. The resulting cavity from the tree removal should be cleaned and observed by this office prior to fill placement. Roots over one-half inch diameter should be removed from the fill and when encountered within the areas of soil removal. Brush should be cut from the slopes and not pulled resulting in disturbance of the slope surface.

10.6.5. Soil Removals

The upper loose or soft topsoil or native alluvial soils and existing non-engineered fill soils should be removed and replaced as engineered compacted fill for the support of the proposed construction. For planning purposes, the minimum removal is estimated at one foot. The removals should be measured from the existing or finished subgrade, whichever is the deeper removal. However, if deeper unsuitable areas are uncovered, the additional removal should be determined based on field observations by this office. Soil removals should be performed within all areas of construction (cut or fill areas) including parking and drive areas.

After removals are completed as addressed above, the exposed ground surface should be observed and tested by a field representative of this office to determine if additional soil removal is required. Fill soils should not be placed until the geotechnical observation of removal areas is complete.

10.6.6. Building Pad Undercut (building pad over-excavation)

If it is desired to have the footings supported uniformly in compacted fill in lieu of bedrock, the building pad should be undercut. The undercut should be performed to a minimum depth of 5 feet below the proposed pad grade within the building footprint or a minimum of 3 feet of compacted fill beneath the footings, whichever is the deeper removal. The removal should extend a minimum of 5 feet past the building footprint. However, the removals may be reduced if a uniform thickness of fill can be place directly over in place bedrock as determined by this office.

After the removals are completed, the exposed removal bottom should be observed by a representative of this office to evaluate if additional removals are needed. After removals, fill can be recompacted as outlined below.

10.6.7. Retaining Wall Soil Removal

No additional retaining wall soil removal is necessary other than a minimum of 1 foot as describe above providing the footings area established in firm in place bedrock, engineered compacted fill, or firm alluvial soils. Expansion joints are suggested where a retaining wall will cross a daylight line. Footing excavations should be observed by this office prior to determine if additional soil removal is necessary.

10.6.8. Oversized Rock
Oversized rock should be anticipated within the cuts made into the Conejo Volcanics along the southern portion of the site. Rock over 8 inches should not be placed in the fill and over 6 inches should not be placed in the building areas. Rock over these sizes should be removed from the site.

10.6.9. Processing
The surface of the in-place soils should be processed prior to fill placement. Processing of the in-place soils should consist of scarification to a depth of 6 to 8 inches. The scarified surface should be relatively free of uneven features that would prevent uniform compaction. Soils should be moisture conditioned and compacted to at least 90% relative compaction. Hard in place rock need not be scarified.

10.6.10. Fill Placement
Soils excavated from within the site may be used as fill providing the soils are cleaned of major vegetation, trash, and debris. However, clayey soils should be salvaged to build fill slope faces (if constructed). Sandier soils may not have sufficient cohesion for slope construction.

Fill soils should be placed in thin uniform lifts not exceeding 8 inches in depth. The moisture content should be controlled so the fills are slightly over the optimum moisture content prior to compaction. Fills should be compacted to a minimum density of 90% relative compaction. Soils placed within building pad areas should be mixed and blended so the completed engineered compacted fill pad is relatively uniform.

10.6.11. Utility Trenches
Utility trenches, including those associated with site drainage piping systems, should be compacted to at least 90% relative compaction. Utilities should be constructed in accordance with current practice and standards (such as the current *Green Book*).

10.6.12. Relative Compaction
Relative Compaction is the ratio of in-place dry soil density to the maximum dry soil density determined in general conformance with ASTM test method D 1557-91.

10.6.13. Shrinkage/Bulking
Shrinkage is the volume loss of soils from cut to fill and from removal areas. Bulking is the volume expansion of the earth materials from cut to fill. The amount of volume change will depend on the material in situ density, the final compacted density achieved, losses due to spillage, etc. Subsidence is considered to account for densification on the upper 6 inches of surface soils over the site and stripping of vegetation from the site, and is expected to remove about 2 to 3 inches of grade. Removal of asphalt and prior construction could result in higher subsidence values.

Shrinkage will vary depending upon placement and compaction and could range from 5 to 10 percent shrinkage (soil bulking is not anticipated). Bulking in the bedrock areas could be 5 percent or more depending upon the amount of oversized rock excavated. Estimated factors based on an assumption the fills will be placed and compacted as recommended herein. The values are provided for gross estimating purposes only.

10.7. EXCAVATIONS

10.7.1. General
The following sections are for support of temporary cuts and excavations for retaining wall construction. Temporary slopes will encounter a varied bedrock/soil profile and should conform to the requirements of CAL/OSHA and any other applicable regulations.

10.7.2. Temporary Slopes

When construction plans are available, they should be evaluated for temporary slopes. Temporary slopes in bedrock may be made at a 1/2(horizontal):1(vertical) gradient. However, if fracture bedrock is exposed in the backcut, the backcut may need to be laid back to a flatter gradient or other protective measures provide to protect against loose rock. Surcharge loading, such as construction equipment or vehicle traffic, should be kept back sufficient distances from excavations onsite.

10.7.3. Shored Excavation

Shoring will be required whenever vertical cuts are made such as for utility installation over the depth allowed for the soil conditions outlined in CAL/OSHA. Temporary shoring should be designed for an active pressure of 30 pounds per cubic foot. Additional recommendations can be provided when the need for shoring is known.

10.8. SHORING

10.8.1. General

Shoring within the site may be required based on the project is laid out and how the cuts are made for the basement and retaining walls. Recommendations for tiebacks are not included in this report, however, they can be provided if necessary.

The project civil engineer should prepare an excavation plan detailing the excavation and relationship to existing utilities and structures. This office should review the excavation plan prior to starting construction. In addition, this office should evaluate possible loads (such as crane loading) than may surcharge the excavation.

10.8.2. Shored Excavation

Shoring for excavation may consist of cantilevered soldier piles. Lagging should be used to support the cut between the piles. Grouting is the preferred method to fill the voids between the cut and lagging. The shoring should be designed to include the lowest construction elevation. Care will be required to avoid damaging buried utilities or foundations of adjacent structures. The shoring will the subsurface profile as described previously herein and in the attached Logs of Subsurface Data (Appendix A).

10.8.3. Surcharge Loading

An area surcharge of 300 psf should be included in the shoring design where the shoring is near street traffic. The lateral pressure on the shoring due to a uniform area surcharge of intensity q (force/area) is equal to a uniform pressure of 0.4q over the entire height of the wall. Surcharge on the shoring from construction equipment (e.g. crane or concrete pump) directly adjacent the top of a shored cut should be evaluated by this office on an individual basis.

10.8.4. Soil Pressure

Shoring should be designed for lateral earth pressure plus lateral pressure imposed by existing adjacent foundations or surcharges. Cantilevered shoring systems should be designed for an active earth pressure distribution of 30 pounds per cubic foot (pcf) with level ground behind the shoring. Additional pressures can be provided based on the shoring locations and loading. This shoring pressure (cantilever walls) does not include lateral loads from surcharges (such as crane loading or adjacent structures) near the top of the excavation. The value of 30 pcf is an ultimate value without a factor of safety. The width of active pressure acting on the pile below the bottom of the excavation should be two pile diameters for a cantilevered soldier pile.

10.8.5. Soldier Pile Passive Pressure and Vertical Capacity

The lower ends of the soldier piles will be seated in alluvial deposits is as described previously herein and in the attached Logs of Subsurface Data (Appendix A). For isolated piles (spaced at least 3 diameters center to center) the passive earth pressure should start at zero at the excavated grade. This value may be increased at a rate of 300 pounds per cubic foot for each foot of depth below the proposed base of excavation to a maximum of 3000 pounds per square foot. The surface area (pile diameter) that the allowable passive pressure may induce passive resistance may be doubled for soldier beams that are a minimum of 3 diameters apart center to center.

For vertical support, a unit friction value of 300 pounds per square foot may be used for that portion of the soldier pile encased in structural concrete or drilled and cast concrete pile extending below the lowest depth of excavation. The unit of friction is independent of the pile diameter; however, the piles should be at least 24-inch diameter with a minimum embedment depth of 15 feet below the lowest excavation depth. Fixity may be assumed at 5 feet below the lowest unsupported grade (such as the basement excavation).

10.8.6. Cantilever Shoring Tilt
Similar to a cantilever retaining wall, cantilever shoring designed for an active pressure can yield at the top to develop full active pressure. Generally, tilt is a function of the wall height and is estimated at .001 to .002 of the wall height.

10.8.7. Lagging
Lagging consisting of treated timber will be required the entire depth of the shored excavation. Wood lagging should be new rough timber (full dimension) Douglas Fir, straight, free of bends, and free from defects that might impair structural strength. Lagging to be left in-place shall be pressure treated for contact with soil. The upper two feet of the shoring and lagging measured from the adjacent grade should be removed when the shoring is no longer needed for support of the excavation. The resulting cavity from removed shoring should be backfilled with grout/slurry or soil compacted to a minimum of 90% relative compaction.

Lagging should be designed to resist an equivalent fluid pressure equal to 30 pcf measured below the ground surface. A maximum lagging pressure of 400 psf may be assumed where the maximum spacing of soldier piles does not exceed 8 feet center to center. An alternate to installing lagging would be to construct the shoring as a continuous gunite/shotcrete wall descending as the excavation proceeds. Cavities behind the lagging and retained soils should be filled with sand/cement slurry (preferred).

10.8.8. General Considerations
The basement excavation can be made with ordinary excavating equipment. Soils between the existing foundations and proposed shoring system should be maintained in an undisturbed and intact condition. Caving of soldier beam excavations should be anticipated since sandy materials will be encountered in the excavations. The shoring contractor should be prepared to provide methods to prevent caving such as the use of hollow stem augers, casing, or drilling mud.

10.8.9. Barricades
Appropriate barricades should be placed at the top of all temporary excavations that are approached by pedestrians or public vehicle traffic (such as in streets or parking areas).

10.8.10. Shoring System Monitoring
The shoring system should be monitored for vertical and horizontal movements at the top of each soldier beam. A licensed surveyor should perform the surveying.

The reference points and pile tops should be read prior to commencing the excavation. To create a baseline, all soldier piles should be surveyed twice (approximately one day apart) before beginning excavation. Additional readings should be performed roughly biweekly throughout construction until the shoring and excavation is complete. More frequent reads may be required at critical times of construction or if significant movement is indicated. After completion of the shoring construction and excavation, readings may be taken biweekly until the shoring is no longer needed for support of the excavation.

The survey data should be submitted to Gorian and Associates, Inc. within 24 hours of the measurements. The tolerable movement for any location within the structure will be evaluated with the data and is dependent on the soil conditions at that location, the stage of construction, and adjacent structures or loading. Some movement of the shoring can be expected and is considered tolerable. In general, movement in excess of 2 inches horizontally or vertically will require supplemental shoring before excavation continues.

10.9. SLOPE CONSTRUCTION

10.9.1. General

Excavations for the proposed development may be supported by cut slopes and retaining walls. Cut slopes within bedrock may be made at a 1-1/2(horizontal):1(vertical) gradient. Fill slopes and slopes within a soil profile should be made at a 2(horizontal):1(vertical) gradient.

10.9.2. Cut Slopes

Cut slopes within bedrock may be made at a 1-1/2(horizontal):1(vertical) gradient. Cut slopes along the southern perimeter of the site will encounter Conejo Volcanic bedrock at shallow depths as illustrated in the attached cross sections. The tops of these slopes should be rounded where topsoil is exposed in the cut. Cut slopes should be observed by an engineering geologist from this office for the presence of adverse geologic conditions. Hard rock conditions may be encountered within the Conejo Volcanics.

10.9.3. Cut Slope Seepage

Cut slopes within the volcanics are known to seep water after significant rainfall through fracturing within the bedrock. Generally, this condition is not detrimental to the slope.

10.9.4. Fill Slopes

Fill slopes (if constructed) should be keyed and benched into firm competent native materials per the City of Thousand Oaks Building Code. All keyways should be a minimum of 15 feet wide and cut to a minimum depth of 2 feet at the toe into firm competent in place materials (see the Soil Removals Section). Keyways should be tilted into the slope and should be at least 3 feet deep at the heel (measured from below the slope toe elevation). A representative of this office should observe the keyways prior to fill placement.

Select grading will be required when placing fill materials within 15 feet of slope faces. Fill soils near slope faces should have enough clay to develop at least 250 pounds per square foot of cohesive shear strength for a 2(horizontal):1(vertical) slope. This is a minimum cohesion based on standard practice to provide for surficial slope stability. However, highly expansive clayey soils should not be placed near a slope face.

Where possible the outer slope faces should be overfilled and trimmed back to provide for firm, well-compacted surfaces. The slope faces should be tested and reworked as necessary to achieve the required compaction.

10.9.5. Slope Maintenance

Slopes constructed within the site will require maintenance or protection to reduce the risk of erosion and degradation with time due to natural or man-made conditions. Future performance of slopes will depend on control of rodents and maintenance of drainage structures and slope vegetation as discussed below. Drainage should be provided away from the top or toe of the slopes.

Slope (fill or soil cut slopes) planting should consist of dense, deep rooting, drought resistant ground-cover and shrubs or trees. Hard rock areas may not be suitable for planting. A reliable irrigation system should be installed, adjusted so over-watering does not occur, and periodically checked for leakage. Over-watering of slopes can cause expansion, erosion, and surficial failures, and should be avoided. Care should be taken to maintain a uniform, near optimum moisture content below the slope surface, and to avoid over drying, or excess irrigation. These conditions can reduce the potential for soil softening and strength loss, which could lead to slumping of the slope face. Drainage structures should be kept in good condition and cleaned the entire length to the outlet in an approved drainage course. Burrowing animals (e.g., ground squirrels) can destroy slopes; therefore, where present, immediate measures should be taken to eliminate them.

10.10. SOIL EXPANSIVENESS

An expansion test performed for the site indicate the onsite soils are moderately expansive (51-90 soil expansion range. Additional expansion tests will be required to determine the expansiveness of the completed building pad.

Expansive soils contain clay particles that change in volume (shrink or swell) due to a change in the soil moisture content. The amount of volume change depends upon the soil swell potential, availability of water, and the soil restraining pressure. Swelling occurs when clay soils become wet due to excessive water. Excessive water can be caused by poor surface drainage, over-irrigation of lawns and planters, and sprinkler or plumbing leaks.

Expansive clay soils can cause distress both as uplift and shrinkage or settlement. Construction on expansive soil has an inherent risk that should be acknowledged and understood by the builder and property owner. Recommendations presented in this report are intended to reduce the potential for expansive soil action. However, these recommendations are not intended, nor designed to provide complete and full mitigation of expansive soil conditions. Additional recommendations can be provided to further reduce the risk of expansive soil movement.

10.11. FOUNDATION RECOMMENDATIONS

10.11.1. General

As previously indicted the building foundations may be supported entirely in bedrock or engineered compacted fill. However, the footings should not be supported in both bedrock and fill. Therefore, if supported in fill, the building should be undercut to provide a minimum of 3 feet of fill below the footings.

10.11.2. Conventional Foundation Design Data

Conventional footings within the underlying bedrock or compacted fill for the building structural support may be designed using an allowable bearing pressure of 4,000 pounds per square foot (psf). Light structures such as site walls or monument structures may be designed using a bearing pressure of 1,500 pounds per square foot when embedded in compacted fill or firm in-place native soils. The bearing pressure is for dead plus live loads and may be increased by one-third when considering wind or seismic loads.

Footings should have a minimum width of 12 and 24 inches for continuous and isolated footings, respectively. The embedment should be a minimum of 24 inches for perimeter and interior footings. The lowest adjacent grade is the lowest soil grade adjacent the footings, interior or exterior. Embedment of interior and retaining wall footings may be measured from the top of the interior concrete slab on-grade. Deepening of the footing may be necessary to reach firm in-place bedrock below any weathered bedrock zone.

Shallow footings adjacent a retaining wall should be stepped down below a 2(horizontal):1(vertical) plane projecting upward from the bottom of the retaining wall footings or the wall should be designed for the added surcharge. Steel reinforcement should be per the structural engineer's recommendations. However, minimum reinforcement for continuous footings should consist of two number five bars in the top and bottom.

10.11.3. Lateral Resistance

Lateral forces on foundations may be resisted by passive earth pressure and base friction. For the sides of footings bearing against engineered compacted fill or competent native materials, the lateral passive earth pressure may be considered equal to an equivalent fluid having a density of 300 pounds per cubic foot (pcf). Base friction may be computed at 0.4 times the normal load. Base friction and passive earth pressure may be combined without reduction and may be increased by one-third when considering wind or seismic loads.

The lateral resistance is an ultimate design in that no safety factor is included to preclude the use of a 1.5 safety factor in the design of retaining walls. However, the values may be increased by one third for temporary loading.

10.11.4. Estimated Foundation Settlements

Settlement of footings should be evaluated once building footing locations and structural loads are known. However, footing settlement for static loading is anticipated on the order of ¼ to ½ ± inch, with a maximum differential settlement of 1/2± inch over a span of approximately 30 feet or between adjacent individual footings. This is provided building construction is started directly after footing excavation, footings are cast soon after the footing excavation, and construction is completed in a timely manner. Settlements due to static loading are expected to occur rapidly as the loads are applied.

All structures settle during construction and some minor settlement of the structures can occur after construction during the life of the project. Minor wall cracking could occur within the structure associated with expansion and contraction of the structural wood members due to thermal or moisture changes. In addition, wall or slab cracking may be associated with settlement or expansive soil movement. Additional settlement/soil movement could occur if the soils become saturated due to excessive water infiltration generally caused by excessive irrigation, poor drainage, etc.

10.11.5. Footing Excavations

Footings should be cut square and level and cleaned of slough. Soil excavated from footing and utility trenches should not be spread over areas of construction unless properly compacted. A representative of this office should observe the footing excavations prior to placing reinforcing steel. Soils silted into the footing excavations during moistening operations should be removed prior to casting the concrete. Footings should be cast as soon as possible to avoid deep desiccation of the footing subsoils.

10.11.6. Footing Subgrade Moisture

Footing subgrade soils should be kept in a moist condition until concrete placement. Saturated soils should be removed from the footing excavations prior to casting the footings.

10.12. SLABS-ON-GRADE

10.12.1. Site Preparation

Concrete slabs on-grade may be supported on compacted engineered fill soils or in place bedrock. Subgrade soils should be recompacted prior to placing the sand subbase, if the soils were disturbed during footing or utility construction.

10.12.2. Design Data

Concrete slabs on-grade should be 5 inches thick and underlain by 6 inches of 3/4± clean aggregate. Recommendations for exterior concrete drives are provided later herein under *Preliminary Pavement Design* later herein. Slab should be reinforced with a minimum of number 3 bars at 18-inch centers in each direction. Reinforcement should be placed and kept at slab mid-depth.

Exterior concrete slabs-on-grade (non-auto traffic) and walkways should be a minimum of 4 inches thick and underlain by a minimum of 4 inches of sand. Exterior slabs should be reinforced with minimum No. 3 bars on 24-inch centers in each direction. Reinforcement should be placed at mid-depth of the slab. Sidewalks may be constructed of non-reinforced concrete provided they are cut into square panels (i.e., 4-foot-wide walks should be cut into 4 by 4-foot squares).

10.12.3. Premoistening

Soils under lightly loaded slabs on-grade should be premoistened to 3% over the optimum moisture content for a depth of 18 inches.

10.12.4. Moisture Vapor Retarder Layer

A moisture vapor retarder layer should be incorporated into the slab on-grade design within the building interior. The water vapor retarder should be one that is specifically designed as a vapor retarder and consist of a minimum 15 mil extruded polyolefin plastic and comply with Class A requirements under ASTM E1745 (*Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*). The vapor retarder should be installed in accordance with ASTM E1643. The water vapor retarder should be installed in direct contact with the concrete slab along with a concrete mix design to control bleeding, shrinkage, and curling (ACI 302.2R). The vapor retarder shall be installed over a 4-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. The vapor retarder should be placed per the manufacture's recommendations ASTM E1643-98(2005) *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*. All joints should be lapped and sealed along with proper sealing of perforations such as for plumbing. In addition, various trades and the concrete contractor should be required to protect the moisture retarder during construction.

Perforations through the moisture vapor retarder such as at pipes, conduits, columns, grade beams, and wall footing penetrations should be sealed per the manufacture's specifications or ASTM E1643. Proper construction practices should be followed during construction of slabs on-grade. Repair and seal tears or punctures in the moisture barrier that may result from the construction process prior to concrete placement.

Minimizing shrinkage cracks in the slab on-grade can further minimize moisture vapor emissions. A properly cured slab utilizing low-slump concrete will reduce the risk of shrinkage cracks in the slab as described herein.

The concrete contractor should be made aware of the moisture vapor retarder and required to protect the layer. The concrete contractor should make the necessary changes in the concrete placement and curing for concrete placed directly over the retarder. Placing the concrete directly on top of the moisture vapor retarder layer allows the layer to be observed for damage directly prior to concrete placement.

The slabs should be tested for moisture content prior to the selection of the flooring and adhesives. Moisture in the slabs should not exceed the flooring manufacture's specifications. The concrete surface should be sealed per the manufacture's specifications if the moisture readings are excessive. It may be necessary to select floor coverings that are applicable to high moisture conditions.

10.13. SUBTERRANEAN DRAINAGE AND WATERPROOFING

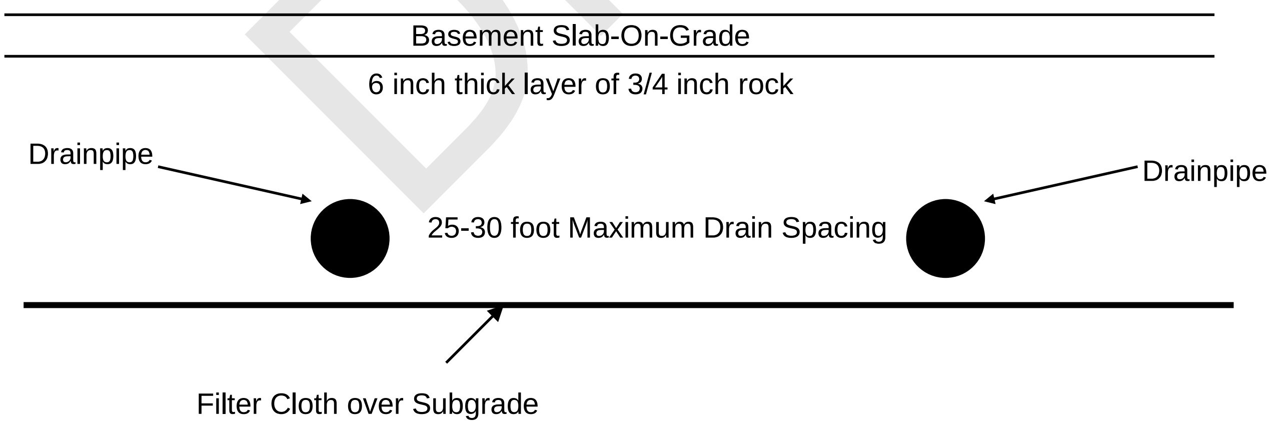
The bedrock within the area of the subterranean portion of the first floor can seep through fractures. Therefore, it is recommended that a subdrain be placed below the slab within the portion of the first floor adjacent the southern building retaining wall. In addition, the retaining walls should be waterproofed with back drains.

10.13.1. Below Slab Drain

Below slab drains are intended to provide drainage of groundwater from below the interior floor. However, drains will not drain water naturally held by the soils or stop vapor migration.

The interior slab in the area of bedrock cut should be constructed on 6 inches of 3/4± rock. An acceptable gradation would be as specified in the Standard Specifications for Public Works Construction (Greenbook) Table 200-1.2, Crushed Rock and Rock Dust for 3/4-inch rock. However, the rock may be rounded or crushed. The rock should be placed on a properly prepared subgrade as addressed herein and should be separated from the subgrade by a single layer of filter cloth. Filter cloth having a maximum equivalent opening of 0.212 mm (70 U.S. sieve size) should be lapped at least 12 inches at the seams and the seams sealed per the manufacture's specifications.

Directly above the filter cloth within the rock, at least one row of 4-inch PVC (Schedule 40) perforated pipe should be placed with holes down roughly parallel to and roughly 10 feet in from the southern retaining wall. Should it be desirous to add additional drains, the drains should be placed at a maximum pipe spacing of 25-30 feet and preferably with a slight slope to drain (or horizontal if necessary). Piping should be routed around footings and grade beams wherever possible however should not extend below any footing. Where piping must cross a structural element, a sleeve should be constructed per the structural engineer's design. Manifold piping or solid piping connecting the drains to the sump system or storm drain may be 4 inch or larger PVC (Schedule 40) that is non-perforated with glued connections. Drainpipes should be connected to a single outlet pipe prior to exiting the building. Connector pipes should be placed preferably with a slight slope to drain (or horizontal if necessary). Rock should be carefully placed over the piping so as not to disturb the pipe layout or distort the piping.



Suggested Below Slab Drain Detail (NTS)

10.14. RETAINING WALLS

10.14.1. General

Retaining walls will be required to develop the site as shown on Plate 1. The walls may be either conventional, segmental (mechanically stabilized earth, MSE), soldier beam and lagging, or soil nailed. All of these walls have been used within the City in one form or another. A soil nail wall would be ideal of the southern wall except that the nail would extend offsite and therefore require an offsite easement. A segmental wall could be used along the eastern boundary of the site. The different wall types should be evaluated for use within the project. Additional, design parameters can be provided based on the selected wall type.

10.14.2. Lateral Earth Pressures

Site retaining walls allowed yield at the top should resist an active pressure exerted by compacted backfill or retained soil. Walls that may yield at the top should be designed for an equivalent fluid pressure equal to 30, 45, 55 pcf for a level, 2(horizontal):1(vertical) and 1-1/2(horizontal):1(vertical) condition behind the wall, respectively. The wall pressures are for low to moderate expansive backfill materials. Wall heights are measured from the top of the retained material to the bottom of the foundation.

Permanent braced retaining walls (including basement walls) should be designed for a pressure of 30H (psf) where H is the height of the retained soil. The pressure distribution may be trapezoidal with the pressure increasing from zero at the base of the wall to full pressure at .2H measured from the base of the wall. H is the wall height. The pressure may be reduced starting at .8H to zero pressure at the ground surface.

Shallow footings adjacent a retaining wall should be stepped down below a 2(horizontal):1(vertical) plane projecting upward from the bottom of the retaining wall footings or the wall should be designed for the added surcharge. Surcharge on the wall from loads directly adjacent the wall can be evaluated by this office on an individual basis.

Surcharges may be treated as additional height of backfill. Assume one foot of additional height for each 125 psf of areal surcharge. Vehicle wheel loads (light to moderate) should be taken as two feet of additional surcharge. Lateral loads imposed by adjacent shallow foundations should be added to the lateral earth pressure. A surface surcharge of 300 pounds per square foot (psf) should be included in the design where the shoring is near street traffic zones.

10.14.3. Seismic Pressure

Walls less than 6 feet in height should not require a seismic pressure. Walls above a height of 6 feet high should be designed for a dynamic load (ΔPae) as provided in Agusti and Sitar (2013) as follows:

Basement (restrained) walls with level backfill: $\Delta P_{ae} = ½ * \gamma * H^2 (0.68\ PGA_M/g)$

Cantilever (unrestrained) wall with level backfill: $\Delta P_{ae} = ½ * \gamma * H^2 (0.42\ PGA_M/g)$

Cantilever (unrestrained) wall with sloping backfill*: $\Delta P_{ae} = ½ * \gamma * H^2 (0.70\ PGA_M/g)$

*Applicable for sloping backfill that is no steeper than 2:1 (horizontal:vertical).

H = retained earth height (use γ = 120 pcf for compacted backfill)

$PGA_M = 0.61g$

For cohesionless soils, the point of application of the dynamic load increment is at 1/3H, where H is the retained height. For soils with cohesion, the point of application may vary between 0.37H to 0.40H; for additional information, see Agusti and Sitar (2013) listed in the references.

10.14.4. Waterproofing
Retaining walls (basement and site) should be waterproofed.

10.14.5. Drainage
On-site retaining walls and subterranean building walls should be constructed with a backdrain consisting of a manufactured composite drain board or a section of aggregate drain material. An aggregate drain should consist of a minimum one-foot wide continuous section of 3/4± rock (or pea gravel). The aggregate drain material should extend from the base of wall to within 2 feet of the top of exterior walls. The upper 2 feet of exterior wall backfill should consist of compacted native soils. A layer of filter cloth should be placed between the drain material and soil to minimize the migration of fines into the drain material. Composite drain boards or aggregate sections should be drained by a perforated drainpipe (perforations 3/8 inch or smaller, perforations down) located in the lower portion of the drain. The drainpipe invert should be at least 6 inches below any adjacent slab-on-grade. Drainpipes may be laid flat along the back of the wall.

10.14.6. Backfilling
Retaining walls should be backfilled with granular on-site materials. The backfill should be placed in 6-inch lifts at slightly over optimum moisture content and compacted to at least 90 percent of the maximum dry density. Backfill should be benched into the backcut slope if the backcut is flatter than ½(horizontal):1(vertical). Light equipment should be used immediately behind the walls to prevent possible overstressing. Backfill should be compacted to a minimum of 90% relative compaction using light equipment.

10.15. PRELIMINARY PAVEMENT DESIGN
The following preliminary pavement designs are for preliminary design. The final structural section designs should be confirmed at the conclusion of grading based on actual R-Value tests performed on the upper subgrade soils. The upper 6 inches of subgrade and all aggregate base material should be compacted to at least 95 percent of the maximum dry density, just prior to placing the asphalt. Planter areas should be graded so that water to drains onto, rather than beneath, adjacent AC pavement and curbs.

Location	**Recommended Structural Section**
Parking Stalls	3"A.C. / 6" A.B.
Drive Areas (light weight vehicles)	3"A.C. / 8" A.B.
Entrance Drive Area, Heavy Truck Traffic Areas	3"A.C. / 10" A.B.
Rolling Oaks or Los Padres Drive widening (if necessary) * *(or meet existing, whichever, is the greater)	4” A.C. / 10” A. B.

A.C. = Asphaltic Concrete A.B. = Class 2 Aggregate Base

Portland Cement Concrete Pavement Designs

Location: Entrance Drive Area, Heavy Truck Traffic Areas, and trash enclosure aprons.

Recommended Concrete Thickness	Recommended Aggregate Base Thickness	Minimum Concrete Design Strength	Recommended Steel Reinforcement
7.5"	4"	3500 psi	#3 bars @ 18" O.C. each way

10.16. SOIL CORROSION

The results are presented herein of analytical laboratory testing to evaluate the potential for corrosion of materials in contact with the onsite soils. Testing was performed by Project X Corrosion Engineering on a soil sample considered to represent the onsite soils (the test results are attached hereto in Appendix B). From ACI Table 19.3.1.1, the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1. The potential for corrosion of metals in contact with the onsite soils is very severely corrosive as determined from Table 1. For specific recommendations, a corrosion engineer should be consulted.

ACI Table 19.3.1.1 – Exposure Categories and Classes

Category	Class	Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass	Dissolved sulfate (SO_4^{2-}) in water, ppm[1]
Sulfate (S)	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \leq SO_4^{2-} < 0.20$	$150 \leq SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \leq SO_4^{2-} < 2.00$	$1500 \leq SO_4^{2-} < 10{,}000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$

1 ppm (parts per million) = milligrams per kilogram mg/kg of dry soil weight

ACI Table 19.3.2.1 – Requirements for Concrete by Exposure Class

Exposure Class	Maximum *w/cm*	Minimum f_c', psi	Cementitious materials - Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II	Types IP, IS, or IT with (MS) designation	MS	No restriction
S2	0.45	4500	V	Types IP, IS, or IT with (MS) designation	HS	Not permitted
S3	0.45	4500	V plus pozzolan or slag cement	Types IP, IS, or IT with (MS) designation plus pozzolan or slag cement	HS plus pozzolan or slab cement	Not permitted

ACI Tables 19.3.1.1 and 19.3.2.1 - ACI 318-14 Building Code Requirements for Structural Concrete

Table 1. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity, ohm-cm	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >10,000	Very mildly corrosive

F. O. Waters, Soil Resistivity Measurements for Corrosion Control, Corrosion. 1952, Vol, No. 12, 1952, p. 407.

10.17. SITE DRAINAGE

Positive drainage should be provided away from structures and hardscape during and after construction per the grading plan or applicable building codes. Water should not be allowed to gather or pond against foundations. In addition, planters near a structure should be constructed so that irrigation water will not saturate footing and slab subgrade soils. Landscape planting and trees should be located to avoid roots extending beneath foundations and slabs. Irrigation lines and landscape watering should be kept away from building lines wherever possible. Irrigation lines and sprinklers should be placed so that water is not sprayed on the footings or saturates the soil adjacent the footings.

11. CLOSURE

This report was prepared under the direction of State registered geotechnical engineer and certified engineering geologist for the addressee and design consultants solely for design and construction of the project as described herein. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Gorian and Associates, Inc. disclaim any and all responsibility and liability for problems that may occur if the recommendations presented in this report are not followed.

This report may not contain sufficient information for other uses or the purposes of other parties. Recommendations should not be extrapolated to other areas or used for other facilities without consulting Gorian and Associates, Inc. Services of this office should not be construed to relieve the owner or contractors of their responsibilities or liabilities.

The scope of the services provided by Gorian and Associates, Inc. and its staff, excludes responsibility and/or liability for work conducted by others. Such work includes, but is not limited to, means and methods of work performance, quality control of the work, superintendence, sequencing of construction and safety in, on, or about the jobsite.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, this office should observe all aspects of field construction addressed in this report. Individuals using this report for bidding or construction purposes should perform such independent investigations as they deem necessary.

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Please do not hesitate to call if you have any questions concerning this geotechnical report or require additional information.

Respectfully,
Gorian and Associates, Inc.

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William F. Cavan, Jr., CEG 1161
Principal Engineering Geologist

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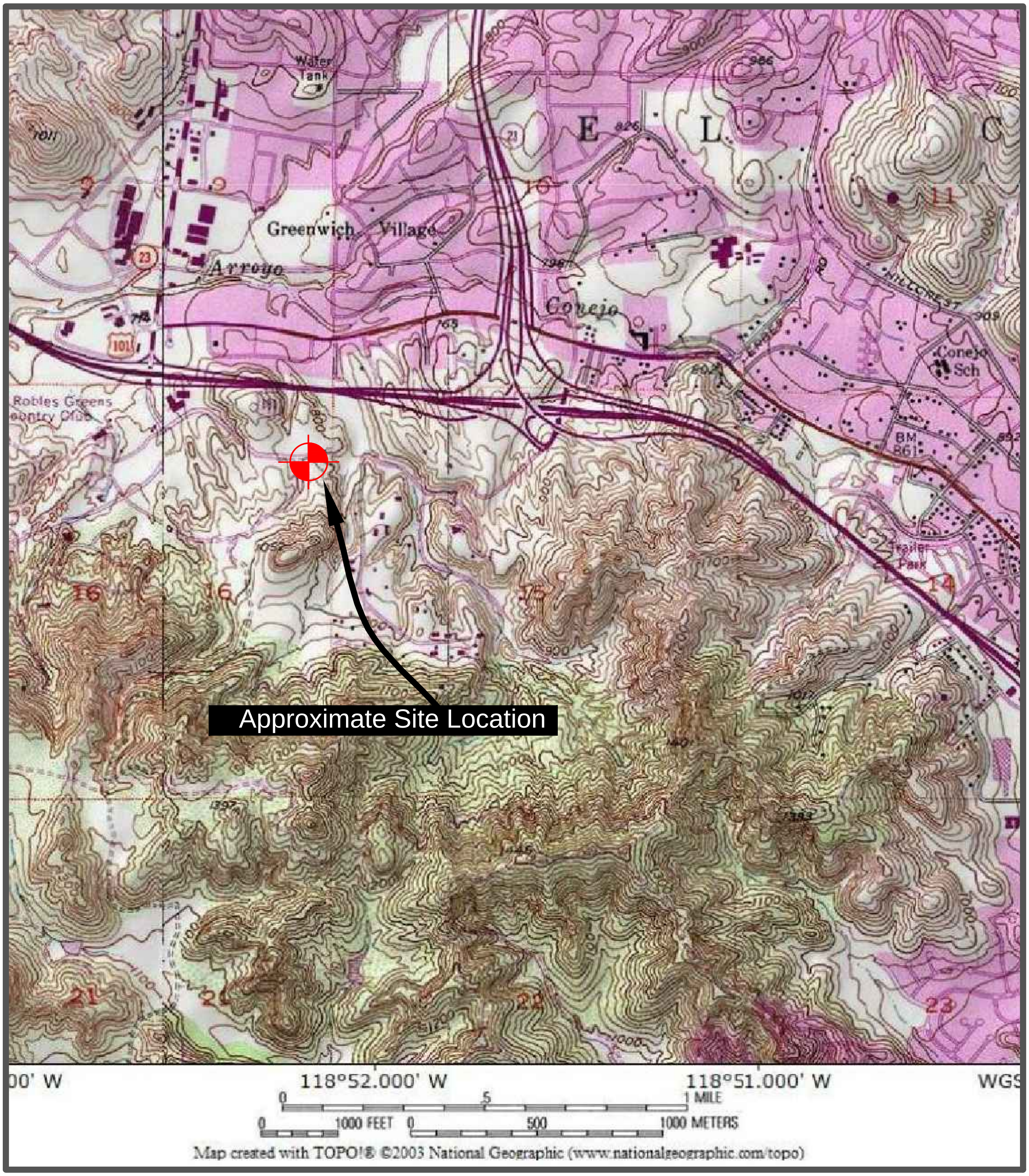
Yerkes, R.F., and P.K. Showalter, 1991. *Preliminary Geologic Map of Thousand Oaks 7.5-minute Quadrangle, Southern California*: in Blake, T.F., and Larson, R.A., eds., *Engineering Geology Along the Simi-Santa Rosa Fault System and Adjacent Areas, Simi Valley to Camarillo, Ventura County, California*: Association of Engineering Geologists, Southern California Section, Field Trip Guidebook, volume 1, pp. 80-86, map scale 1:24,000.

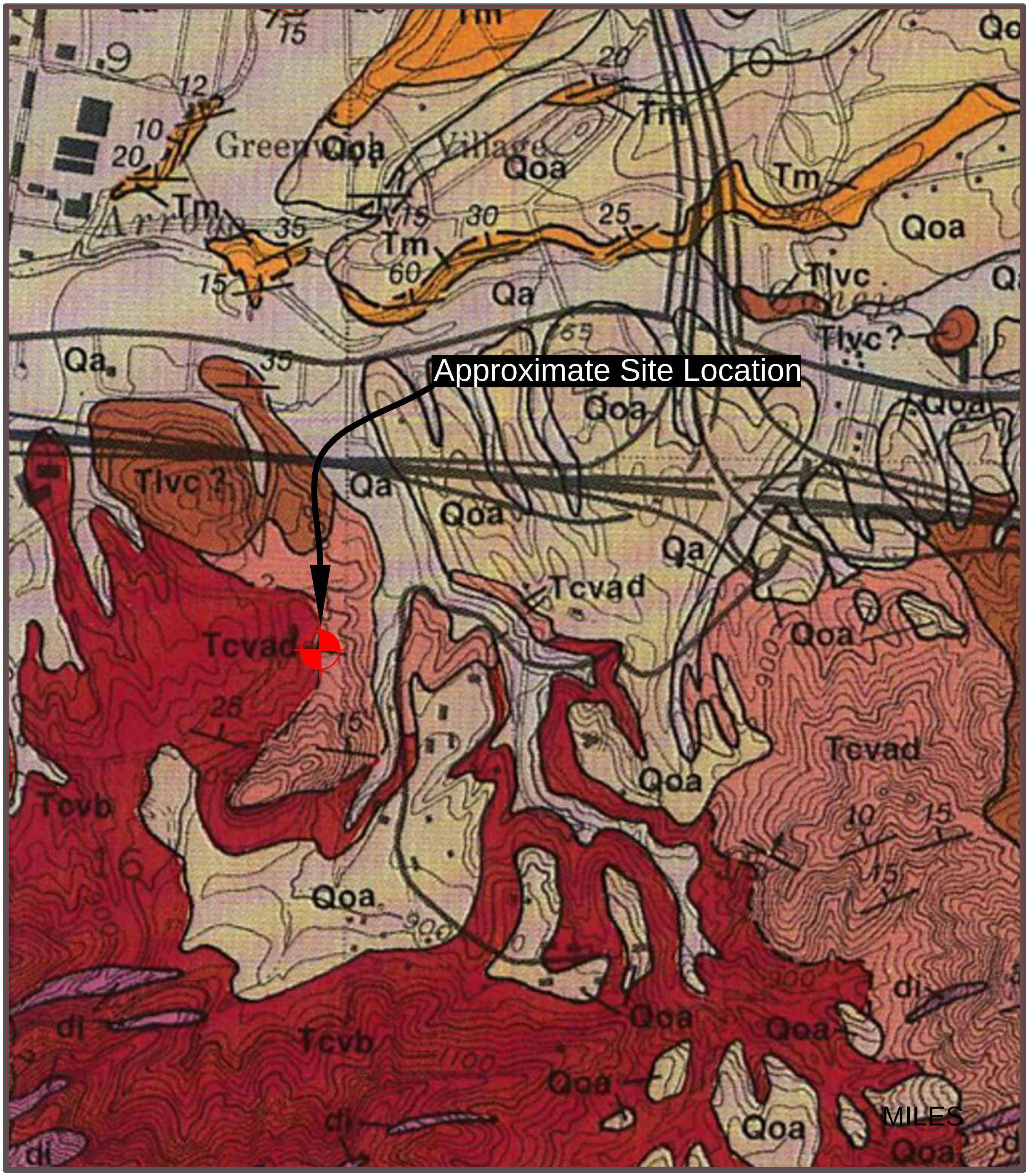


Source
United States Geological Survey, *Thousand Oaks Quadrangle, California-Ventura County*, 7.5 Minute Series (Topographic)

SITE LOCATION MAP

400 Rolling Oaks Dr.
Thousand Oaks, California

Gorian & Associates, Inc. *Applied Earth Sciences*		
Job No: 64-0-0-100		Date: April 2020
Scale: 1" = 2000'	Drawn by: Approved by:	Figure 1



Source: Dibblee, Jr., Thomas W. (1992) and Helmut E. Ehrenspeck (1992), *GEOLOGIC MAP OF THE THOUSAND OAKS QUADRANGLE, VENTURA COUNTY, CALIFORNIA,* Dibblee Geology Center Map # DF-49.

Explanation

Tcvad - andesite-dacite breccia of Westlake; light colored, composed of moderately to poorly sorted, mostly cobble-boulder sized angular fragments of very fine grained feldspathic andesite-dacite in semi-coherent, detrial or tuffaceous (?) matrix of same rock; crudely stratified.

Tcvb - basaltic flows and breccias; dark colored, massive to vaguely bedded, incoherent and crumbly where weathered, weakly resistant to erosion; range from basalt to basaltic andesite composed of feldspar and ferromagnesian minerals.

REGIONAL GEOLOGIC MAP

400 Rolling Oaks Drive
Thousand Oaks, California

Gorian & Associates, Inc. *Applied Earth Sciences*		
Job No: 64-0-0-100		Date: April 2020
Scale:1" = 1000'	Drawn by: Approved by:	Figure 2

APPENDIX A

LOGS OF SUBSURFACE DATA

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

GORIAN & ASSOCIATES, INC.

SUBSURFACE LOG

Excavation Number: B-1

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						SM		ARTIFICIAL FILL: Brown silty fine SAND (moist, loose).	
			1/3/4			SC		Yellowish brown clayey fine to coarse SAND, some fine to coarse gravels (moist, loose).	
5			14/40/45			CL		OLDER ALLUVIUIM: Pale brown sandy silty CLAY (damp, hard). Few fine gravels.	
			14/4"					CONEJO VOLCANICS: Dark gray fine-grained BASALT (damp, indurated.)	
10								Total Depth 8'4" No Caving Observed No Groundwater Encountered BORING DEVELOPED AS INFILTRATION TEST WELL; -8' to -7'; medium bentonite chips -7' to -2'; Slotted 2" (0.002) PVC pipe ANNULAR SPACE -7' to 1.5'; #3 Sand -1.5' to -0.5'; medium bentonite chips Native soil on top Presoaked	
15									
20									
25									
30									
35									

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

SUBSURFACE LOG

Excavation Number: B-2

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						P		ASPHALTICE CONCRETE (3") ON AGGREGATE BASE (7") (Damp, dense).	
						SC		ARTIFICIAL FILL: Yellowish brown clayey fine to coarse SAND, some fine to coarse gravels (damp, dense).	
								CONEJO VOLCANICS: Dark gray mottled with red fine-grained BASALT (damp, indurated.) Heavy chatter.	
5									
								Total Depth 7' (Refusal) No Caving Observed No Groundwater Encountered Backfilled with cuttings	
10									
15									
20									
25									
30									
35									

GORIAN & ASSOCIATES, INC.

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

SUBSURFACE LOG

Excavation Number: B-3

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0								ASPHALTICE CONCRETE (2") ON AGGREGATE BASE (4") (Damp, dense).	
			12/ 15/ 20			SC		ARTIFICIAL FILL: Brown clayey fine to coarse SAND, some fine to coarse gravels (damp, dense).	
5			6/ 17/ 13						
			19/ 45+			SC		Gray clayey fine to coarse SAND, some fine to coarse gravels (moist, dense).	
								CONEJO VOLCANICS: Red fine-grained BASALT (damp, indurated.)	
10								Total Depth 9' No Caving Observed No Groundwater Encountered BORING DEVELOPED AS INFILTRATION TEST WELL; -7.5' to -6.5'; medium bentonite chips -6.5' to -1.5'; Slotted 2" (0.002) PVC pipe ANNULAR SPACE -6.5' to 1'; #3 Sand -1' to G.S.; medium bentonite chips Presoaked	
15									
20									
25									
30									
35									

GORIAN & ASSOCIATES, INC.

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

SUBSURFACE LOG

Excavation Number: B-4

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						P		ASPHALTICE CONCRETE (2") ON AGGREGATE BASE (5") (Damp, dense).	
						SC		ARTIFICIAL FILL: Brown clayey fine to coarse SAND, some fine to coarse gravels (damp, dense).	
5			10/ 17/ 17						
								CONEJO VOLCANICS: Red to dark gray mottled with red fine-grained BASALT (damp, indurated.)	
10			20						
								Yellowish brown fine-grained BASALT (damp, indurated). At 14'; heavy chatter.	
15								Total Depth 14' (Refusal) No Caving Observed No Groundwater Encountered Backfilled with cuttings A.C. Cold Patch on top	
20									
25									
30									
35									

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

GORIAN & ASSOCIATES, INC.

SUBSURFACE LOG

Excavation Number: B-5

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						CL		ARTIFICIAL FILL: Brown sandy silty CLAY (wet, soft). Mottled.	
			2	--	--			CONEJO VOLCANICS: Brown mottled with green fine-grained BASALT, some manganese oxide staining (damp, indurated.) Difficult drilling 2' to 3'.	
								Total Depth 3' (Refusal) No Caving Observed No Groundwater Encountered	
5									
10									
15									
20									
25									
30									
35									

GORIAN & ASSOCIATES, INC.

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

SUBSURFACE LOG

Excavation Number: B-6

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						SC		ARTIFICIAL FILL: Brown clayey SAND (wet, loose). Mottled.	
	■	☒	20/ 8"	-- --	-- --			CONEJO VOLCANICS: Red mottled with green fine-grained BASALT (damp, indurated.) Difficult drilling.	
								Total Depth 2½' (Refusal) No Caving Observed No Groundwater Encountered Backfilled with cuttings	
5									
10									
15									
20									
25									
30									
35									

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

Work Order: 64-0-0-100

SUBSURFACE LOG

Excavation Number: B-7

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						CL		ARTIFICIAL FILL: Brown sandy CLAY (wet, soft).	
			11/ 4"	--	--			CONEJO VOLCANICS: Red mottled with green fine-grained BASALT (damp, indurated.) Difficult drilling.	
5								Total Depth 3' (Refusal) No Caving Observed No Groundwater Encountered Backfilled with cuttings	
10									
15									
20									
25									
30									
35									

Project: HCA, 400 East Rolling Oaks Dr., Thousand Oaks

SUBSURFACE LOG

Work Order: 64-0-0-100

Excavation Number: B-8

Page Number: 1

Date(s) Excavated 3/20/20	Logged By CHD	Excavation Location See Geotechnical Map	Approximate Surface Elevation
Excavation Dimension 8" Dia.	Equipment Contractor Choice Drilling	Equipment Type CME 85	Hammer Data 140#, Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0						SC/G		ARTIFICIAL FILL: Dark brown clayey fine SAND and GRAVEL (moist, loose).	
								CONEJO VOLCANICS: Dark gray fine-grained BASALT (damp, indurated.) Difficult drilling.	
								Total Depth 1½' (Refusal) No Caving Observed No Groundwater Encountered Backfilled with cuttings	
5									
10									
15									
20									
25									
30									
35									

APPENDIX B

LABORATORY TESTING

General
A series of laboratory tests were conducted on selected relatively undisturbed and bulk samples. The tests were performed to evaluate physical and engineering properties of the encountered earth materials. Test procedures and results are described below.

Maximum Density-Optimum Moisture
Three maximum density/optimum moisture tests (compaction characteristics) were performed on selected bulk samples of the soils encountered. The tests were performed in general accordance with ASTM test method D 1557. The results are as follows: The test results from Calwest, 2016 are attached in this appendix.

Boring Number	Depth (feet)	Visual Classification	Maximum Dry Density – pcf	Optimum Moisture Content - %
B-7	0-1	Fill, brown sandy clay.	115.2	13.6

Soil Expansion Test
A soil expansion index test was performed on a selected bulk sample of the upper soils in general accordance with ASTM test method D4829. The results are as follows:

Sample	Expansion Index	Expansion Index Range	Expansion Potential
B-7 @ 0-1'	87	51-90	Moderate Expansion

Direct Shear Test
Direct shear testing was performed on a remolded sample of the earth materials encountered during our exploratory program. The sample set was saturated prior to shearing under axial loads ranging from 920 to 3,680 pounds per square foot at a rate of 0.02 inches per minute. The shear strength results are attached as a graphic summary.

Soil Analysis Lab Results

Client: Gorian & Associates, Inc.
Job Name: Rolling Oaks
Client Job Number: 64-0-0-100
Project X Job Number: S200326B
March 30, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Flouride** F_2^-	**Phosphate** PO_4^{3-}
						As Rec'd	**Minimum**												
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ohm-cm)**	**(Ohm-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B-7	0.0-1.0	23.0	0.0023	7.7	0.0008	737	737	7.9	361.0	3.6	0.3	ND	ND	104.1	0.7	71.6	197.1	6.4	1.6

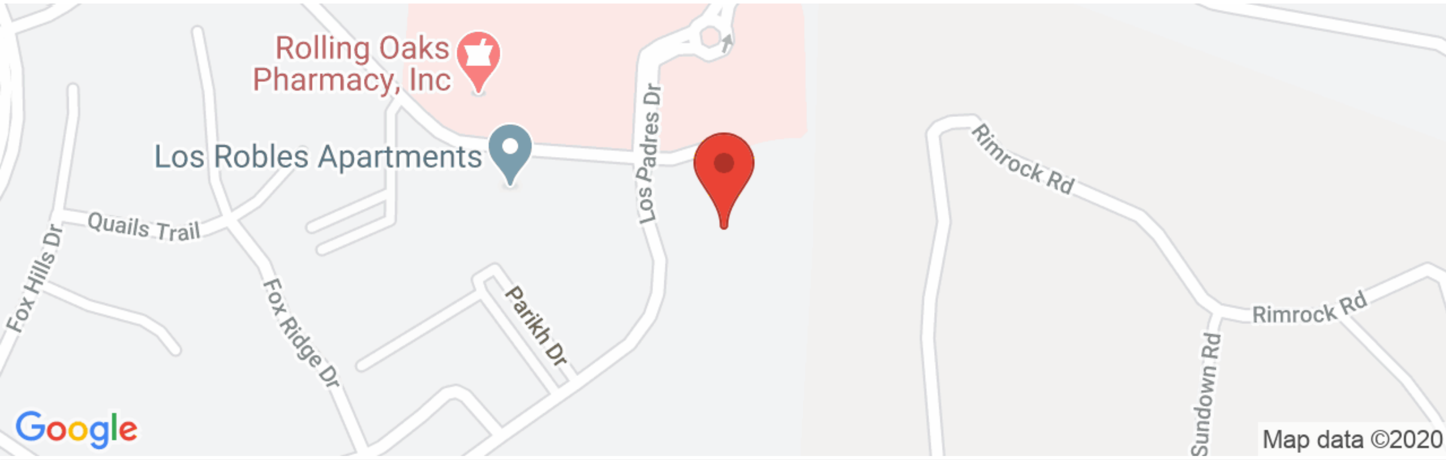
Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract

APPENDIX
c

DESIGN MAPS DETAIL REPORT

HCA Medical Office Building

Latitude, Longitude: 34.1737, -118.8692



Date	4/21/2020, 12:01:12 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.45	MCE_R ground motion. (for 0.2 second period)
S_1	0.519	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.74	Site-modified spectral acceleration value
S_{M1}	0.769	Site-modified spectral acceleration value
S_{DS}	1.16	Numeric seismic design value at 0.2 second SA
S_{D1}	0.512	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.481	Site amplification factor at 1.0 second
PGA	0.504	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.605	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.45	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.579	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.519	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.569	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.504	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.919	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s

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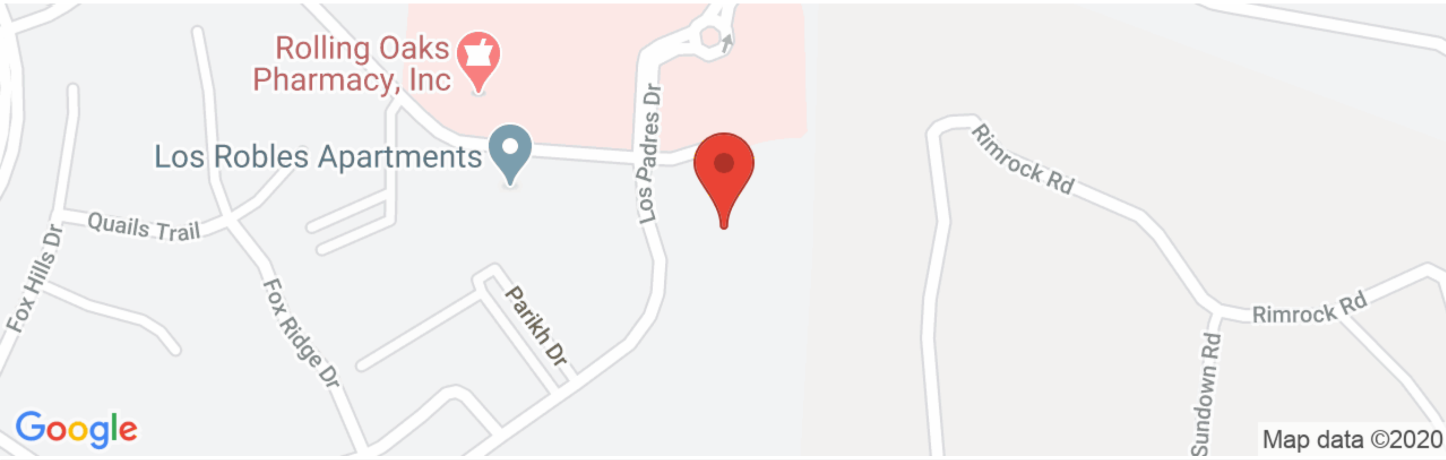
HCA Medical Office Building

Latitude, Longitude: 34.1737, -118.8692



Date	4/21/2020, 12:01:12 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
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S_{MS}	1.74	Site-modified spectral acceleration value
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Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.481	Site amplification factor at 1.0 second
PGA	0.504	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.605	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.45	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.579	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.519	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.569	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.504	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.919	Mapped value of the risk coefficient at short periods
C_{R1}	0.912	Mapped value of the risk coefficient at a period of 1 s

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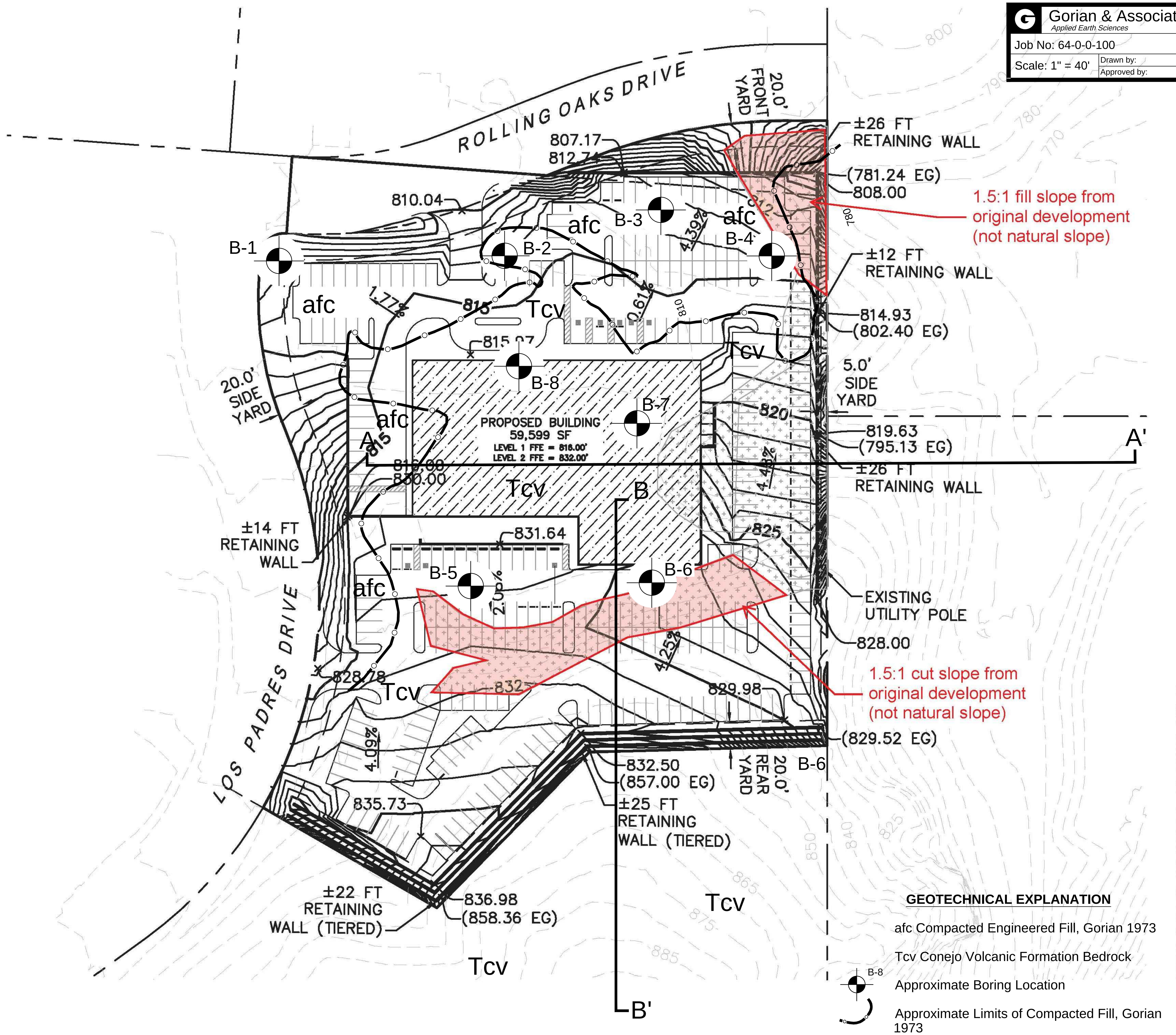
GEOTECHNICAL MAP

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 64-0-0-100		Date: April 2020
Scale: 1" = 40'	Drawn by: Approved by:	PLATE 1



XXX.XX — PROPOSED GRADE

(XXX.XX EG) — EXISTING GRADE

ESTIMATED ONSITE NATURAL SLOPES GREATER THAN 25% (0.75–1.25 ACRES)

EARTHWORK ESTIMATE:

CUT = 21,000 CY

FILL = 29,000 CY

NET = 8,000 CY (IMPORT)

NOTE: THE ABOVE QUANTITIES ARE APPROXIMATE IN PLACE VOLUMES CALCULATED FROM THE EXISTING GROUND TO THE PROPOSED FINISHED GRADE. EXISTING GROUND IS DEFINED BY THE CONTOURS AND SPOT GRADES ON THE BASE SURVEY. PROPOSED FINISHED GRADE IS DEFINED AS THE FINAL GRADE AS INDICATED ON THE GRADING PLAN(S).

THE EARTHWORK QUANTITIES ABOVE ARE FOR PERMIT PURPOSES ONLY. THEY HAVE NOT BEEN FACTORED TO ACCOUNT FOR CHANGES IN VOLUME DUE TO BULKING, CLEARING AND GRUBBING, SHRINKAGE, OVER- EXCAVATION AND RE-COMPACTION, AND CONSTRUCTION METHODS. NOR DO THEY ACCOUNT FOR THE THICKNESS OF PAVEMENT SECTIONS, FOOTINGS, SLABS, REUSE OF PULVERIZED MATERIALS THAT WILL UNDERLIE NEW PAVEMENTS, ETC. THE CONTRACTOR SHALL RELY ON THEIR OWN EARTHWORK ESTIMATES FOR BIDDING PURPOSES.

GEOTECHNICAL EXPLANATION

afc Compacted Engineered Fill, Gorian 1973

Tcv Conejo Volcanic Formation Bedrock

B-8 Approximate Boring Location

Approximate Limits of Compacted Fill, Gorian 1973

No.	REVISIONS	DATE	BY

Kimley»Horn

660 SOUTH FIGUEROA STREET, SUITE 2050
LOS ANGELES, CA 90017
PHONE: 213-261-4040
WWW.KIMLEY-HORN.COM

KHA PROJECT 099691002
DATE 3/5/2020
SCALE AS SHOWN
DESIGNED BY
DRAWN BY
CHECKED BY

CONCEPTUAL GRADING PLAN

LOS ROBLES CANCER CENTER
400 ROLLING OAKS DRIVE
PREPARED FOR HCA
THOUSAND OAKS CA

SHEET NUMBER
EX-02

GEOTECHNICAL CROSS SECTIONS

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 64-0-0-100	Date: April 2020
Scale: 1" = 20'	Drawn by: Approved by:
	PLATE 2