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GEOTECHNICAL EVALUATION

GEOTECHNICAL
PROFESSIONALS INC.

UPDATED GEOTECHNICAL EVALUATION REPORT FOR CEQA
PROPOSED RETAIL/RESTAURANT AND RESIDENTIAL BUILDINGS
THE COMMONS AT CALABASAS
4799 COMMONS WAY
CALABASAS, CALIFORNIA

Prepared for:
THE COMMONS AT CALABASAS, LLC
c/o CARUSO
101 The Grove Drive
Los Angeles, CA 90036

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211
info@gpi-ca.com

Project No. 3063.I

August 14, 2023

GEOTECHNICAL
PROFESSIONALS INC.

August 14, 2023

The Commons at Calabasas, LLC
c/o Caruso
101 The Grove Drive
Los Angeles, CA 90036

Attention: Tasha Reeder
Project Manager, Construction

Subject: Updated Geotechnical Evaluation Report for CEQA
Proposed Retail/Restaurant and Residential Buildings
The Commons at Calabasas
4799 Commons way
Calabasas, California
GPI Project No. 3063.I

Dear Tasha:

Transmitted herewith is our updated geotechnical evaluation report for CEQA for the subject project. The report presents the results of our evaluation of the geologic and seismic hazards at the subject site. This updated report addresses comments provided by the City of Calabasas in their letter dated July 12, 2023.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have any questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Dylan Boyle, G.E.
Senior Engineer
(dylanb@gpi-ca.com)

Justin J. Kempton, G.E.
Principal
(justink@gpi-ca.com)

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1.0 INTRODUCTION

1.1 GENERAL

This updated report presents the results of a geologic/seismic hazards study performed by Geotechnical Professionals Inc. (GPI) for the proposed mixed-use retail/restaurant and residential buildings planned at the Commons at Calabasas center in Calabasas, California. This includes addressing the geologic and seismic related issues listed in the attached Environmental Checklist Form of the State California Environmental Quality Act CEQA Guidelines. This updated report supersedes our May 26, 2023 report and addresses comments provided by the City of Calabasas in their letter dated July 12, 2023. The location of the site is shown on Figure 1, Site Location Map. The layout of the site with proposed and existing conditions is shown on Figures 2 and 3, Site Plans.

GPI previously conducted design-level geotechnical investigations for the proposed retail/restaurant and residential buildings and presented the results in reports dated December 22, 2022 (GPI, 2022a) and December 23, 2022 (GPI, 2022b), respectively. This study includes review of recent subsurface explorations and data from the referenced geotechnical investigations and other available subsurface and geologic information, and engineering and geologic analyses.

1.2 PROJECT DESCRIPTION

We understand the proposed project will consist of four new retail and restaurant buildings and a new 8-story residential building within the Commons at Calabasas center. The center is surrounded by Calabasas Road to the north, Park Granada to the east, a slope ascending up to Park Granada to the south, and the Calabasas City Hall and Library to the west.

The residential building will be approximately 43,500 feet in plan and will be constructed within the general location of the existing movie theatre building. The building will be a podium style structure including 5 stories of residential over 2 stories of above-grade parking and one subterranean parking level. In general, the proposed finish floor elevation (FFE) of the at-grade parking level of the residential building is anticipated to be near the same elevation as the existing building and the lowest parking level is anticipated to extend on the order of 10 to 15 feet below surrounding grades. Site grading for the proposed residential building will occur within the existing developed area and is anticipated to include cuts up to approximately 15 feet and fills up to 3 feet. The existing reinforced concrete cantilever retaining wall that provides a grade separation between the walkway behind the theatre building and the southern access drive for the center will remain. Final design will include a horizontal separation between the building wall and the retaining wall to allow for seismic movement.

The retail/restaurant buildings will be constructed within the existing parking lot north of the existing movie theatre. These buildings will range from approximately 2,000 to 11,000 square feet in plan, be single- to 3-level structures, and be supported at-grade or underlain by a single-level-subterranean parking garage that will extend up to 15 feet below existing grades. Retail and restaurant space is planned at the first floor for each building and residential apartments and amenities are planned for levels 2 and 3. Site grading for the proposed retail/restaurant buildings is anticipated to include cuts up to approximately 15 feet and fills up to 10 feet.

The project will also include parking lot improvements, minor site walls, pedestrian hardscape, and landscaping in the remainder of the project area.

2.0 SCOPE OF WORK

Our scope of work included review of existing subsurface exploration and laboratory data, in-house geotechnical reports and those made available to us, readily available subsurface and geologic data from previous geotechnical reports by others, on-line open file geologic hazards reports, geology maps, vintage stereoscopic aerial photographs, and groundwater data, and preparation of this report. This report presents the results of our study to address potential geotechnical and geologic/seismic hazards for the development as outlined in the previously described CEQA Guidelines.

We previously performed a field exploration program and provided two comprehensive geotechnical investigation reports (retail and residential projects) that provide more detailed analysis and geotechnical recommendations for design and construction (GPI, 2022a and 2022b). Our field exploration program for the overall development consisted of seven hollow stem auger borings performed to depths of approximately 21 to 51 feet below existing grades. The locations of the subsurface explorations for the development are shown on Figures 2 and 3. Details and the results from the field exploration and laboratory testing programs are presented in the referenced geotechnical investigation reports (GPI, 2022a and 2022b).

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 EXISTING SITE CONDITIONS

The project site is located within the existing movie theatre building (Building I) located in the southwest corner of the Commons at Calabasas shopping center and within the existing parking lot north of the existing movie theatre and adjacent retail buildings. On the south side of Building I, there is an existing reinforced concrete cantilever retaining wall that provides the grade separation between a walkway, approximately 6 to 8 feet wide, immediately south of Building I and the southern access drive for the center. The center is surrounded by Calabasas Road to the north, Park Granada to the east, a slope ascending up to Park Granada to the south, and the Calabasas City Hall and Library to the west. The parking lot consists predominately of asphalt drives and parking stalls with portland concrete cement curbs and gutters and landscaping.

The proposed retail/restaurant building area generally slopes downward gradually from the southwest to the northeast, with a change in ground surface elevation from about Elevation +971 feet to +958 feet across the site. The floor slab grades within the existing Building I, the main entrance, landscape, and the walkways around the building range from approximately Elevation +971 to +972 feet. The finish floor elevations (FFEs) of adjacent buildings are reportedly at +971.2 to +972.7 feet based on the 1998 rough grading plans. The site grades within the southern access drive south of the existing theatre building range from approximately Elevation +983 to +993 feet (approximately 12 to 21 feet above the approximate FFE of the movie theatre building and adjacent buildings.

The City of Calabasas has a Driveway Easement within the southern access drive and numerous underground utilities are located within the access drive. South of the southern access drive (approximately 40+ feet behind the existing movies theatre and adjacent buildings to the east), an existing soldier pile with tieback retaining wall is supporting a vegetated slope that ascends to Park Granada. The height of the tieback wall varies from no wall/zero feet on the west to approximately 45 feet on the east.

Based on the Rough Grading Plans for the center dated March 2, 1998, prior discussion with Caruso representatives, and review of a geotechnical evaluation report by Kleinfelder dated July 2, 2009 (Kleinfelder, 2009), we understand grading of the overall site was completed in 1998 and construction of the adjacent buildings was completed in 1999. We understand the soldier pile with tieback wall was constructed before grading of the development in 1998.

3.2 SUBSURFACE MATERIALS

Our field investigation disclosed a subsurface profile consisting of shallow fill soils overlying natural bedrock. Detailed descriptions of the conditions encountered, including logs of our explorations, are presented in the referenced Geotechnical Investigation Reports (GPI, 2022a and 2022b) for the project site.

We encountered undocumented fills with varying depths across the site from about two feet below existing grades in the existing parking lot north of the existing theatre building and up to about 7 feet below existing grade in the borings drilled within the southern access road to the south of the existing movie theatre building. The fill materials encountered consisted of slightly moist to moist silty and sandy clays. The fill materials are considered undocumented because documentation of the fill has not been made available for our review. It is likely these fills were placed during original grading of the center around 1998 and during backfill of the cantilever retaining wall south of the existing movie theatre building. Expansion index testing on representative samples of the sandy clay soils indicates the materials have a low to medium potential for expansion.

The underlying natural materials encountered consisted of hard, moist to very moist, siltstone bedrock to the depths explored. The bedrock is mapped as the Modelo Formation. The bedrock materials have moderate to high strength and low compressibility characteristics. Expansion Index testing on a representative remolded sample of the bedrock indicates the materials (when remolded) have a medium potential for expansion. Prior testing of the bedrock materials (Kleinfelder, 2017) concluded that samples of the bedrock materials tested contained over 2 percent pyrite (iron sulfide). Oxidation of pyrite minerals present in the bedrock will form gypsum crystals within the exposed bedrock fractures and surfaces that can result in ground expansion. Additional details on the potential for heave of the bedrock are provided in the previously referenced geotechnical investigation reports (GPI, 2022a and 2022b).

4.0 GEOLOGIC CONDITIONS

4.1 REGIONAL GEOLOGY

The subject site is located in the southwestern end of the San Fernando Valley, a broadly alluviated basin filled with Tertiary age marine sedimentary rocks at depth and mantled by Recent and Pleistocene age non-marine alluvial sediments deposited by washes and streams flowing from adjacent hillside areas, including the Santa Monica Mountains to the south, Santa Susana Mountains to the northwest, Simi Hills to the west, and San Gabriel Mountains to the northeast.

Regionally the site is located north of the border between two of California's geomorphic provinces, the Transverse ranges to the north and the Peninsular Ranges to the south. The active Santa Monica Fault, located approximately 15½ kilometers south of the site, forms the boundary of the two geomorphic provinces. The Santa Monica Fault is part of an east-west trending active fault complex termed the Santa Monica-Hollywood-Raymond Fault System, which also includes the Malibu Coast and Anacapa Dume Faults, located in the western portion of the fault complex. This fault system generally forms the southern boundary of the Santa Monica and San Gabriel Mountains north of the fault system, and the Los Angeles Basin south of the fault system.

The Transverse Ranges are characterized by east-west trending mountain ranges, including the Santa Monica and San Gabriel Mountains, which are oriented oblique to the trend of the other major structural trends in California, including the San Andreas Fault, Sierra Nevada Mountains, and other mountain ranges in Southern California, which trend northwesterly.

The Peninsular Ranges are characterized by northwesterly trending active faults and mountain ranges related to the San Andreas and other major fault systems in the province. The province extends from the Santa Monica-Hollywood-Raymond Fault System, within the Los Angeles Basin, southeast to Baja California.

More specifically, the site is located at the southwestern end of the San Fernando Valley, an east-west-trending structural trough whose origin is closely related to uplift and deformation of the San Gabriel mountain range to the north. As the range has been elevated and deformed as a result of crustal shortening during Cenozoic time, the San Fernando Valley has subsided and become filled with sediment.

4.2 SITE GEOLOGY

Prior to grading at the site around 1998, the subject site consisted of low relief hillsides sloping northeasterly into the San Fernando Valley and towards the alignment of U.S. Route 101. Much of the site is located within a northeasterly trending, low relief ridgeline generally underlain by sedimentary bedrock of the Modelo Formation consisting of shale and sandstone (per Weber, 1984; equivalent to Sisquoc Shale by Dibblee, 1989) and, further up the ridgeline to the southwest, underlain by Monterey Formation shale and Upper Topanga Formation sandstone. More specifically, within the limits of the subject site, the underlying bedrock is mapped as consisting of claystone and siltstone, moderately to vaguely bedded and crumbly where weathered (Tush; mapped by Dibblee, 1992).

During prior grading related to the current site development, cuts on the order of 50 feet were performed and the remaining low-relief hillside was retained via a soldier pile with tieback anchor supported retaining wall. Prior grading within the limits of the subject site resulted in manmade fills on the order of 7 feet (where explored). Documentation of this grading was not readily available.

As encountered in our explorations to depths of 51 feet, and as characterized by others in prior investigations at the site, the subsurface profile primarily consists of fills overlying formational bedrock materials, with localized deposits of alluvial soils encountered between the fill soils and bedrock. The fills consisted primarily of silty and sandy clays, likely generated from the underlying siltstone and claystone bedrock. Fill depths extended up to 7 feet below existing site grades in our exploration located at the southwestern edge of the site, within an existing access road. Outside of this area, our explorations encountered undocumented fills on the order of 2 feet deep.

The underlying natural materials consisted of bedrock of the Modelo Formation [Unnamed Shale (Tush) of Dibblee], comprised of friable siltstone with trace amounts of sandstone. The Modelo Formation is a marine, biogenic, and clastic deposit, moderately to vaguely bedded. Subsurface testing of the bedrock materials indicates they are generally very hard when using soil consistency terminology. The geologic conditions in the site area are shown on Figure 4, Regional Geologic Map. Detailed logs of the subsurface conditions encountered in our explorations were presented in our referenced geotechnical investigation reports (GPI, 2022a and 2022b).

4.3 GROUNDWATER

Historical data provided by the California Geological Survey (CGS, 1997) does not provide a clear indication of the shallowest groundwater depth in the site vicinity due to a lack of data points. The nearest mapped groundwater level contour indicates a historical shallowest depth to groundwater of approximately 10 feet below prevailing site grades in a drainage channel located roughly ¼ mile southeast of the site. Based on ground surface elevations obtained from internet sources (Google Earth), this drainage course is at roughly the same elevation as the subject site.

Groundwater was not encountered in our recent explorations up to a depth of 51 feet below existing grades. However, moist to very moist bedrock materials were encountered as shallow as 2 feet below existing grades. Groundwater was observed in prior explorations by others at the site (Kleinfelder, 2017) at depths of 3 to 9½ feet below existing grades at the end of drilling. These depths were below the fill/bedrock contact. Details of the groundwater depths in the vicinity of the site are shown on the Historical High Groundwater Map, Figure 5.

4.4 TECTONIC SETTING

4.4.1 Regional Fault Systems

The geologic structure of southern California is dominated by northwest trending faults associated with the San Andreas Fault System. Faults such as the Newport-

Inglewood, Whittier, Palos Verdes Hills, and San Jacinto are considered active and are associated with the San Andreas, collectively forming the boundary between the North American and Pacific tectonic plates. Most of these faults have ruptured the ground surface historically and/or produced significant earthquakes.

Anomalous to the general northwest structural fabric are a series of active east-west trending reverse or thrust faults. The majority of these occur as north dipping planes projecting along the southern base of the Santa Monica and San Gabriel Mountains in the greater Los Angeles area. The known active thrust faults in the region include the Cucamonga, Sierra Madre, San Fernando, Raymond, Santa Monica, and Hollywood faults.

4.4.2 Concealed Faults

Another category of fault known as "blind thrusts" was recognized as a significant seismic hazard following the 1987 magnitude 6.0 Whittier Narrows Earthquake and then again by the 1994 San Fernando magnitude 6.7 Earthquake. A blind thrust is a deeply buried, shallow dipping thrust fault, which does not project to the ground surface. Blind thrusts are capable of generating a major earthquake that may cause uplift in the form of anticlinal hills. Some uplands that surround the Los Angeles Basin, including Elysian Park-Repetto Hills area, are products of blind thrusts. Because blind thrusts do not intersect the ground surface, primary surface fault rupture is considered unlikely as a potential hazard. Major portions of the Los Angeles Basin are now believed to be underlain by various blind thrust ramps. Due to continued north-south convergence (shortening) across the Los Angeles Basin, slippage along these features will generate future earthquakes.

At the present time, the potential magnitudes and recurrence intervals of blind thrust produced earthquakes cannot be quantified with confidence due to the fact that many characteristics of these features (including areal extent and Quaternary slip rates) are poorly understood. Nonetheless, the proximity to densely populated urban centers and their history of producing damaging earthquakes clearly demonstrate the risk that blind thrusts pose to large metropolitan areas and surrounding cities.

4.4.3 Nearby Seismogenic Sources

The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (CGS, 1998). In addition, named surface faults are not mapped projecting towards or through the site. The site is therefore not subject to surface fault rupture hazard from an active fault.

We reviewed the 2008 National Seismic Hazard Maps Source Parameters (USGS, 2008) to identify known active faults within a 100-mile radius of the project site which could produce ground motion related hazards to the site. The names and distances of the faults lying within 25 miles of the project site are provided in the following table (Table 4.4-1). We present a map showing the significant regional faults in Figure 6, Regional Fault Map.

Table 4.4-1 – Significant Regional Faults

Fault Name	Approximate Distance* (mi)
Malibu Coast	7.9
Santa Monica	9.7
Anacapa-Dume	10.4
Simi-Santa Rosa	10.8
Santa Susana	11.6
Palos Verdes	13.6
Sierra Madre (San Fernando)	14.1
Hollywood	14.5
Verdugo	14.9
Northridge	15.0
Newport-Inglewood	16.5
Oak Ridge	17.5
San Gabriel	18.9
Holser	19.0
Puente Hills (Los Angeles)	19.6
Elysian Park (Upper)	20.2
San Cayetano	20.3
Sierra Madre	22.0
Raymond	24.3

* Defined as the closest distance to projection of rupture area along fault trace.

Brief details for some of the faults closest to the subject site are as follows:

<u>Hollywood and Santa Monica Faults</u>

The Hollywood and Santa Monica Faults comprise the western and central portions of the Santa Monica-Hollywood-Raymond fault system, a generally east-west trending series of oblique, reverse and left-lateral strike-slip faults, which also includes the Malibu Coast and Anacapa Dume faults in the western portion of the fault system. The faults are mapped along the foot of the southern flank of the east-west trending Santa Monica Mountains approximately 9.7 kilometers to the south of the site at closest approach. Mapping of the feature indicates the faults have a length projecting from the coast eastward to the Los Angeles River channel. The faults have been studied by several groups including Dr. Kerry Sieh at CIT (1993). Locations of the faults are poorly constrained in the field due to alluvial cover and urban development. The faults are believed to be high angle, north dipping thrust faults and have been partly responsible for uplift of the Santa Monica Mountains. Carbon dating methods indicate the faults have moved at least once between 8,000 to 17,000 years ago, which places in into a likely active category. No significant historic earthquakes have been associated with the faults. The faults are capable of producing a moment magnitude (Mw) 6.5 earthquake, and perhaps larger if coupled with simultaneous movement on an adjacent fault. Dolan et al. (2000) dated the most recent surface rupture of the Hollywood fault at between about 6,000 and 11,000 years ago, with a possible earlier surface rupture about 22,000 years ago, indicating a relatively long recurrence interval between surface rupture events.

Newport-Inglewood Fault

The Newport-Inglewood Fault forms the southwesterly side of the Los Angeles Basin and is defined by a series of low disconnected hills and mesa surfaces. Strike slip faulting is associated with anticlinal folding. This has resulted in the accumulation of petroleum resources along its entire length from offshore Newport Beach to the Santa Monica Mountains. In 1933 the destructive Long Beach Earthquake occurred on the fault just offshore of Newport Beach. The event caused considerable damage and a high loss of life. Since then, the various strands of the fault have produced many minor earthquakes, all of which have been at a magnitude of 4.5 or less. The fault lies at a distance of approximately 14.0 kilometers to the southwest of the project sites at its closest approach. A maximum earthquake magnitude of 6.9 and slip rate of 1.0 mm/yr has been assigned to the fault.

Santa Susana Fault

The Santa Susana fault extends 28 kilometers west-northwest from the northwest edge of the San Fernando Valley into Ventura County and is at the surface on the south flank of the Santa Monica Mountains. The Pico Canyon earthquake of April 4, 1893, of M 5.5-5.9 (Toppozada, 1995) which might have occurred on the Santa Susana fault, caused damage in Newhall, Saugus, and Castaic (Richter, 1973), in addition to Los Angeles, Pasadena, and Fillmore.

The total dip-slip displacement on the Santa Susana fault is based on the offset of the Fernando formation (Huftile and Yeats,1996). The displacement is 4.9 to 5.9 kilometers, resulting in a dip-slip rate of 2.1 to 9.8 mm/yr. The horizontal component of displacement is 4.1 kilometers, with a horizontal shortening rate of 5.7 mm/yr (Huftile and Yeats, 1996).

Simi-Santa Rosa Fault

The Simi fault zone is best known from oil exploration; ground water studies have also helped locate the faults, especially western sections. Surface traces are known principally from thesis mapping, later compilations, and recent geotechnical studies, but some sections of the fault zone are still only moderately well located at the surface. Age control for most-recent surface rupture and Holocene fault history is limited to the Springville fault and one site in the middle of the Simi fault. Camarillo and Santa Rosa Valley faults are interpreted principally from geomorphology and subsurface data, with sparse confirmation as surface faults. It is not known if the various faults comprising the zone rupture together or as semi-independent elements and sections or segments have not been previously defined in the literature.

The Simi-Santa Rosa fault zone is dominated by moderate to high-angle north-dipping reverse faults that probably also have a left-lateral component of displacement (Treiman, 1998). The fault zone extends for 40 km in an east-northeast direction within the southern California Transverse Ranges. Simi fault is a Tertiary fault with up to 1,600 m vertical separation (Oligocene Sespe) and continued Quaternary activity (Hanson, 1981). In a westward direction late-Quaternary activity steps left from the Simi across the Santa Rosa, Santa Rosa Valley, and Camarillo fault elements of the zone, and also northwest (right-step) to the Springville fault. Slip rate assigned by Petersen and others (1996) for probabilistic seismic hazard assessment for the State of California was 1.0 mm/yr (with minimum and maximum assigned slip rates of 0.5mm/yr and 1.5 mm/yr, respectively).

4.4.4 Seismic Exposure

As is the case with most locations in Southern California, the subject site is located in a region that is characterized by moderate to high seismic activity. The project site and vicinity has historically experienced strong ground shaking due to earthquakes. The locations of earthquake epicenters with respect to the subject site are shown graphically on Figure 7, Regional Seismicity.

5.0 GEOLOGIC-SEISMIC HAZARDS

5.1 GENERAL

A summary of the requirements of Section VII. Geology and Soils of CEQA Appendix G: Environmental Checklist are presented below and followed by the results of our geologic and seismic hazards evaluation for the proposed development.

5.2 THRESHOLDS OF SIGNIFICANCE

In accordance with guidance provided in Section VII Geology and Soils of Appendix G of the State CEQA Guidelines, the project could have a potentially significant impact if it were to:

(a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:

   i. Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault.

   ii. Strong seismic ground-shaking.

   iii. Seismic-related ground failure, including liquefaction.

   iv. Landslides.

(b) Result in substantial soil erosion or the loss of topsoil.

(c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction, or collapse.

(d) Be located on expansive soil, as identified in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property.

(e) Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater.

(f) Directly or indirectly destroy a unique paleontological resource or site or unique geological feature.

The attachment to this report provides input for Section VI Geology and Soils of the CEQA Appendix G Environmental Checklist Form based on our evaluation of potential geologic and seismic hazards discussed herein. Note that septic tanks/systems will not be used on-site for the proposed project, and GPI's evaluation did not include assessment of paleontological resources.

5.3 SURFACE FAULT RUPTURE

The site does not lie within an Alquist-Priolo (AP) Earthquake Fault Zone as designated by the California Geological Survey (CGS, 1998). Surface faults have not been mapped projecting towards or through the site area. As such, surface ground rupture is considered unlikely at this site.

5.4 SEISMIC GROUND SHAKING

As is the case with most locations in Southern California, the subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. The subject site will likely experience strong ground shaking caused by earthquakes on active, regional faults in the future. The effects of strong seismic ground shaking can be mitigated by design and construction in conformance with current building codes and engineering practices. The project will be designed in accordance with the California Building Code.

5.5 LIQUEFACTION AND SECONDARY EFFECTS

Loosely compacted/deposited granular soils located below the water table can fail through the process of liquefaction during strong earthquake-induced ground shaking. In this process, there is a rapid decrease in shearing resistance of cohesionless soils, caused by a temporary increase in the pore water pressure. Factors known to influence liquefaction potential include soil type and depth, grain size, relative density, ground-water level, degree of saturation, and both intensity and duration of ground shaking.

As a result of liquefaction, a typical building structure may be exposed to several hazards, including liquefaction-induced settlement, foundation bearing failure, and lateral displacement or lateral spreading. The surface manifestation of liquefaction in deeper soil deposits often takes place in the form of sand boils and ground subsidence. Such phenomena often lead to loss of adequate support for building foundations (bearing failures) and cause tilting, excessive movement, and cracking of superstructures. The severity of ground subsidence depends largely on the relative thickness of the surficial non-liquefiable layer compared to the thickness of layers undergoing liquefaction.

According to the published State Seismic Hazard Zones map for the Calabasas Quadrangle, the site is not located in an area designated by the State Geologist as a “zone of required investigation” due to the potential for earthquake-induced liquefaction. In addition, the subsurface materials underlying the proposed site are primarily bedrock materials not subject to liquefaction hazard. As such, the potential for damage due to liquefaction, seismic-induced lateral spreading, and seismically induced settlement is low. Details of the liquefaction zones in the vicinity of the site are shown on Figure 8, Seismic Hazard Zones Map.

5.6 LANDSLIDES

According to the published Earthquake Zones of Required Investigation map for the Calabasas Quadrangle, the southeast portion of the site is located within an Earthquake-Induced Landslide Zone (CGS, 1998). Earthquake-induced landslide zones are identified as areas where previous occurrence of landslide movement, or local topographic, geological, or geotechnical and subsurface water conditions indicate a potential for permanent ground displacements that would be addressed through compliance with the California Building Code with City of Calabasas amendments, the City of Calabasas Municipal Code, and the policies in the General Safety Plan Element. The approximate location of the planned improvements relative to the mapped landslide zone is shown on the attached Figure 8, Seismic Hazards Zones Map.

The seismic hazard zone map, which was published in 1998 and near the time site grading was completed, includes the slope that ascends to the south of the project area. The map also includes topographic contours that predate site development and construction of the soldier pile with tieback anchor wall located south of southern access road for the center. The soldier pile wall is located approximately 40 feet from the existing movie theatre building. The portion of the site within the mapped landslide zone was cut up to approximately 45 feet and the ascending slope is currently retained by the soldier pile with tieback anchor wall. It is our understanding that the soldier pile with tie back anchor wall was designed by others to support the ascending slope and mitigate the potential for slope instability.

A Geotechnical Engineer and Engineering Geologist from GPI performed a site reconnaissance on August 9, 2023 to observe the general conditions of the slope behind the tieback anchor wall in the vicinity of the planned improvements. Adverse slope conditions that would need to be addressed during conceptual design were not observed during the site reconnaissance.

In general, the topography of the project area slopes downward to the north. Site grades on the south side of the existing buildings (directly north of the soldier pile with tieback anchor retaining wall) range from Elevation +983 to +993 feet. Grades of the existing movie theatre and retail building floor slabs north of the southern access drive range from approximately +971 to +973 feet. Within the parking area north of the existing retail buildings, ground surface elevations range from about +958 feet to +971 feet.

Site grading for the proposed buildings will occur within the existing developed area of the center and is anticipated to include cuts up to approximately 15 feet and fills up to 10 feet. Significant new permanent cut or fill slopes are not planned.

Based on prior site grading, planned site grading, construction of the soldier pile with tieback anchor wall, and the presence of near-surface bedrock materials, the potential for seismic-related ground failure due to landsliding for the project is considered to be low.

5.7 TSUNAMIS AND SEICHES

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. The site is located approximately 8 miles inland from the Pacific Ocean at elevations of approximately +958 to +973 feet above mean sea level (based on Google Earth). Due to the distance to the coast and elevation at the site, the probability of flooding due to a tsunami is considered to be nonexistent.

The subject site is located approximately ¼ mile northwest of Calabasas Lake, a privately owned manmade lake with an approximate surface area of 17.8 acres and average depth of roughly 4 feet. The elevation of the lake (based on Google Earth) is approximately +943 feet, roughly 15 to 30 feet below the predominant elevation of the subject site. We did not identify other significant bodies of water in the vicinity of the subject site. Based on the distance and elevation differential between the subject site and Calabasas Lake, the probability of site flooding due to seiche is also considered to be nonexistent.

5.8 EXPANSIVE AND COLLAPSE POTENTIAL

Expansive soils generally consist of clays that can shrink and swell with changes in moisture content. Movement of soils in response to shrinkage and swelling has the potential to impact near-surface improvements such as lightly loaded foundations, floor slabs, and flatwork. Based on the data reviewed, near surface soils are anticipated to have a low to moderate expansion potential when subject to changes in moisture content. In addition, mineralogical testing performed by others during a prior investigation at the site (Kleinfelder, 2017), indicated the bedrock materials at the site contained over 2 percent pyrite (iron sulfide). Oxidation of pyrite minerals present in the bedrock will form gypsum crystals within the exposed bedrock fractures and surfaces that can result in ground expansion (Bryant, 2003).

Based on the above, the potential for expansive soils/bedrock to adversely affect the project if not mitigated is considered to be high. The project design should include design features to reduce the adverse impact of expansive soil/bedrock on the proposed project. Recommendations regarding these design features were provided in the referenced design-level geotechnical reports (GPI, 2022a and 2022b).

Collapsible soils generally consist of relatively dry, low-density materials that become weaker and more compressible with the addition of water or excessive loading. Due to the cohesive and very stiff to hard nature of the onsite materials, the potential for collapse of soils at this site to impact the project is considered very low.

5.9 SUBSIDENCE AND SETTLEMENT

The project site is not within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation (natural decay of organic peat materials), or hydrocompaction (compression of soils due to the introduction of water). Therefore, the potential for subsidence and associated settlement is considered to be low.

5.10 FLOODING AND INUNDATION

According to a flood map (Map Number 06037C1269J, dated September 26, 2008) prepared by the Federal Emergency Management Agency (FEMA), the project site is not located within a mapped flood zone (msc.fema.gov). Based on this information, the potential for flooding to negatively impact the project is considered to be very low.

5.11 SEDIMENTATION AND EROSION

The majority of the ground surface at the site is relatively level and is, or will be, covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site. During construction, provisions should be in place to address potential temporary erosion and sedimentation conditions. These provisions will be provided in the design-level geotechnical report and incorporated into the project civil and landscaping plans.

5.12 CORROSIVE SOILS

Corrosivity laboratory test data presented in our geotechnical investigation reports (GPI, 2022a and 2022b) suggests that the on-site soils and bedrock are severely corrosive to concrete and buried ferrous metals. The bedrock materials are also considered to be generally acidic. A corrosion engineer should be consulted to provide recommendations to address the impact of corrosive soils on the proposed project. These recommendations may include specific concrete mix designs, structural details for reinforced concrete foundations, and utility line protection that conforms to the California Building Code. As such, corrosive soils are not considered to be a hazard at the site after proper measures are implemented.

6.0 LIMITATIONS

The report and other materials resulting from GPI's efforts were prepared exclusively for use by The Commons at Calabasas, LLC c/o Caruso, and their consultants in planning and designing the proposed development. The report is not intended to be suitable for reuse on extensions or significant modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers and Engineering Geologist practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER DYLAN J. BOYLE No. 3139 GEOTECHNICAL STATE OF CALIFORNIA

Dylan J. Boyle, G.E.
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER JUSTIN J. KEMPTON No. 2385 GEOTECHNICAL STATE OF CALIFORNIA

Justin J. Kempton, G.E.
Principal

REGISTERED GEOLOGIST THOMAS G. HILL No. 1100 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Thomas G. Hill, C.E.G.
Consulting Geologist

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BASE MAP REPRODUCED FROM © CALTOPO

GEOTECHNICAL PROFESSIONALS, INC.

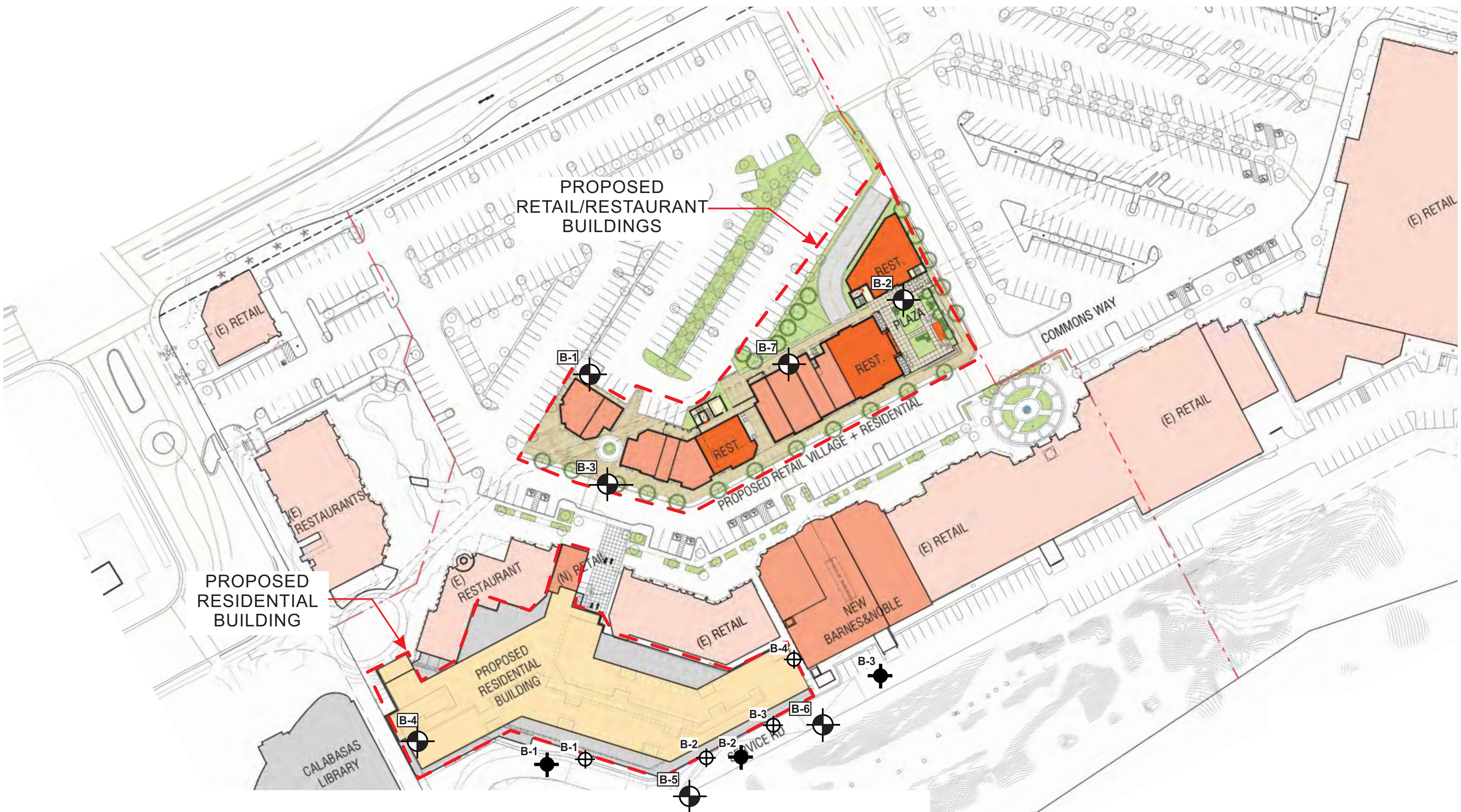
THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I

SCALE: 1" = 2000 FEET

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

APPROXIMATE LIMITS OF PROPOSED PROJECT

BASE MAP REPRODUCED FROM SITE PLAN
BY ELKUS MANFREDI ARCHITECTS, UNDATED

GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO.: 3063.I

SCALE: 1" = 150'

SITE PLAN

(Proposed Conditions)

FIGURE 2

EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

APPROXIMATE LIMITS OF PROPOSED PROJECT

BASE MAP REPRODUCED FROM GOOGLE EARTH @ 2021

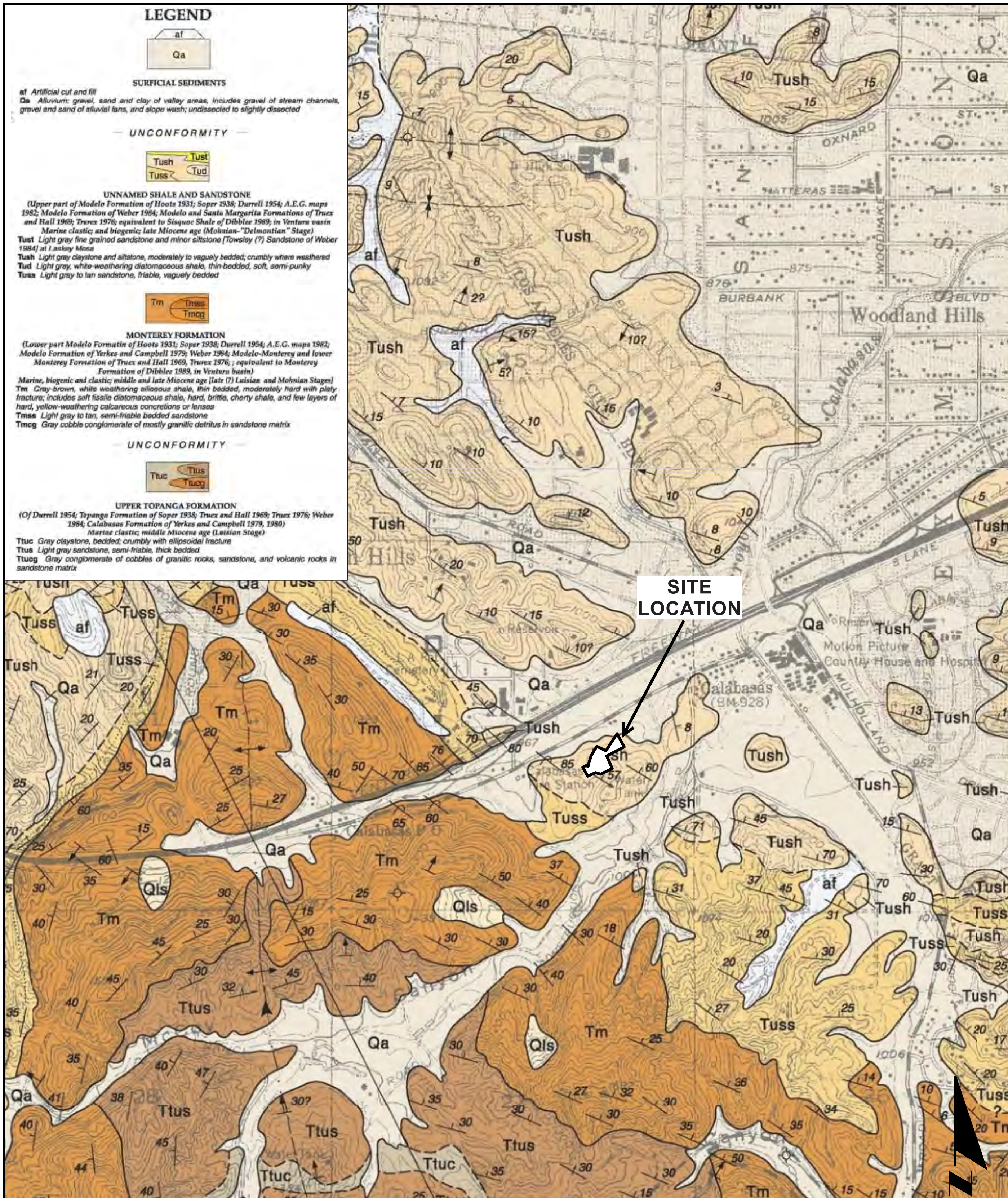
GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS - RESIDENTIAL BUILDING

GPI PROJECT NO.: 3063.I | SCALE: 1" = 150'

SITE PLAN
(Existing Conditions)

FIGURE 3



BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE CALABASAS QUADRANGLE (#DF-37) PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 1992

0 2000 4000 FEET

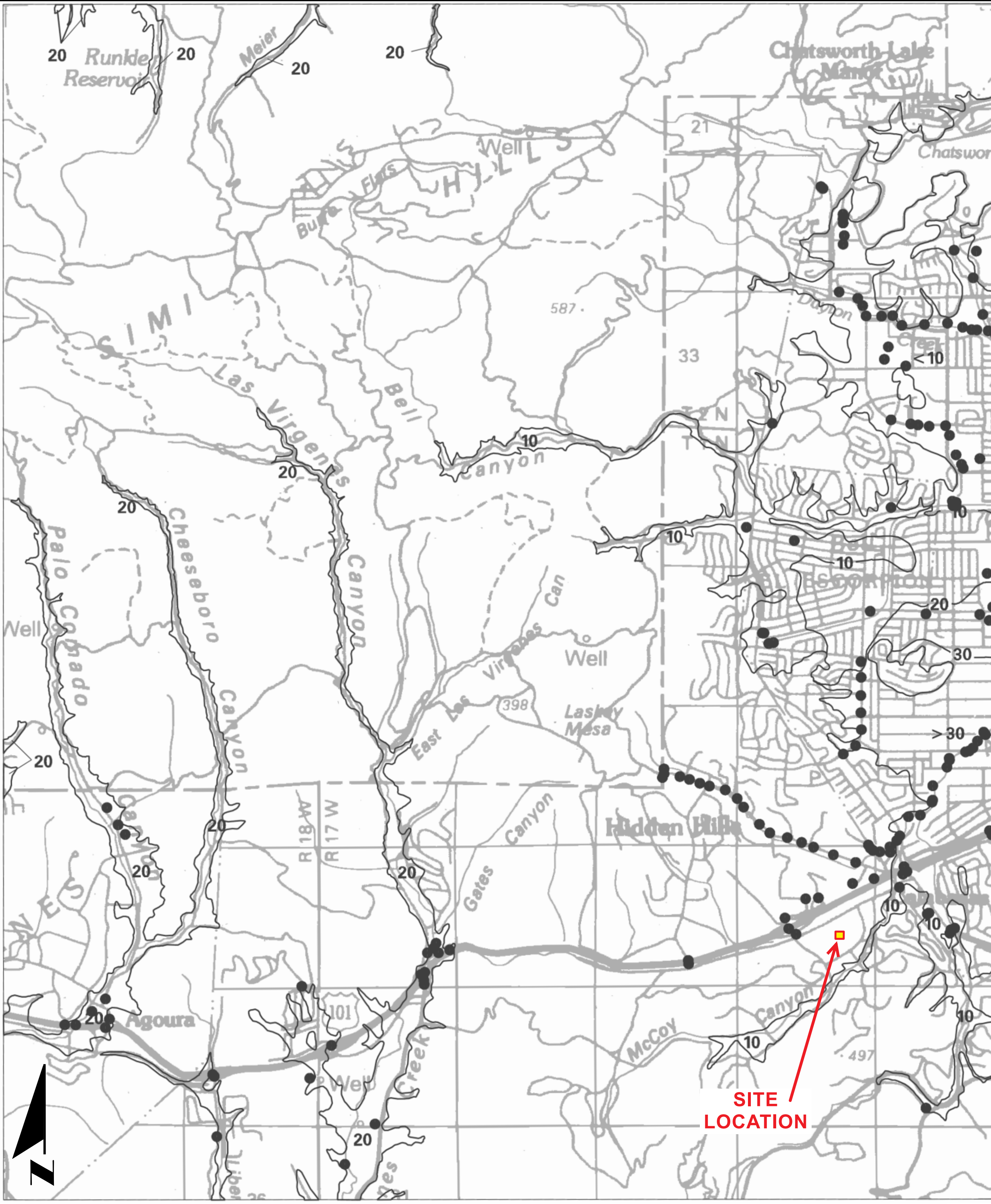
GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO.: 3063.I	SCALE: 1” = 2000’

REGIONAL GEOLOGIC MAP

FIGURE 4



BASE MAP REPRODUCED FROM HISTORICAL GROUNDWATER MAP (PLATE 1.2) FROM SEISMIC HAZARD ZONE REPORT 06 (1997)

GEOTECHNICAL PROFESSIONALS, INC.

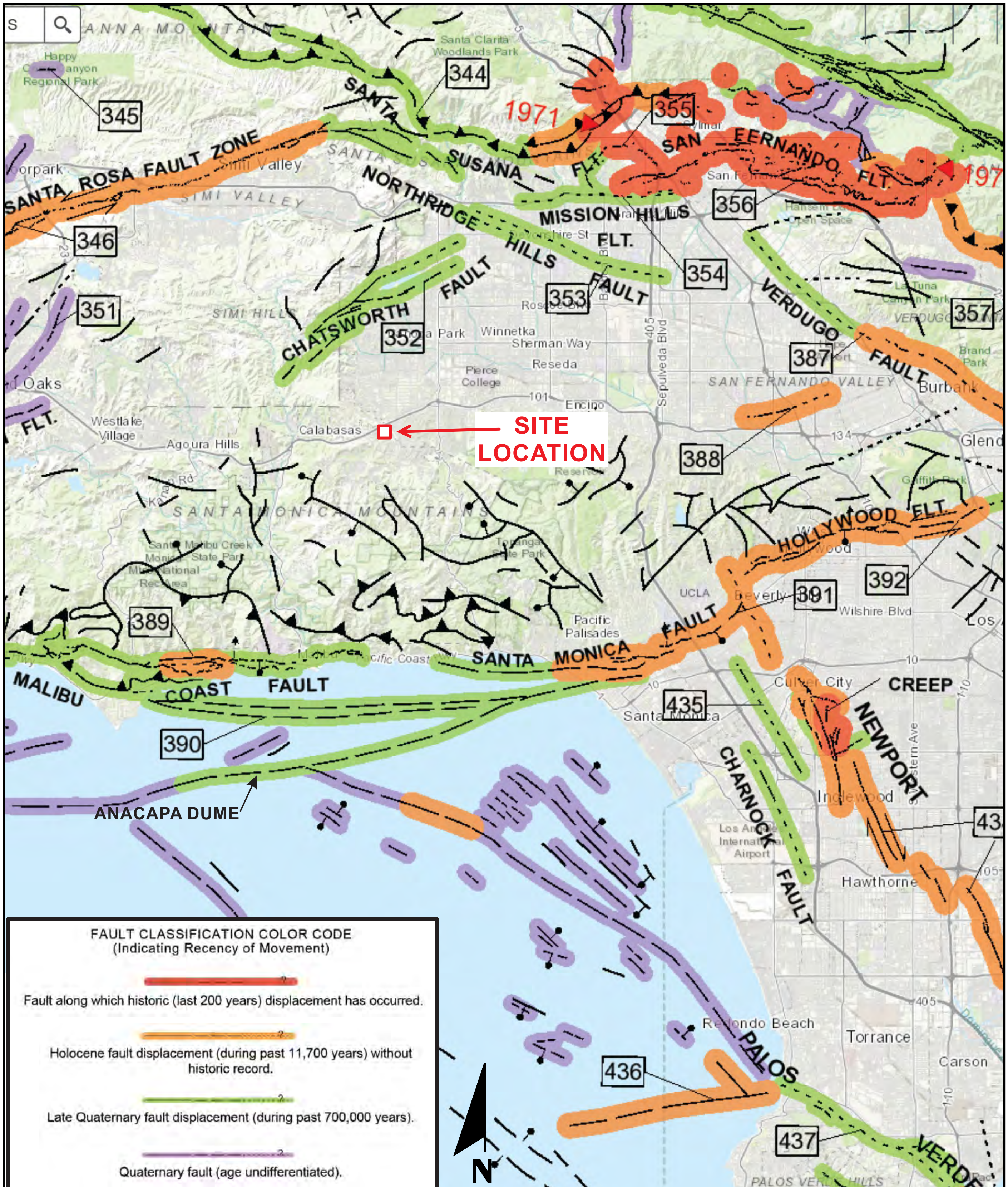
THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I

SCALE: 1" = 1 MILE

HISTORICAL HIGH GROUNDWATER MAP

FIGURE 5



0 5 10 MILES

BASE MAP REPRODUCED FROM FAULT ACTIVITY MAP OF CALIFORNIA BY THE CALIFORNIA GEOLOGICAL SURVEY, C.W. JENNINGS, W.A. BRYANT: DATED 2010

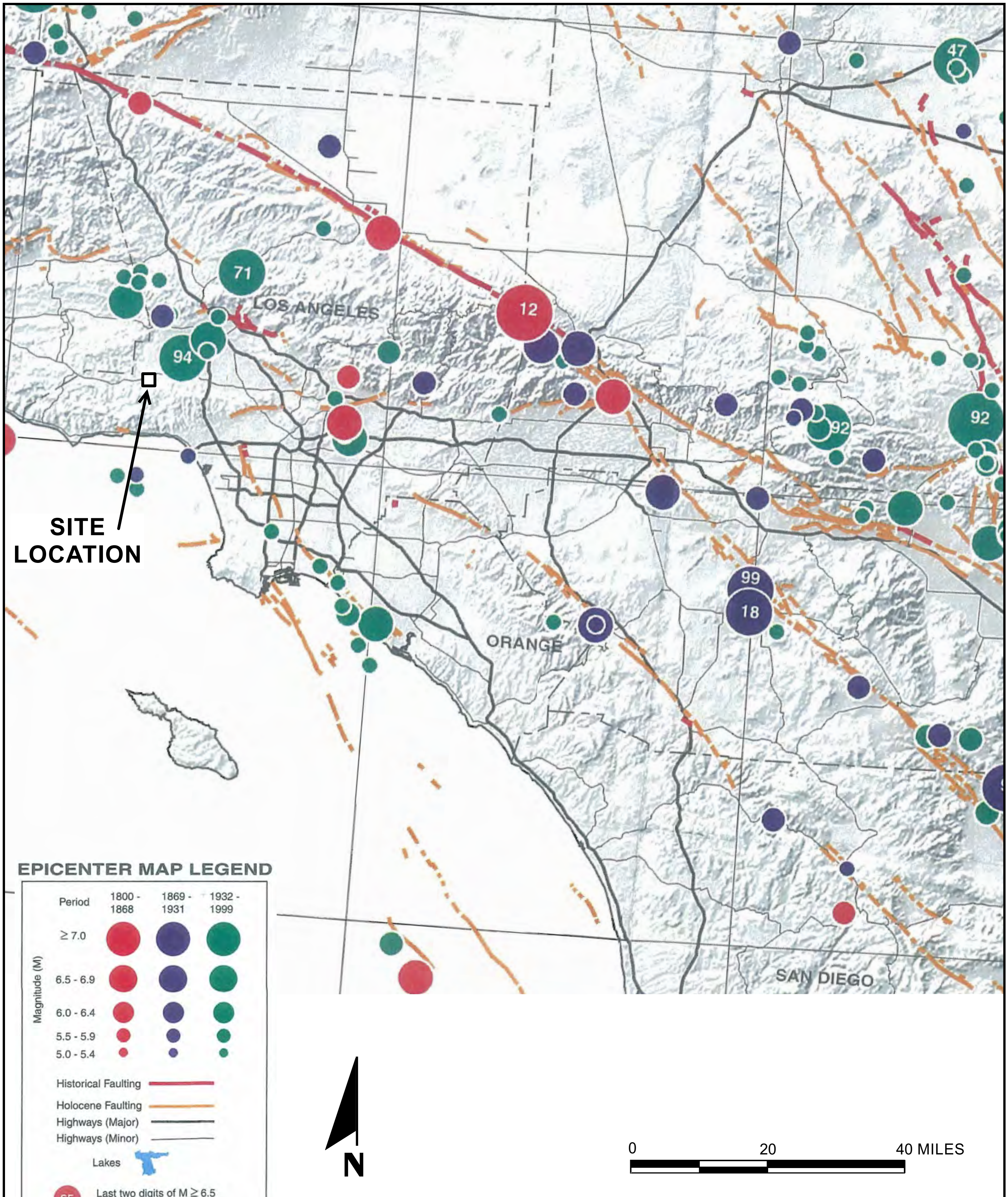
GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I | SCALE: 1" = 5 MILES

REGIONAL FAULT MAP

FIGURE 6



REGIONAL FAULT MAP REPRODUCED FROM EARTHQUAKE FAULTS PLAN FROM MONTEREY PARK GENERAL PLAN BY BING YEN & ASSOCIATES, INC.: DATED JULY 2001

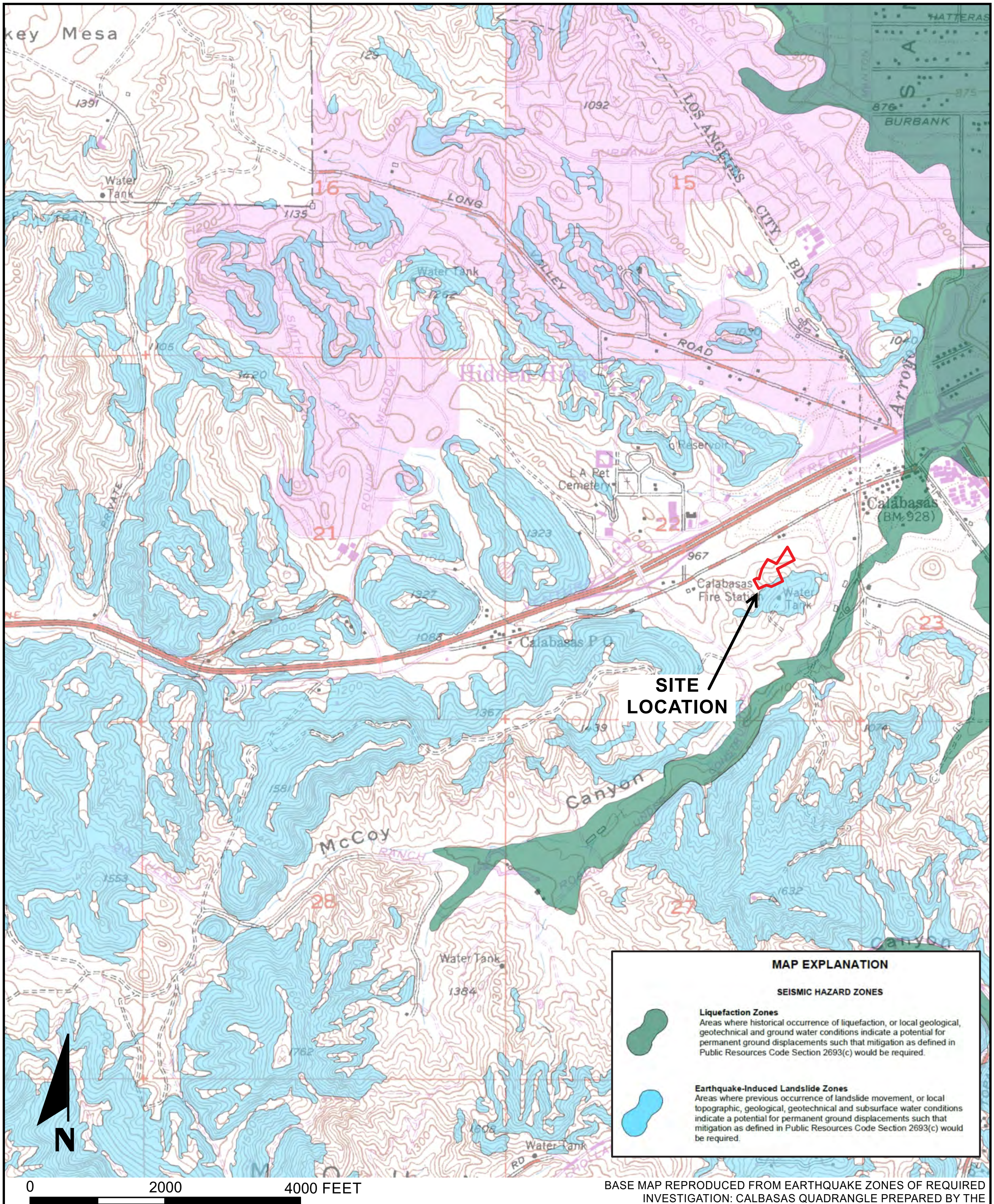
GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO.: 3063.I | SCALE: 1" = 20 MILES

REGIONAL SEISMICITY

FIGURE 7



BASE MAP REPRODUCED FROM EARTHQUAKE ZONES OF REQUIRED INVESTIGATION: CALBASAS QUADRANGLE PREPARED BY THE CALIFORNIA GEOLOGIC SURVEY: RELEASED FEBRUARY 1, 1998

GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I | SCALE: 1" = 2000'

SEISMIC HAZARDS ZONES MAP

FIGURE 8

ATTACHMENT

The checklist below is provided for input into Section VI. Geology and Soils of the Appendix G CEQA Environmental Checklist Form for the proposed development. A brief explanation is provided below each item.

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant with Mitigation Incorporated	Less Than Significant Impact	No Impact
VI. GEOLOGY AND SOILS. Would the project:				
a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:				
i) Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault? Refer to Division of Mines and Geology Special Publication 42.	☐	☐	☒	☐
Brief Explanation: The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (CGS). Therefore, impacts would be less than significant.				
ii) Strong seismic ground shaking?	☐	☐	☒	☐
Brief Explanation: The site could be subjected to strong ground shaking in the event of an earthquake, which could constitute a potential hazard to the project. The effects of strong seismic ground shaking can be mitigated by design and construction in conformance with current building codes and engineering practices. The project will be designed in accordance with the California Building Code. Therefore, impacts would be less than significant.				
iii) Seismic-related ground failure due to liquefaction?	☐	☐	☒	☐
Brief Explanation: The site is not located within an area designated by the State Geologist as a "zone of required investigation" with respect to the potential for liquefaction. In addition, the subsurface soils consist primarily of fine-grained, plastic soils overlying hard bedrock materials. As such, liquefaction is considered unlikely at this site, and impacts would be less than significant.				
iv) Seismic-related ground failure due to landslides?	☐	☐	☒	☐
Brief Explanation: The southeast portion of the site is located within an earthquake-induced landslide zone as designated by the State Geologist(CGS, 1998). The map, which was published near the time site grading was completed, includes the slope that ascends to the south. The portion of the site within the mapped landslide zone was cut up to approximately 45 feet and the ascending slope is currently retained by the soldier pile with tieback anchor wall. The soldier pile with tie back anchor wall was designed by others to support the ascending slope and mitigate the potential for slope instability. Adverse slope conditions that would need to be addressed during conceptual design were not observed during the site reconnaissance on August 9, 2023 by GPI's Geotechnical Engineer and Engineering Geologist.				

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant with Mitigation Incorporated	Less Than Significant Impact	No Impact
Site grading for the proposed buildings is anticipated to include cuts up to approximately 15 feet and fills up to 10 feet. Significant new permanent cut or fill slopes are not planned. Based on prior and planned site grading, construction of the soldier pile with tieback anchor wall, as well as the presence of near-surface bedrock materials, the potential for seismic-related ground failure due to landsliding for the project is considered less than significant. *Furthermore, the redevelopment of the site would comply with the California Building Code with City of Calabasas amendments, the City of Calabasas Municipal Code, and the policies in the General Plan Safety Element. Therefore, impacts would be less than significant.*				
b) Result in substantial soil erosion or the loss of topsoil?	☐	☐	☒	☐
Brief Explanation: The potential for ongoing erosion during operation of the project is considered to be very low. There is a potential for erosion of soils during construction, but the project would be required to comply with the SWRCB's General Permit for Discharges of Stormwater Associated with Construction Activity (Construction General Permit Order 2009-0009-DWQ, as amended by 2010-0014-DWQ and 2012-0006-DWQ). Additionally, development of the site would require preparation and implementation of a SWPPP. The SWPPP would include site-specific BMPs that would be implemented to prevent erosion and loss of topsoil and would include applicable monitoring programs to be implemented as necessary. *During operation, the project would continue to comply with the City's LID ordinance as outlined in Calabasas Municipal Code Chapter 8.28.160 and maintain BMPs (Chapter 8.28.120). Compliance with the City's LID ordinance and continuation of BMPs would prevent soil erosion and the loss of topsoil. Furthermore, there would be no additional soil erosion and/or loss of topsoil during operation because the site would be fully developed with structures, impervious surfaces, and landscaping. Therefore, impacts would be less than significant.*				
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?	☐	☐	☒	☐
Brief Explanation: The potential for landslide to impact the proposed development was detailed in Section (a, iv). Because the potential for seismic-related liquefaction is considered unlikely at this site (see Section (a, iii), the potential for lateral spreading to occur during liquefaction is also considered to be remote. Because the site is underlain by bedrock, the potential for subsidence and/or collapse is remote. Further redevelopment of the site would comply with the California Building Code with City of Calabasas amendments, the City of Calabasas Municipal Code, and the policies in the General Plan Safety Element. Therefore, impacts would be less than significant.				
d) Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial risks to life or property?	☐	☐	☒	☐

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant with Mitigation Incorporated	Less Than Significant Impact	No Impact
Brief Explanation: Low to moderately expansive soils were encountered in recent and prior explorations at the site. In addition, the bedrock materials at the site were found to contain at least 2 percent pyrite (iron sulfide), which can result in bedrock heave if the pyrite is exposed and oxidizes to create gypsum crystals within bedrock fractures. Based on the above, the potential for expansive soils/bedrock to adversely affect the project if not addressed is considered to be high. The potential impact of expansive soils and bedrock on the proposed project can be reduced by design and construction in conformance with current building codes and engineering practices. The project will be designed in accordance with the California Building Code. Therefore, impacts would be less than significant.				
e) Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?	☐	☐	☐	☒
Brief Explanation: The project does not include a septic system and would connect to the exiting sewer system. Therefore, the impacts would be less than significant.				
f) Directly or indirectly destroy a unique paleontological resource or site or unique geological feature.	☐	☐	☐	☐
Brief Explanation: Not applicable. An assessment of impacts to paleontological resources is outside the scope of this Geotechnical Investigation. Potential impacts to paleontological resources will be addressed by others. The project is not expected to destroy a unique geological feature.				

GEOTECHNICAL INVESTIGATION
PROPOSED RETAIL AND RESTAURANT BUILDINGS
THE COMMONS AT CALABASAS
4799 COMMONS WAY
CALABASAS, CALIFORNIA

Prepared for:
THE COMMONS AT CALABASAS, LLC
c/o CARUSO
101 The Grove Drive
Los Angeles, CA 90036

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 3063.I

December 22, 2022

GEOTECHNICAL
PROFESSIONALS INC.

December 22, 2022

The Commons at Calabasas, LLC
c/o Caruso
101 The Grove Drive
Los Angeles, CA 90036

Attention: Tasha Reeder
Project Manager, Construction

Subject: Report of Geotechnical Investigation
Proposed Retail and Restaurant Buildings
The Commons at Calabasas
4799 Commons way
Calabasas, California
GPI Project No. 3063.I

Dear Tasha:

Transmitted herewith is our report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have any questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Patrick McGervey, P.E.
Project Engineer
(pmcgervey@gpi-ca.com)

Justin J. Kempton, G.E.
Principal
(justink@gpi-ca.com)

Distribution: Addressee (PDF)

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed retail and restaurant buildings at the subject site in Calabasas, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of four new retail and restaurant buildings to be constructed within the existing parking lot of The Commons at Calabasas center. The buildings will range from approximately 2,000 to 11,000 square feet in plan, and either be single- to 3-level structures. The structures will be supported at-grade or underlain by a single-level-subterranean parking garage that will extend up to 15 feet below existing grades. Retail and restaurant space is planned at the first floor for each building and residential apartments and amenities are planned for levels 2 and 3.

We were provided with a general site layout showing the locations of the proposed buildings in a Conceptual Site Plan by Elkus Manfredi Architects (undated). The conceptual layout of the proposed buildings is shown on Figure 2, Site Plan. The above grade portions of the buildings will be of wood frame construction and the subterranean parking level is anticipated to be of reinforced concrete construction. The buildings are anticipated to be supported on shallow spread footings with slab on grade floors at grade or at the basement level. A parking garage access ramp is planned in the northeast portion the site.

The project will also include parking lot improvements, minor site walls, pedestrian hardscape and landscaping in the remainder of the project area. Based on the known subsurface conditions, stormwater infiltration is not anticipated for the project.

Proposed finish floor elevations for the ground floor levels of the proposed buildings are anticipated to be approximately +969 to +971 feet and the subterranean level is anticipated to be at approximate elevation +955 to +960 feet. Accordingly, site grading is anticipated to include cuts up to approximately 15 feet and fills up to 10 feet. Proposed structural loads were not available at the time this report was prepared. Based on similar past projects, we assume that maximum column and wall loads will be on the order of 100 to 300 kips and 3 to 7 kips per lineal foot, respectively (dead plus live loads).

A 10-foot wide sanitary sewer easement traverses the project area. Based on the ALTA Survey by Hennon dated September 18, 2000, the invert elevation of the sewer line in the easement and just east of the project area is approximately +952.4 (approximately 8 to 9 feet below existing grade).

Based on the Rough Grading Plans for the center dated March 2, 1998, and prior discussion with Caruso representatives, we understand grading of the center was originally performed in 1998 and construction of the adjacent buildings was completed in 1999.

Our recommendations are based upon the above structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to either confirm or modify our recommendations. Also, when the project grading and foundation plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork, and design of foundations and pavements.

2.0 SCOPE OF WORK

Our scope of work included review of existing data, subsurface exploration, laboratory testing, engineering analysis and the preparation of this report. We performed concurrent geotechnical investigations for the proposed retail and restaurant buildings discussed herein and the proposed residential building (to be constructed south of the retail and restaurant buildings) and have incorporated the subsurface explorations and laboratory testing for both sites into this report.

We were provided with prior geotechnical reports by others that addressed floor slab and flatwork distress (Kleinfelder, 2009 and Kleinfelder, 2017) for adjacent retail buildings in The Commons at Calabasas center. The subsurface soil information presented in the referenced reports was reviewed as part of this study.

Our subsurface exploration program consisted of a total of seven hollow stem auger borings performed to depths of approximately 21 to 51 feet below existing grades. Borings B-1 through B-6 were drilled in August 2021 and Boring B-7 was drilled in October 2022 as part of this report update. A description of field procedures and logs of the explorations are presented in Appendix A. The approximate locations of our subsurface explorations, as well as the locations of the nearby prior subsurface explorations by others are shown on the Site Plans, Figures 2 and 3.

Laboratory soil tests were performed on selected representative samples from the borings as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size analyses, Atterberg limits, shear strength, corrosivity, expansion index, R-value and maximum density. Laboratory testing procedures and results are summarized in Appendix B.

Corrosivity testing was performed by HDR under subcontract to GPI. R-value testing was performed by Geologic Associates under subcontract to GPI. Their test results are presented Appendix B.

Engineering evaluations were performed to provide earthwork criteria, foundation design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site is located within the existing at grade parking lot for The Commons at Calabasas shopping center, just north of the cinema and retail buildings that exist along the southern side of the center. The center is surrounded by Calabasas Road to the north, Park Granada to the east, a slope leading up to Park Granada to the south, and the Calabasas City Hall and Library to the west. The parking lot consists predominately of asphalt drives and parking stalls with portland concrete cement curbs and gutters and landscaping. At the boring locations, we encountered asphalt pavement sections consisting of 3 to 5 inches of asphalt concrete over 5 to 6 inches of aggregate base.

In general, the site gradually slopes downward from the southwest to the northeast, with a change in ground surface elevation from about Elevation +971 feet to +958 feet across the site.

3.2 SUBSURFACE MATERIALS

Our field investigation disclosed a subsurface profile consisting of shallow fill soils overlying natural bedrock. Detailed descriptions of the conditions encountered are shown on the Log of Borings in Appendix A.

We encountered undocumented fills up to approximately two feet below existing grade in the explorations in the area near the proposed retail and restaurant buildings. The fill materials encountered consisted of slightly moist to moist sandy clays. The fill materials are considered undocumented because documentation of the fill has not been made available for our review. It is likely these fills were placed during original grading of the center around 1998. Expansion Index testing on representative samples of the sandy clay bedrock indicates the materials have a low to medium potential for expansion.

The underlying natural materials encountered consisted of hard, moist to very moist, siltstone bedrock to the depths explored. According to reports by others (Kleinfelder, 2009 and Kleinfelder, 2017) the bedrock is mapped as the Modelo Formation. The bedrock materials have moderate to high strength and low compressibility characteristics. The moisture content of the siltstone bedrock materials was consistently moist to very moist within our explorations ranging from 17 to 35 percent with an average of 27 percent, which is about 4 percent above the optimum moisture content. Expansion Index testing on a representative remolded sample of the bedrock indicates the materials (when remolded) have a medium potential for expansion. Additional discussion regarding the expansion potential of the bedrock materials is presented in the following section.

The fill and bedrock materials encountered in our current explorations are comparable to materials reportedly encountered in prior nearby borings by others (Kleinfelder, 2009 and Kleinfelder, 2017).

Corrosivity testing of the upper site soils and bedrock materials indicates they are severely corrosive to buried metal and concrete elements. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. If corrosion recommendations are required, a corrosion engineer such as HDR should be consulted.

3.3 BEDROCK EXPANSION POTENTIAL

Prior testing of the bedrock materials (Kleinfelder, 2017) indicated that when remolded, the bedrock materials had a low to medium expansion potential and that the in-situ bedrock had a slight potential for moisture induced heave when the confining pressures were less than 200 pounds per square foot (psf). The in-situ bedrock materials had a relatively minimal potential for moisture induced heave when confining pressures were at least 1,500 to 2,000 psf.

Mineralogical testing was also performed previously on the bedrock materials within the upper 10 feet at the site by others (Kleinfelder, 2017) to determine the mineral constituents in the bedrock. Their testing concluded that all the samples tested contained over 2 percent pyrite (iron sulfide). Oxidation of pyrite minerals present in the bedrock will form gypsum crystals within the exposed bedrock fractures and surfaces that can result in ground expansion (Bryant, 2003).

The performance of the bedrock underlying adjacent buildings has been linked to significant differential movement (heave) of floor slabs at the center (Kleinfelder, 2017). Significant heave of floor slabs has resulted in cracked and distressed floors, partition walls, door jams, and other distress to lightly loaded fixtures and racks supported on floors. Differential heave of the bedrock has been attributed to a combination of unloading of the bedrock (by site grading), minor swelling due to changes in moisture content, and mineralogical changes in the bedrock. The oxidation of pyrite minerals to form gypsum within bedrock fractures and exposed surfaces was identified as the primary cause of differential floor slab movement (up to 5 inches) in the nearby Barnes and Noble store (Building G) located immediately south of the project area (Kleinfelder, 2017). Differential heave of floor slabs has also been documented in retail units east and west of the existing Barnes and Noble Store.

3.4 GROUNDWATER CONDITOINS

Groundwater was not encountered in our explorations to depths of 51 feet. However, moist to very moist bedrock materials were encountered at depths starting at approximately 2 feet below existing grades.

Published data by the California Geologic Survey (CGS 1998) does not map the historical high groundwater at the site, however it does indicate historical high groundwater at a depth of approximately 10 feet below the ground surface to the south and east of the site. Additionally, groundwater was encountered as shallow as 3 to 9½ feet below existing grades in prior borings by others (Kleinfelder, 2017) in the walkway south of the Edwards Cinema Building and between the Edwards Cinema Building and the Barnes and Noble Store.

Caving was not noted in the small diameter borings performed and is not expected to be a constraint during construction.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed, provided the geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed buildings are as follows:

- The bedrock materials encountered in our explorations for the proposed buildings are comparable to the materials identified in prior nearby explorations by others at the center that have historically been associated with localized differential heave of slab on grade floors. The heave has been attributed to unloading of the bedrock (site grading), changes in moisture content of the bedrock, and oxidation of pyrite minerals exposed in bedrock surfaces and fractures. The latter mechanism (oxidation of pyrite minerals) appears to be the predominant factor causing the observed heave related distress to slab on grade floors at the center. The formation of gypsum crystals results in expansion (swelling) of the underlying bedrock and the upward forces have been sufficient to heave slab on grade floors. Floor slab heave impacts partition walls, doors, fittings, and mechanical equipment in addition to other distress and nuisances caused by non-level and distressed floor slabs. If a subterranean parking floor slab were to heave, there is concern that reduced height limits in parking level would result.

- Options are provided in the report to mitigate the potential distress of floor slabs that could be caused by heave of the underlying bedrock. The optional recommendations are based on using design measures to allow for movement of the bedrock and/or reducing access of air (oxygen) to the freshly exposed bedrock and bedrock fractures. For Option 1, we recommend the building floor slabs be structurally supported on spread foundations and suspended above the bedrock a minimum of 12 inches. The space between the bottom of the floor slab and bedrock could be filled with compressible fill material that will support construction of the floor slab and compress as the bedrock materials expand. For Option 2, we recommend the floor slab be underlain by a select non-expansive and low-permeability compacted fill material placed over the exposed bedrock to reduce the potential for future exposure of the bedrock to moisture changes and oxygen.

- By observation, heave of foundations has not been historically noted at the center. This could potentially be attributed to the higher pressures imposed by foundations either counteracting potential uplift forces from the bedrock or impeding the oxidation process of the pyrite minerals.

- To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on subterranean walls, we recommend walls below grade be backfilled with granular soils following temporary sloped excavations in the bedrock made at an inclination of 1:1 to the outside edge of the spread foundation supporting the wall. Alternatively, if there is insufficient space for sloped excavations, we recommend the vertical excavation required to construct the subterranean wall be set back 3 feet from the back of the wall to the top of wall footing and the subterranean wall be backfilled with select low-permeability and non-expansive compacted fill material. The 3-foot recommended setback could be reduced to 8 inches provided the vertical excavation sidewalls are coated with a water-based membrane/vapor barrier (such as Liquid Boot) to seal the bedrock and reduce its exposure to oxygen and compressible fill material is placed between the back of wall and bedrock surface.

- We recommend the proposed buildings be supported on shallow spread foundations underlain uniformly by bedrock materials or properly compacted fill. We recommend that stepped footings be avoided.

- The on-site clay soils have a low to medium expansive potential and will shrink and swell with changes in moisture content. We recommend concrete pedestrian hardscape be supported on select non-expansive fill.

- Undocumented fills were reported to depths of up to 2 feet below existing grade at the site. Although the fills were likely placed during original rough grading in 1998 for the existing parking lot, the fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. For the proposed improvements, we recommend removal and recompaction of the fill to provide uniform support for the planned foundations and floor slabs. Below new pavements, we recommend the existing fill be scarified, moisture conditioned and recompacted in place prior to placement of new fill or aggregate base and paving.

- Current moisture contents of the bedrock and overlying fill soils are moist to very moist and at and above optimum moisture content. As such, mixing/discing and extensive moisture conditioning will be required to achieve suitable moisture contents of the onsite fill soils. The onsite bedrock materials are not considered suitable for use as compacted fill within the building areas. The earthwork contractor should evaluate the moisture content of the existing soils when planning the earthwork.

- Corrosivity testing of the upper site soils and bedrock materials indicates they are severely corrosive to buried metal and concrete elements. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. If corrosion recommendations are required, a corrosion engineer such as HDR should be consulted.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

The site is in a seismically active area of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2019 or 2022 California Building Code (CBC) criteria. We do not anticipate significant changes with respect to Site Class and seismic design parameters discussed herein. For the 2019 CBC, a Site Class C may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate seismic design maps, the corresponding seismic design parameters from the CBC are as follows:

2019 CBC:

$S_S = 1.619g$	$S_{MS} = F_a * S_S = 1.942g$	$S_{DS} = 2/3 * S_{MS} = 1.295g$
$S_1 = 0.57g$	$S_{M1} = F_V * S_1 = 0.815g$	$S_{D1} = 2/3 * S_{M1} = 0.543g$

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the Malibu Coast Fault, which is located 7.6 miles from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.80 for a mean magnitude 6.8 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2019 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture at this site due to faulting is considered unlikely.

4.2.4 Liquefaction and Seismic Settlement

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is not located within an area mapped by the State of California as having a potential for soil liquefaction (Calabasas Quadrangle, CGS 1997). Due to the bedrock materials beneath the proposed structures, the potential for liquefaction and associated settlement at the site is considered to be low.

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. Due to the bedrock materials beneath the proposed structures, the potential for seismic ground subsidence to adversely affect the site is considered to be low.

4.3 EARTHWORK

The earthwork for the planned improvements is anticipated to consist of clearing and excavation of undocumented fill and bedrock materials, as necessary, subgrade preparation, importing select soils for placement beneath floor slabs, pedestrian hardscape and behind subterranean walls, and the placement and compaction of fill. Site grading is anticipated to include cuts up to approximately 15 feet for the subterranean level and fills up to 10 feet.

With favorable weather, we anticipate active mechanical drying using earthwork equipment such as a disc will be a feasible option to lower the soil moisture content. In the rainy season, we would anticipate significantly longer drying times or the need for drying with cement treatment.

4.3.1 Clearing

Prior to grading, performing excavations or constructing the proposed improvements, the areas to be developed should be cleared of existing structures, debris, and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during clearing should be removed from the site. Existing vegetation should not be mixed into the soils. Inert demolition debris, such as concrete and asphalt, may be crushed for reuse in engineered fills in accordance with the criteria presented in the “Material for Fill” section of this report.

Although not encountered in our explorations, and unlikely at the site, if cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill, excavation of the proposed subterranean level, footing excavations, and trenching for proposed utility lines.

Removals and Overexcavations

To provide uniform support for the planned improvements and to mitigate the potential impacts of expansive materials, prior to placement of fills or construction of the building, the existing fill soils and a portion of the bedrock within the proposed building pad should be removed and replaced with select materials. Removals for the building pads should extend to a depth of at least 2 feet below existing grades and to the minimum depths below floor slabs as recommended below, whichever is deeper.

- For (Option 1) floor slabs that will be structurally supported on spread foundations and suspended above exposed bedrock materials, removals should extend to a depth below floor slabs that will be sufficient to place at least 12 inches of compressible fill materials between the exposed bedrock and bottom of floor slab.

- For (Option 2) floor slabs that will be supported on select compacted fill, removals should extend at least 3 feet below bottom of floor slab. For this option, the floor slab is recommended to be underlain by at least 3 feet of 'non-expansive' compacted fill at least the lower 24 inches placed on the exposed bedrock consisting of 'low-permeability' fill. These materials are further defined in Section 4.3.4 of this report.

For planning purposes, if the foundations are embedded in bedrock materials, removals below the proposed foundations are not required unless the bedrock materials are disturbed. The actual depths of removal should be determined in the field during grading by a representative of GPI.

For minor at-grade supported structures, such as site walls, canopies, and short retaining walls, the existing fills should be removed and the footings should be underlain by at least 2 feet of properly compacted, non-expansive fill or the undisturbed bedrock materials. Removals below pedestrian hardscape should extend at least 18-inches below finished subgrade so that at least 18 inches of imported non-expansive fill can be placed below pedestrian hardscape. For pavement outside the building, removals are not required unless the depth of disturbance exceeds 6-inches.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally at least 5 feet beyond the building lines or edge of foundations, or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls.

Excavation of the soils and bedrock at the site should be readily achieved using conventional methods. Difficulties in drilling our borings in the upper bedrock were not encountered.

We recommend excavations of the bedrock be conducted with the least possible disturbance of the bedrock below grade and behind basement walls. Fracturing of the bedrock provides increased access to air and promotes the expansion of the bedrock by the pyrite oxidation process.

Existing Utility Trenches

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pads. This is especially important for deeper fills associated with existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities that are 5 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan. Wet utilities left in-place outside building areas should be capped to reduce the potential for water to infiltrate into the building pad.

Temporary Sloped Excavations

The upper soils and bedrock at the site are expected to have a low caving potential when exposed in open cuts. Excavations in bedrock should be evaluated by a geologist of GPI to evaluate the excavations for the presence of adverse bedding conditions. Temporary construction excavations may be made vertically into the existing fill to depths of 4 feet and undisturbed bedrock to a depth of 5 feet below adjacent grade, without shoring. For cuts up to 12- and 20- feet deep, the slopes should be properly shored or sloped back to at least 1:1 and 1½:1 (H:V), respectively, or flatter. The allowable slope inclinations are measured from the toe to the top of the cut. Even at these inclinations, some raveling should be anticipated. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards. Recommendations for temporary shoring are provided in Section 4.5 of this report.

Subdrains

Groundwater was not encountered in our explorations to depths of 51 feet. However, moist to very moist bedrock materials were encountered at depths starting at approximately 2 feet below existing grades. Additionally, groundwater was encountered as shallow as 3 to 9½ feet below existing grades in prior borings by others (Kleinfelder, 2017) in the walkway south of the Edwards Cinema Building and between the Edwards Cinema Building and the Barnes and Noble Store.

We recommend that a representative of GPI observe the exposed subgrade conditions following excavations for signs of seepage or overly wet soils that would require installation of subdrains. If needed, GPI will provide recommendations to collect and direct subsurface water away from the exposed subgrade below floor slabs.

4.3.3 Subgrade Preparation

Loose or disturbed soils should be removed from the subgrade prior to placement and compaction of the overlying fill soils. Scarification of the bedrock subgrade is not required.

In areas to receive pavements (outside of the structures), and where fill is to be placed over existing fill evaluated to be suitable to leave in place, the upper 12 inches of the exposed subgrade soils should be scarified, moisture-conditioned, and compacted to a minimum of 90 percent of the maximum dry density.

4.3.4 Material for Fill

The on-site soils and bedrock materials are, in general, suitable for use as compacted fill with the exception of retaining wall backfill or placed within 36 inches of the finished subgrade for floor slabs and within 18 inches of the finished subgrade for pedestrian hardscape. In general, we recommend the onsite bedrock materials not be used as compacted fill within 3 feet of the building footprint.

We recommend the soils placed within 36 inches of the finished subgrade for floor slabs and within 18 inches of the finished subgrade for pedestrian hardscape consist of imported, non-expansive (EI< 20) soils. In addition, the lower 24-inches of select materials placed under the floor slabs and directly over the exposed bedrock should consist of low-permeability (contain no less than 60 percent fines – portion passing No. 200 sieve) soils to reduce the potential exposure of the bedrock to changes in moisture and to oxygen. The low-permeability soils may be considered as part of the non-expansive fill provided it meets the applicable criteria.

Soils used for general wall backfill should be predominately granular (contain no more than 40 percent fines – portion passing No. 200 sieve) and non-expansive (E.I. less than 20). To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on the subterranean walls, the subterranean walls supporting bedrock materials should either be backfilled with select low-permeability and non-expansive compacted fill material that extends at least 3 feet laterally beyond the back of wall or the walls should be constructed adjacent to bedrock that is sealed immediately after excavation.

Suitable 'non-expansive' and 'low-permeability/non-expansive' soils are not anticipated to be available on-site and will need to be imported. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of debris and pieces larger than 6 inches in greatest dimension. If approved by the client and regulatory agencies, the on-site portland cement concrete and asphalt concrete can be crushed/pulverized and mixed with the on-site soils prior to performing the overexcavation. The material should be crushed so that the resulting particle size is less than 3 inches in diameter if used for stabilization, and it should be mixed with the on-site soils if used for general fill. If used to support pavements, it should be crushed to meet the specifications of Caltrans Class II or Greenbook crushed miscellaneous base (CMB).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. The aggregate base material should be compacted to a relative compaction of at least 95 percent. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field.

The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	3-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-9 inches
Scrapers, Heavy loaders, and large vibratory rollers	9-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content for the sandy soils and 1 to 4 percent over the optimum moisture content for the fine-grained soils in order to readily achieve the required compaction. Current moisture contents of the upper soils are generally well above the optimum moisture content; moisture conditioning (drying) will be required. Compacted fills should not be allowed to dry out prior to covering. If the fills are allowed to dry out, additional moisture conditioning and processing will be required.

4.3.6 Trench/Wall Backfill

Utility trench backfill consisting of the on-site materials or imported soil, or wall backfill consisting of granular material should be mechanically compacted in lifts. The on-site fine-grained soils and bedrock derived fill should not be placed as retaining wall backfill (fill placed within a distance of the wall equal to the height of the wall). Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning (drying) of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry (controlled low strength material, CLSM) may be substituted for compacted backfill. The slurry should contain two sacks of cement per cubic yard and have a maximum slump of 5 inches. When placed against retaining structures, the Project Structural Engineer should be consulted to determine the maximum wet slurry lift height of the wet slurry.

It is important that where utilities are placed within the layer of low-permeability fill, the trench backfill consists of the comparable low-permeability backfill. Alternatively, the trench backfill could consist of CLSM.

If open-graded rock is used as backfill, the material should be placed in lifts and mechanically densified. Open-graded rock should be separated from the on-site soils by a suitable filter fabric (Mirafi 140N or equivalent).

4.3.7 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 Foundation Type

The proposed structures may be supported on conventional spread footings founded in properly compacted fill or bedrock materials. To reduce the potential for moisture migration under the buildings from adjacent planters, we recommend continuous footings, extending at least 24 inches below grade, be constructed around the perimeter of each building.

4.4.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the natural and recompacted on-site materials, a static allowable net bearing pressure of up to 5,000 pounds per square foot (psf) may be used for continuous and isolated column spread footings for the proposed building embedded at least one foot into competent underlying bedrock materials. A static allowable net bearing pressure of up to 3,500 psf may be used for both continuous and isolated spread footings for buildings and minor structures underlain by engineered fill. These bearing pressures are for dead-plus-live-loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

For minor structures, such as site walls and property line screen walls, where lateral limits of the overexcavation may be limited, we recommend a maximum allowable bearing capacity of 1,500 pounds per square foot be used.

4.4.3 Minimum Footing Width and Embedment

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure.

Building Foundations Embedded in Competent Bedrock

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
5,000	30	36
4,000	18	36

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction. If interior footings are not fully loaded before the slab is in-place, the depth of interior footings may be taken from the top of the floor slab.

Foundations for Structures Underlain by Compacted Fill

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
3,500	36	24
2,500	24	24
1,500	18	24

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction. If interior footings are not fully loaded before the slab is in-place, the depth of interior footings may be taken from the top of the floor slab.

A minimum footing width of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

4.4.4 Estimated Settlements

Total static settlement of isolated pad or continuous wall footings (up to 300 kips for columns and 7 kips per lineal foot for walls) underlain by competent bedrock and minor structure foundations underlain by properly compacted fill is expected to be on the order of ¾-inch or less. Differential static settlement between similarly loaded column footings or along a 40-foot span of a continuous footing is expected to be on the order of ½-inch or less.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

4.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying materials and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill and/or bedrock. These values may be used in combination without reduction.

4.4.6 Foundation Inspection

Prior to placement of concrete and reinforcing steel, a representative of GPI should observe and approve foundation excavations.

4.4.7 Foundation Concrete

Laboratory testing by HDR (Table 1 in Appendix B) on selected samples soil and bedrock indicates that the near surface soils exhibit a soluble sulfate content ranging from 497 to 9,860 mg/kg. For the 2019 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for severe levels of soluble sulfate exposure from the on-site soils, (Category S2). Chloride levels in the samples tested were found to range from 11 to 35 mg/kg, which is considered to be low. Considering this and that the foundation concrete will be exposed to moisture, we recommend a chlorine exposure level of C1 as outlined in ACI 318. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3.

4.5 RETAINING STRUCTURES AND SHORING

The following recommendations are provided for basement and retaining walls up to 15 feet tall and shoring that does not extend more than 18 feet in height. We recommend that conventional retaining walls be backfilled as recommended in Section 4.3 of this report. The onsite clay soils and bedrock materials are considered to have medium to high expansion potential and should not be used as retaining wall backfill.

To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on subterranean walls, we recommend walls below grade be backfilled with granular soils following temporary sloped excavations in the bedrock made at an inclination of 1:1 to the outside edge of the spread foundation supporting the wall. Alternatively, if there is insufficient space for sloped excavations, we recommend the vertical excavation required to construct the subterranean wall be set back 3 feet from the back of the wall to the top of wall footing and the subterranean wall be backfilled with select low-permeability and non-expansive compacted fill material. The 3-foot recommended setback could be reduced to 8 inches provided the vertical excavation sidewalls are coated with a water-based membrane/vapor barrier (such as Liquid Boot) to seal the bedrock and reduce its exposure to oxygen, and compressible fill material is placed between the back of wall and bedrock surface.

Shoring, such as soldier piles constructed in drilled holes, may be used to support vertical excavation for the proposed basement level.

4.5.1 Basement and Retaining Walls

Active pressure may be used in the design of the subterranean walls if the total movement of the wall is sufficient to mobilize the active pressure (yielding at least ½-inch laterally in 10 feet of wall height). For cantilever walls with level, drained backfill comprised of imported granular soils, active pressures equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf) may be used in design. For cantilever walls with level backfill comprised of non-expansive and low-permeability fill soils or the wall is constructed against compressible fill material as discussed above, we recommend an active pressure equivalent to the pressures imposed by a fluid weighing 50 pounds per cubic foot (pcf) may be used in design.

At-rest pressures should be used for restrained walls or basement walls that remain rigid enough to be essentially non-yielding. At-rest earth pressures imposed by a fluid with an equivalent unit weight of 55 pounds per cubic foot may be used for drained, level, and granular backfill conditions. At-rest earth pressures imposed by a fluid with an equivalent unit weight of 70 pounds per cubic foot may be used for drained and level backfill comprised of non-expansive and low-permeability fills soils, or compressible fill material as discussed above, in close proximity to an exposed bedrock surface.

To account for seismic loads, an additional lateral earth pressure equal to 23 pcf (equivalent fluid pressure distribution) should be added to the above active pressure. If walls are designed using at-rest pressures, a total lateral earth pressure may be limited to the 70 pcf and 80 pcf, respectively, for the conditions sated above, when considering seismic loads.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of

100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders supported on the ground adjacent to the walls can impose lateral surcharge loads if they are supported adjacent to the basement walls (or shoring). Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed, the walls locally reinforced to support the surcharge from such loads.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. For retaining walls constructed adjacent to temporary shoring, a composite geotextile drain may be used with a manifold-type collection drain at the base of the wall. A composite geotextile drain system could also be used if the walls are backfilled with select non-expansive and low-permeability fills soils, or compressible fill material as discussed above

If the wall backfill is not drained, the combined earth and water pressures could be much higher. For undrained backfill conditions or for the portion of the wall backfill below bottom of the wall drain system, we recommend the cantilever and restrained (basement) walls be designed to resist an additional hydrostatic pressure equivalent to a fluid with a density of 42 and 32 pounds per cubic foot, respectively. This pressure should be added to the lateral earth pressures provide above.

As a minimum, if the walls below grade are drained, we recommend that they be damp-proofed to reduce the adverse effects of moisture intrusion into the structure. If additional protection is desired, the walls below grade should be water-proofed. Building walls with retained earth and walls designed for undrained conditions should also be waterproofed.

The Structural Engineer should specify the use of select and/or granular wall backfill on the plans for walls that are to be backfilled. Wall footings should be designed as discussed in the "Foundations" section.

Once preliminary design of the basement walls is completed and information regarding adjacent building foundations are known, we recommend GPI review the plans and provide lateral surcharge recommendations due to adjacent building foundations.

4.5.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes and backfilled with concrete. We do not anticipate that tie-back earth anchors or rakers will be required to laterally support the soldier piles. Utilities in the adjacent streets should be considered when planning the shoring.

Lateral Earth Pressures

For cantilever shoring with level backfill consisting of the on-site materials, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). It should be noted that the provided lateral earth pressure assumes a fully drained condition and do not include hydrostatic pressures.

Shoring subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained conditions, respectively. In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders supported on the ground adjacent to the shoring can impose lateral surcharge loads if they are supported adjacent to the shoring. Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed.

Soldier Piles and Lagging

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 600 pounds per square foot at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

Soldier piles are recommended to be installed in drilled holes. Driven/vibrated soldier piles are not recommended.

Continuous lagging will be required between the soldier piles where there is existing fill. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper.

Where bedrock is exposed, the excavation sidewalls between the soldier piles could be coated with a water-based membrane/vapor barrier such as Liquid Boot to seal the bedrock and reduce its exposure to oxygen. Lagging could then be placed between the soldier piles and the basement wall could then be constructed against the shoring system. We recommend that the voids between the lagging and bedrock be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

Shoring Deflection and Monitoring

It is difficult to accurately predict the amount of deflection of the shored excavation. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limit deflection to 1-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing buildings and/or other settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures. We recommend limiting the lateral deflection of shoring adjacent to any buildings to ½-inch or less.

We recommend performing a detailed survey of the improvements and existing structures to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the basement walls.

4.6 BUILDING FLOOR SLABS

The bedrock materials at the site have the potential to heave causing distress to proposed slab on grade floors. We recommend the following options for building floor slabs based on using design measures to allow for movement of the bedrock and/or reducing access of air to the freshly exposed bedrock.

Option 1: We recommend the building floor slabs be structurally supported on spread foundations and suspended above the bedrock a minimum of 12 inches. The space between the bottom of the floor slab and bedrock could be filled with compressible fill material that will support construction of the floor slab and compress as the bedrock materials expand. By providing space between the expansive bedrock and the floor slab, the material is allowed to expand without structural damage.

Option 2: We recommend the floor slab be underlain by a select non-expansive and low-permeability compacted fill material placed over the exposed bedrock to reduce the potential for future exposure of the bedrock to moisture changes and oxygen. The intent of the select material is to reduce bedrock access to air and provide sufficient non-expansive material to distribute localized uplift forces from the bedrock surface. Other impermeable coatings such as use of grout or bitumen products could also be placed on the bedrock to further reduce the potential exposure to oxygen. With Option 2, there is still a potential for floor slab heave. The underlying select compacted fill combined with a strengthened floor slab is intended to reduce and/or distribute the potential effects of localized heave occurring in the underlying bedrock.

For Option 2, the slab-on-grade floors should be supported on at least 36 inches of imported, non-expansive (EI< 20) soils. In addition, at least 24-inches of low-permeability (contain no less than 60 percent fines – portion passing No. 200 sieve) soils should be placed over the exposed bedrock to reduce the potential exposure of the bedrock to changes in moisture and to oxygen. The low-permeability soils may be considered as part of the non-expansive fill provided it meets the applicable criteria. The non-expansive and low-permeability soils should be placed as compacted fill soils as discussed in the "Placement and Compaction of Fills" section.

It is important that where utilities are placed within the 2-foot layer of low-permeability fill, the trench backfill consists of the comparable low-permeability backfill. Alternatively, the trench backfill could consist of CLSM.

For Option 2, we recommend a minimum floor slab on-grade thickness of 6 inches with reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Both the slab-on-grade thickness and reinforcing should be confirmed by the Structural Engineer, as structural loads on the floor slab may govern these items. Option 1 includes a structurally supported slab.

For elastic design of slabs-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 75 pounds per cubic inch (pounds per square inch per inch of deflection) may be used for imported non-expansive soils. The structural design should consider both long-term loads related to building operations and short-term construction loads.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.) or will be storing moisture sensitive supplies. Currently, common practice is to use a 15-mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water to cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations), as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

4.7 EXTERIOR CONCRETE FLATWORK (PEDESTRIAN HARDSCAPE)

Exterior concrete pads and pedestrian hardscape should be supported on an 18-inch-thick layer of non-expansive soil. This includes exterior sidewalks, stamped concrete, non-traffic pavement, and concrete ramps and stairs. Prior to placement of concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report. We suggest minimum reinforcement of No.3 rebar spaced at 18-inches on center be used in concrete pads and pedestrian hardscape to help reduce the potential distress due to expansive materials.

If landscape planters are planned adjacent to building slab or pedestrian hardscape areas, we recommend the planters include a cut-off barrier (perimeter building footings may provide a suitable cut-off barrier for the building floor slab) to reduce the potential for landscape water to migrate beneath the floor slab or pedestrian hardscape, saturate the expansive materials, and cause swelling.

4.8 PAVEMENTS

A test on the upper soils resulted in an R-value of 13. Due to variability in subsurface conditions, we have used an R-value of 10 in our design. The following pavement sections are recommended for planning purposes only. These recommendations assume that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended for typical traffic uses:

ASPHALT CONCRETE PAVEMENT ON UNTREATED SUBGRADE

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT CONCRETE	AGGREGATE BASE COURSE
Auto Parking	4	3	6
Auto Drives	5.5	3.5	10
Truck Traffic	7	4	14

PORTLAND CEMENT CONCRETE PAVEMENT ON 3-INCHES OF AGGREGATE BASE OVER SUBGRADE

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)		
		f’c = 3,000 psi PCC	f’c = 3,500 psi PCC	f’c = 4,000 psi PCC
Auto Parking/Drives	5.5	7	6.5	6
Truck Traffic	7.0	8	7.5	7

Because of the clay soils anticipated in the finished subgrade within the planned pavement areas, we recommend portland cement concrete (PCC) pavement be underlain by 3 inches of aggregate base. Besides improving overall support, the aggregate base will serve to maintain the moisture content of the properly compacted clays and provide a working surface prior to the placement of the PCC.

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation". The subgrade soils should not be allowed to dry-out prior to covering with the aggregate base or pavement, and a representative from GPI should test the subgrade moisture content immediately prior to covering. If the soils are allowed to dry-out, additional processing and moisture conditioning will be required to achieve the moisture contents discussed previously in the Placement and Compaction of Fills section of this report.

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except processed miscellaneous base).

The above recommendations assume that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.9 CORROSION

Resistivity and soluble sulfate testing of representative samples of the on-site soils and bedrock indicates that they are severely corrosive to buried ferrous metals and concrete. The bedrock materials are considered to be generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. Soil corrosion with regards to foundation concrete was addressed in a prior section of this report. GPI does not practice corrosion protection engineering. If corrosion protection recommendations are required, a corrosion engineer such as HDR should be consulted to provide recommendations to protect these elements from corrosion.

4.10 DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings. If planters are planned adjacent to pedestrian hardscape or pavement, we recommend that the planters be lined and drained to reduce the potential for water to infiltrate into the adjacent expansive soils.

4.11 STORMWATER INFILTRATION

To the depth explored (approximately 50 feet below existing grade) the site materials consist of low permeable soils and relatively non-permeable bedrock. Accordingly, we do not recommend stormwater infiltration at the site.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for The Commons at Calabasas, LLC c/o Caruso and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on projects other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If others perform the construction phase services, they must accept full responsibility for all geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER
PATRICK I.F. MCGERVEY
No. C 90770
CIVIL
STATE OF CALIFORNIA

Patrick I.F. McGervey, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
JUSTIN J. KEMPTON
No. 2385
GEOTECHNICAL
STATE OF CALIFORNIA

Justin J. Kempton, G.E.
Principal

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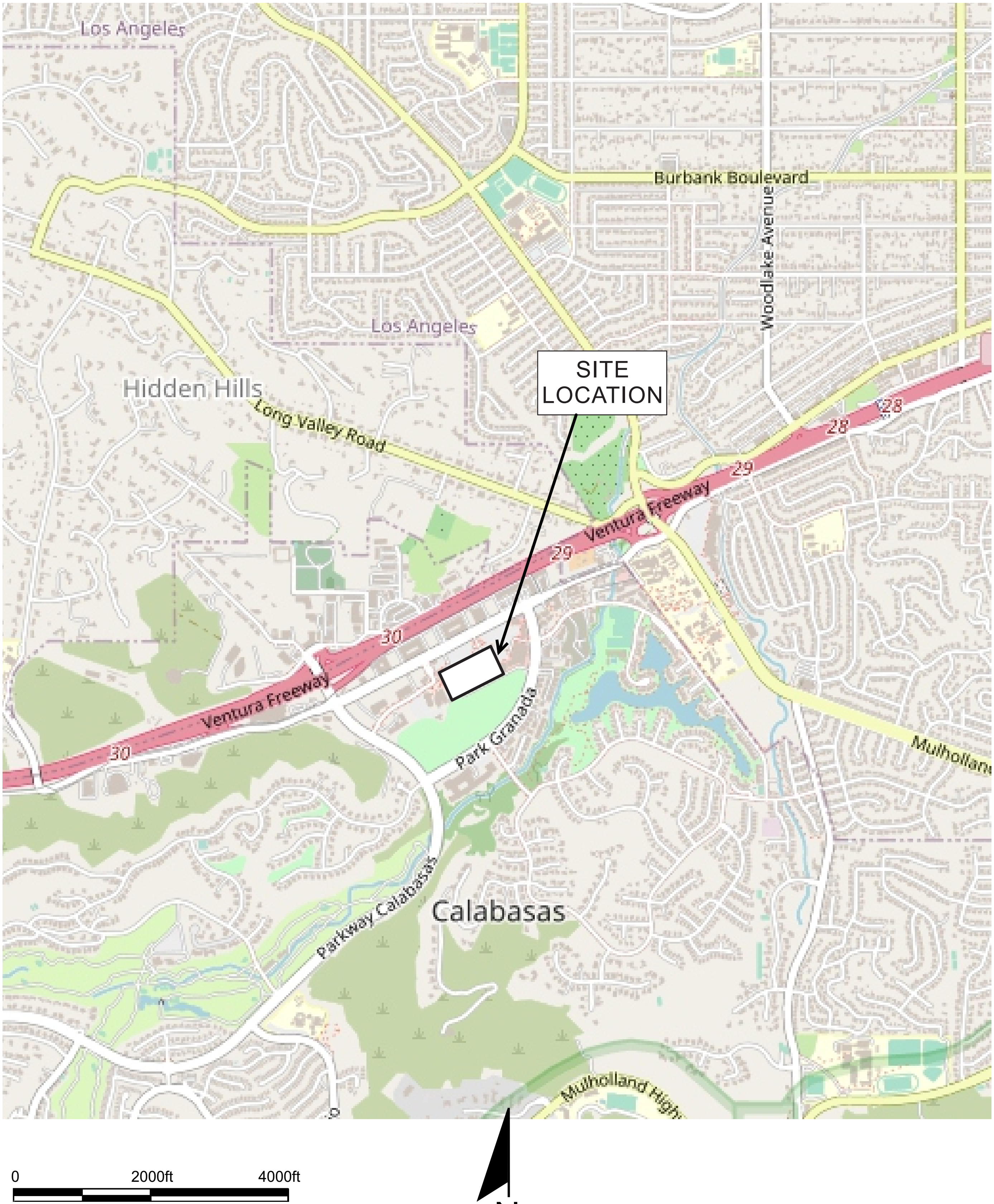


BASE MAP REPRODUCED FROM © CALTOPO

GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I	SCALE: 1" = 2000 FEET

SITE LOCATION

FIGURE 1

EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

0 150 300 FEET

BASE MAP REPRODUCED FROM SITE PLAN BY ELKUS MANFREDI ARCHITECTS, UNDATED

GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS - RETAIL/RESTAURANT BUILDINGS

GPI PROJECT NO.: 3063.I | SCALE: 1" = 150'

SITE PLAN

(Proposed Conditions)

FIGURE 2

EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

0 150 300 FEET

N

BASE MAP REPRODUCED FROM GOOGLE EARTH @ 2021

GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS - RETAIL/RESTAURANT BUILDINGS

GPI PROJECT NO.: 3063.I	SCALE: 1" = 150'

SITE PLAN

(Existing Conditions)

FIGURE 3

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling seven exploratory borings. Six of the borings (B-1 through B-6) were drilled in August of 2021, and one of the borings (B-7) was drilled in October of 2022. The borings were advanced to depths ranging from approximately 21 to 51 feet below the existing ground surface. The locations of the explorations are shown on Figures 2 and 3, Site Plan (Proposed Conditions and Existing Conditions).

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-7 in this appendix.

The borings were backfilled with drill cuttings and patched with cold patch asphalt. Drilling permits (permit nos. PW210025 and PW2200307) were obtained from the City of Calabasas.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from ALTA/NSPS Land Title As-Built Survey of 4710-4799 Commons Way plans by Hennon Surveying and Mapping, Inc. dated September 18, 2020.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	4-Inch AC over 6-Inch BASE	
						Fill: **SANDY CLAY (CL)** light brown, slightly moist	970
	29.2	80	80/11"	D		Natural: **SILTSTONE** light reddish brown, moist to very moist, hard, with fine grained sand, friable	
	17.2	118	50/5"	D	5		
	21.5	93	82/11"	D			965
	27.5	87	72/11"	D	10	@ 10 feet, dark brown	
							960
	25.1	91	74/10"	D	15		
							955
	25.8	91	50/6"	D	20		
						Total Depth 21 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0		3-Inch AC over 5-Inch BASE	
							Fill: **SANDY CLAY (CL)** brown, slightly moist	
	35.2	83	89	D			Natural: **SILTSTONE** light reddish brown, moist to very, hard, friable	
	33.7	82	67	D	5			965
	28.5	79	62/9"	D			@ 7 feet, with fine sand	
			50/4"	D	10		@ 10 feet, no recovery	960
	26.3	89	50/5"	D	15		@ 15 feet, dark brown	955
	25.0	94	59/11"	D	20			950
							Total Depth 21 feet	

SAMPLE TYPES
C Rock Core
S Standard Split Spoon
D Drive Sample
B Bulk Sample
T Tube Sample

DATE DRILLED:
8-27-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	5-Inch AC over 6-Inch BASE	
					Fill: **SANDY CLAY (CL)** dark brown, moist	970
29.4	85	50	D		Natural: **SILTSTONE** dark brown, moist to very moist, hard, friable	
33.1	83	68	D	5		
34.9	79	83	D			965
26.6	81	50/6"	D	10	@ 10 feet, with sand	
						960
32.2	75	70	D	15	@ 15 feet, dark brown	
						955
21.1	79	77	D	20		
						950
				25		
						945
22.0	96	50/5"	D	30		
						940
				35		
						935

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 8-26-21
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	25.3	69	50/5"	D	40		**SILTSTONE** dark brown, moist to very moist, hard, friable	
							Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	3-Inch AC over 4-Inch BASE	980
	19.5	98	18	D		Fill: **SILTY CLAY (CL)** mottled light brown and dark brown, moist, stiff	
	23.9	97	17	D	5		975
	31.6	88	71	D		Natural: **SILTSTONE** light red brown, moist to very moist, hard, friable	
	31.7	88	78/9"	D	10		970
	22.1	96	50/5"	D	15	@ 15 feet, dark brown	965
	21.4	100	75/10"	D	20		960
					25		955
	21.9	90	50/5"	D	30		950
					35		945

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-27-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-4

FIGURE A-4

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	20.5	80	50/5	D	40	**SILTSTONE** dark brown, moist to very moist, hard, friable	940
						Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-27-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	4-Inch AC over 6-Inch BASE	
					Fill: **SILTY CLAY (CL)** brown, slightly moist, hard, trace sand	
24.0	95	98/9"	D			
					Natural: **SILTSTONE** red brown, moist to very moist, hard, friable, trace sand	
				5		985
23.8	95	50/5"	D		@ 5 feet, dark brown	
23.3	88	505"	D			
				10		980
22.0	103	50/5"	D			
				15		975
19.2	93	50/5"	D			
				20		970
21.9	85	62/9"	D			
				25		965
				30		960
21.1	79	50/5"	D			
				35		955
						950

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-5

FIGURE A-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
21.0	84	50/6"	D	40	**SILTSTONE** dark brown, moist to very moist, hard, friable, trace sand	
				45		945
21.4	82	88/11"	D	50		940
					Total Depth 51 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	6-Inch AC over 8-Inch BASE	
						Fill: **SILTY CLAY (CL)** light brown, slightly moist	
	23.7	94	90/9"	D		Natural: **SILTSTONE** black, wet, hard, friable	
	22.7	95	87/11"	D	5		980
	21.7	98	50/6"	D			
	22.4	97	50/6"	D	10		975
	23.0	86	80/10"	D	15		970
	23.2	91	50/6"	D	20		965
					25		960
						Refusal @ 29 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-6

FIGURE A-6

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)
	26.2	85	50/6"	D	
	35.4	79			
	25.7	77	50/4"	D	5
	20.7		50/3"	D	
	13.3		50/4"	D	
	22.1	85	50/4"	D	10
	21.5	84	50/4"	D	
	26.6	91	50/4"	D	15
	25.8	84	50/5"	D	20
	26.2	90	50/5"	D	25
	26.1	75	50/5"	D	30
	23.5	83	50/5"	D	35

DESCRIPTION OF SUBSURFACE MATERIALS

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

DEPTH (FEET)	DESCRIPTION	ELEVATION (FEET)
0	4-Inch AC OVER 6-Inch BASE	
	Fill: **SANDY CLAY** brown, wet, very dense, with silt	970
	Natural: **SILTSTONE** red brown, wet, hard, with sand	
5		965
	@ 6 feet, very moist	
	@ 8 feet, moist	
10	@ 10 feet, very moist, dark brown	960
15	@ 15 feet, wet	955
20		950
25		945
30		940
35	@ 35 feet, very moist	935

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 10-6-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-7

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	20.6	78	50/4"	D	40			
							Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
10-6-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-7

FIGURE A-1

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the tables and figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

ATTERBERG LIMITS

The liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 to B-5.

COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D1557 on representative bulk samples of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-3	0 – 5	Mixture of Sandy Clay (CL) and Siltstone	95	23
B-4	0 - 5	Silty Clay (CL)	111	16

EXPANSION INDEX TEST

Expansion index tests were performed on representative bulk samples of the site soils. The tests were performed in accordance with ASTM D4829 to assess the expansion potential of the on-site soils. The results of the tests are summarized below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX, EI	EXPANSION POTENTIAL
B-3	0-5	Mixture of Sandy Clay (CL) and Siltstone	70	Medium
B-4	0-5	Silty Clay (CL)	37	Low
B-7	0-5	Mixture of Sandy Clay (CL) and Siltstone	58	Medium

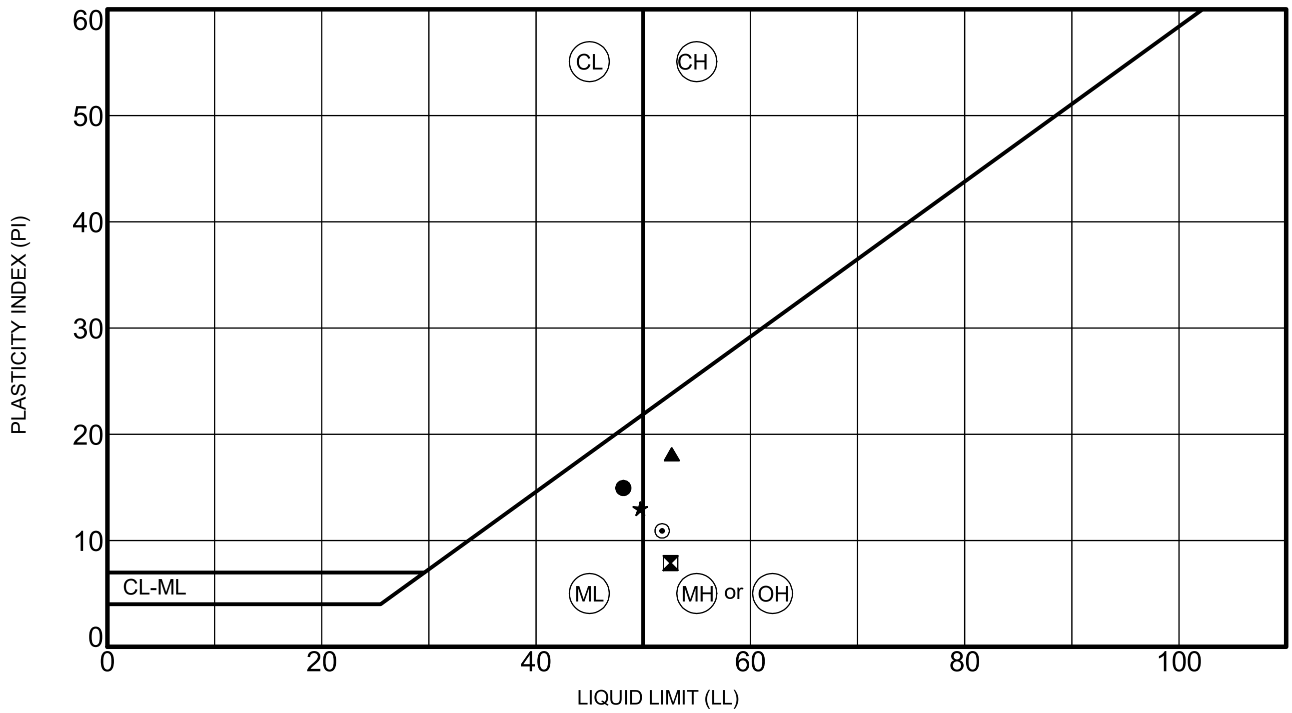
R-VALUE

Suitability of the near-surface soils for pavement was evaluated by conducting an R-value test. The test was performed in accordance with ASTM D2844 by GeoLogic Associates (GLA) under subcontract to GPI. The result of the test is as follows.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-3	0 – 5	Mixture of Sandy Clay (CL) and Siltstone	13

CORROSIVITY

Soil corrosivity testing was performed by HDR soil and bedrock samples provided by GPI. The test results are summarized in the tables by HDR included at the end of this Appendix.

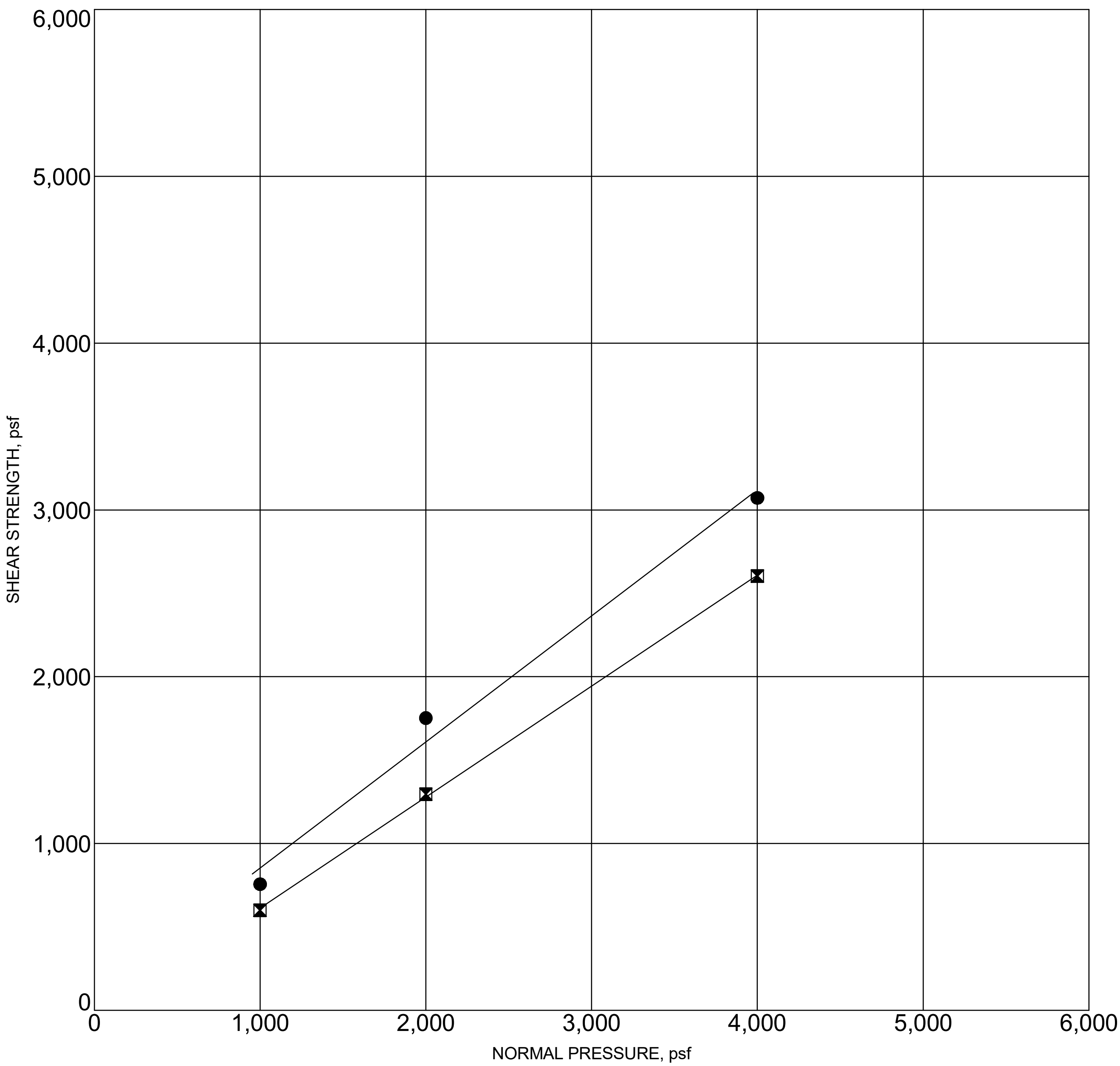


	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	0.0	48	33	15		SILTSTONE
⧗	B-2	5.0	53	45	8		SILTSTONE
▲	B-5	10.0	53	35	18		SILTSTONE
★	B-5	30.0	50	37	13		SILTSTONE
⊙	B-7	12.5	52	41	11		SILTSTONE

PROJECT: COMMONS AT CALABASAS PROJECT NO. 3063.I

GPI

ATTERBERG LIMITS TEST RESULTS



● **PEAK STRENGTH**
Friction Angle= 37 degrees
Cohesion= 96 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 33 degrees
Cohesion= 0 psf

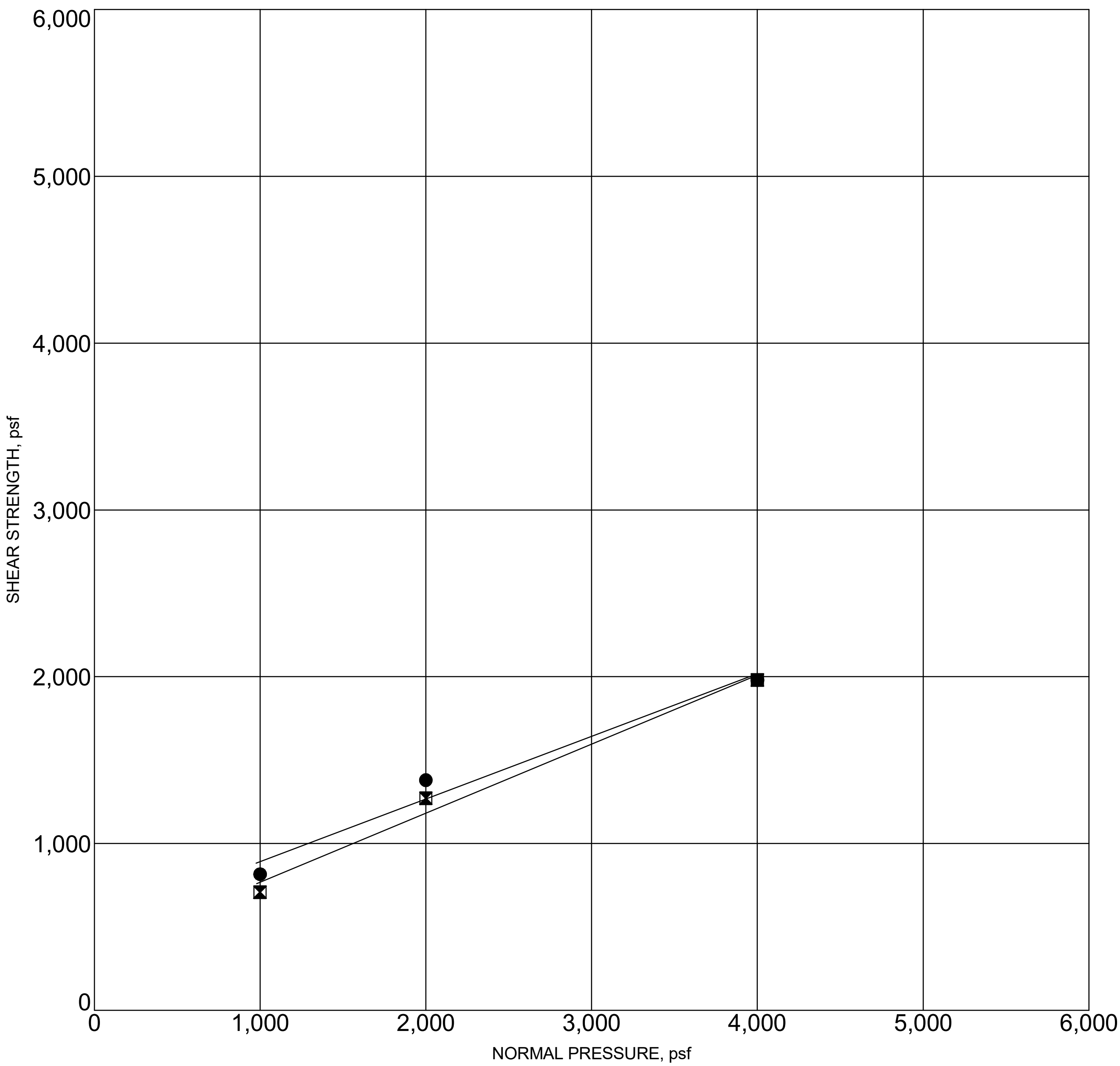
Sample Location		Classification	DD,pcf	MC,%
B-2	5.0	SILTSTONE	82	33.7

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 21 degrees
Cohesion= 516 psf

ULTIMATE STRENGTH
Friction Angle= 22 degrees
Cohesion= 354 psf

Note: Samples remolded to 90% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-3	0-5	MIXTURE OF SANDY CLAY (CL) AND SILTSTONE	86	23.0

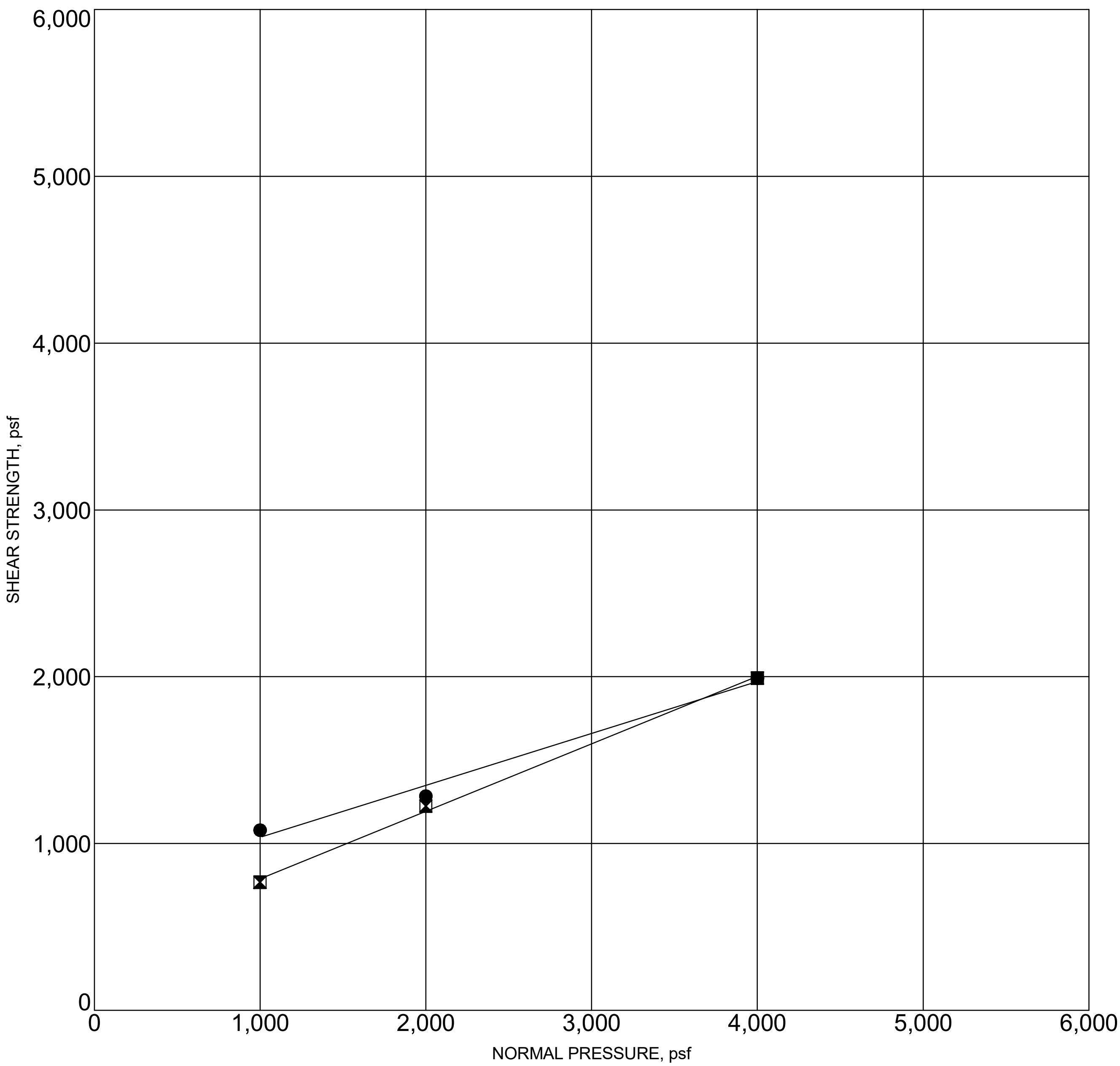
PROJECT: COMMONS AT CALABASAS

PROJECT NO.: 3063.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-3



● **PEAK STRENGTH**
Friction Angle= 17 degrees
Cohesion= 726 psf

ULTIMATE STRENGTH
Friction Angle= 22 degrees
Cohesion= 384 psf

Note: Samples remolded to 90% of maximum dry density.

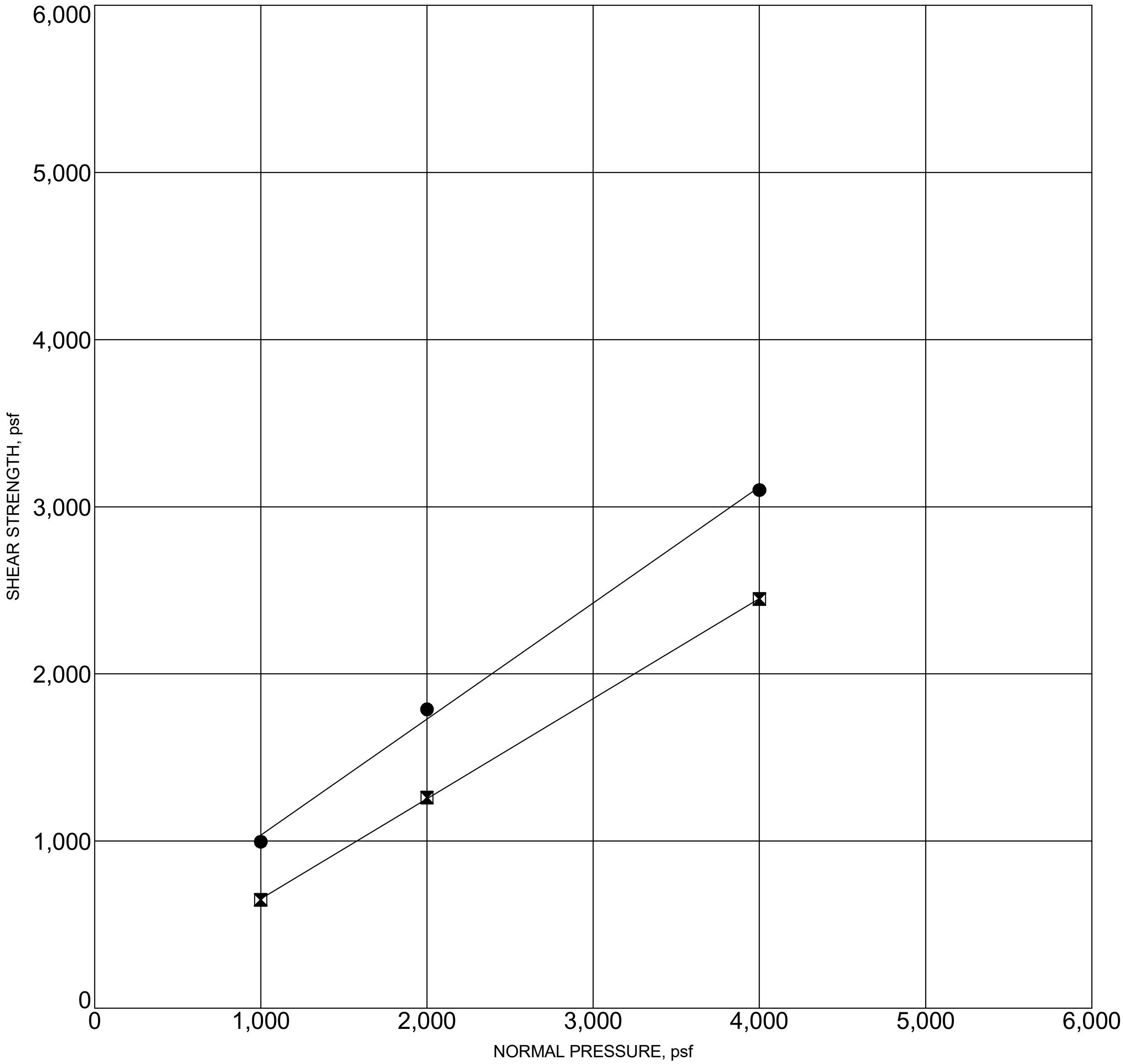
Sample Location		Classification	DD,pcf	MC,%
B-4	0-5	SILTY CLAY (CL)	100	16.0

PROJECT: COMMONS AT CALABASAS   PROJECT NO.: 3063.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 35 degrees
Cohesion= 340 psf

ULTIMATE STRENGTH
Friction Angle= 31 degrees
Cohesion= 54 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	10.0	SILTSTONE	97	22.4

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-5

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Commons Calabasas
Your #3063.I, HDR Lab #21-0820LAB
17-Sep-21

Sample ID

			B-3 @ 0-5'	B-4 @ 0-5'	B-1 @ 0-5'	B-4 @ 30'	B-6 @ 20'
Resistivity		**Units**					
as-received		ohm-cm	9,200	12,800	13,200	60,000	56,000
saturated		ohm-cm	680	1,120	960	440	600
pH			6.2	8.8	6.3	5.0	4.6
Electrical Conductivity		mS/cm	2.14	0.44	0.63	1.20	1.66
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	3,190	185	247	537	1,550
magnesium	Mg^{2+}	mg/kg	153	0.8	21	246	601
sodium	Na^{1+}	mg/kg	139	127	183	233	92
potassium	K^{1+}	mg/kg	154	23	100	186	174
ammonium	NH_4^{1+}	mg/kg	84	ND	ND	84	75
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	77	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	275	46	214	186	79
fluoride	F^{1-}	mg/kg	42	14	20	16	15
chloride	Cl^{1-}	mg/kg	21	19	35	14	11
sulfate	SO_4^{2-}	mg/kg	9,860	866	1,520	3,460	5,990
nitrate	NO_3^{1-}	mg/kg	61	6.5	20	3.6	14
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND	0.6	ND
Other Tests							
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
COMMONS CALABASAS
Your #3063.I, HDR Lab #22-0988LAB
24-Oct-22

Sample ID

			B-7 @ 6
Resistivity		**Units**	
as-received		ohm-cm	48,000
saturated		ohm-cm	1,720
pH			5.3
Electrical Conductivity		mS/cm	0.39
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	119
magnesium	Mg^{2+}	mg/kg	16
sodium	Na^{1+}	mg/kg	160
potassium	K^{1+}	mg/kg	89
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	107
fluoride	F^{1-}	mg/kg	3.1
chloride	Cl^{1-}	mg/kg	31
sulfate	SO_4^{2-}	mg/kg	497
nitrate	NO_3^{1-}	mg/kg	3.7
phosphate	PO_4^{3-}	mg/kg	20
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
THE COMMONS AT CALABASAS
4799 COMMONS WAY
CALABASAS, CALIFORNIA

Prepared for:
THE COMMONS AT CALABASAS, LLC
c/o CARUSO
101 The Grove Drive
Los Angeles, CA 90036

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 3063.I

December 23, 2022

GEOTECHNICAL
PROFESSIONALS INC.

December 23, 2022

The Commons at Calabasas, LLC
c/o Caruso
101 The Grove Drive
Los Angeles, CA 90036

Attention: Tasha Reeder
Project Manager, Construction

Subject: Report of Geotechnical Investigation
Proposed Residential Building
The Commons at Calabasas
4799 Commons way
Calabasas, California
GPI Project No. 3063.I

Dear Tasha:

Transmitted herewith is our updated draft report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have any questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Patrick McGervey, P.E.
Project Engineer
(pmcgervey@gpi-ca.com)

Justin J. Kempton, G.E.
Principal
(justink@gpi-ca.com)

Distribution: Addressee (PDF)

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LIST OF FIGURES

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APPENDIX A

APPENDIX C

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed mixed-use residential building at the subject site in Calabasas, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a new 8-story residential building approximately 43,500 feet in plan. The building will be constructed within the general location of the existing movie theatre building (Building I) in the Commons at Calabasas center. We were provided a general site layout showing the location and elevations of the proposed building in a Concept Design Package by Steinberg Hart, dated August 5, 2021 and a Conceptual Site Plan by Elkus Manfredi Architects (undated). The conceptual layout of the proposed building is shown on Figure 2.

The proposed building will be a podium style structure including 5 stories of residential over 2-stories of above-grade parking and one subterranean parking level. The finish floor elevation (FFE) of the at-grade parking level (P2) will be at about the same elevation as the existing FFE of Building I (Elevation +972 feet). The finish floor elevation of the lowest at-grade parking level (P3) will be at about Elevation +957 to +959 feet, approximately 13 to 15 feet below the existing grade of FFE of Building I. The proposed residential building will be of reinforced concrete construction for Parking Levels 1 through 3 and of wood frame construction for Residential Levels 1 through 5 (five levels). A swimming pool is planned on Residential Level 5. The project will also include parking lot improvements, minor site walls, pedestrian hardscape and landscaping in the remainder of the project area.

Access to Parking Level 2 will be from the main drive on the north side of the building at about Elevation +972. The south side of the Parking Levels 1 and 2 may be partially subterranean as the southern access drive is at about Elevation +983 to +993 feet.

The existing retail Buildings H and J, located adjacent to the northeast and northwest corners of Building I are planned to remain in place. The existing Barnes & Noble retail building (Building G), located immediately east of Building I, will also remain in place. An existing cantilever reinforced concrete retaining wall, located on the south side of Building I, that separates the existing buildings and the southern access drive, is also planned to stay in place, if possible. The cantilever retaining wall appears to have a maximum height on the order of 18 to 20 feet and to be supported on spread foundations.

On the south side of the building, between the building and the cantilever retaining wall supporting the southern access drive, a walkway approximately 6 to 8 feet wide is proposed to be removed and the building constructed against the existing retaining wall. On the south side of the southern access drive behind the building, an existing tieback retaining wall is supporting a vegetated ascending slope. The height of the tieback wall varies from no wall/zero feet on the west to approximately 45 feet on the east. We understand the tie-back wall was constructed before grading of the development in 1998.

In general, proposed finish floor elevations are anticipated to be within approximately 1 to 2 feet of existing grades. Proposed structural loads were not available at the time this report was prepared, but based on similar past projects we assume that maximum column and wall loads will be on the order of 900 kips and 12 kips per lineal foot, respectively (dead plus live loads).

Based on the Rough Grading Plans for the center dated March 2, 1998, and prior discussion with Caruso representatives, we understand grading of the center was originally performed in 1998 and construction of the adjacent buildings was completed in 1999. Based on the 1998 topography, grading of the site in 1998 included cuts on the order of 3 to 22 feet to the building pad elevation of +972.2 feet.

Because of the subsurface conditions, stormwater infiltration is not anticipated for the project.

Our recommendations are based upon the above structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to either confirm or modify our recommendations. Also, when the project grading and foundation plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork, and design of foundations and pavements.

2.0 SCOPE OF WORK

Our scope of work included review of existing data, subsurface exploration, laboratory testing, engineering analysis and the preparation of this report. We performed concurrent geotechnical investigations for the proposed mixed-use residential building discussed herein and the proposed retail/restaurant buildings (to be constructed north of the mixed-use residential building) and have incorporated the subsurface explorations and laboratory testing for both sites into this report. Evaluation of the reinforced concrete cantilever retaining wall and the tie-back wall are outside the scope of this study.

We were provided with prior geotechnical reports by others that addressed floor slab and flatwork distress (Kleinfelder, 2009 and Kleinfelder, 2017) for adjacent retail buildings in The Commons at Calabasas center. The subsurface soil information presented in the referenced reports was reviewed as part of this study.

Our subsurface exploration program consisted of a total of seven hollow stem auger borings performed to depths of approximately 21 to 51 feet below existing grades. Borings B-1 through B-6 were drilled in August 2021 and Boring B-7 was drilled in October 2022 as part of this report update. A description of field procedures and logs of the explorations are presented in Appendix A. The approximate locations of our subsurface explorations, as well as the locations of the nearby prior subsurface explorations by others are shown on the Site Plans, Figures 2 and 3.

Laboratory soil tests were performed on selected representative samples from the borings as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size analyses, Atterberg limits, shear strength, corrosivity, expansion index, R-value and maximum density. Laboratory testing procedures and results are summarized in Appendix B.

Corrosivity testing was performed by HDR under subcontract to GPI. R-value testing was performed by Geologic Associates under subcontract to GPI. Their test results are presented Appendix B.

Engineering evaluations were performed to provide earthwork criteria, foundation design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site is located in the existing movie theatre building (Building I) at The Commons at Calabasas shopping center. The center is surrounded by Calabasas Road to the north, Park Granada to the east, a slope leading up to Park Granada to the south, and the Calabasas City Hall and Library to the west. At the boring locations within the parking lot and drives surrounding the building, we encountered asphalt pavement sections consisting of 3 to 5 inches of asphalt concrete over 5 to 6 inches of aggregate base.

In general, the site grades on the south side of the building (in the southern access drive) range from approximately Elevation +983 to +993 feet. The floor slab grades within the existing Building I, the main entrance, landscape, and the walkways around the building range from approximately Elevation +971 to +972 feet. The finish floor elevations of adjacent Buildings G, H, and J are reportedly at +972.2, +972.7, and +971.2 feet, respectively based on the 1998 rough grading plans. They are generally within 6 inches of the finish floor elevation of Building I and the proposed building's at-grade Parking Level 2.

3.2 SUBSURFACE MATERIALS

Our field investigation disclosed a subsurface profile consisting of shallow fill soils overlying natural bedrock. Detailed descriptions of the conditions encountered are shown on the Log of Borings in Appendix A.

We encountered undocumented fills up to approximately two feet below existing grade in Boring B-1 through B-3 located in the existing parking lot just north of the proposed building location. Undocumented fill was also encountered up to 7 feet below existing grade in borings B-4 through B-6 drilled in the southern access road behind the cantilever retaining wall. The fill materials encountered consisted of slightly moist to moist silty and sandy clays. The fill materials are considered undocumented because documentation of the fill has not been made available for our review. It is likely these fills were placed during original grading of the center around 1998 and during backfill of the cantilever retaining wall. Expansion Index testing on representative samples of the sandy clay indicates the materials have a low to medium potential for expansion.

Prior nearby borings by others (Borings B-1 through B-3, Kleinfelder, 2009) encountered backfill behind the cantilever retaining wall that consisted of sandy clay and sandy silts to depths up to 16 feet below grade. Prior borings by others (Boring B-1 through B-4, Kleinfelder, 2017) were drilled in the walkway around the southern and eastern sides of Building I. These borings encountered a 4¾ to 6-inch concrete walkway underlain by 8 to 10 inches of base material. A concrete footing generally 13½ to 16 inches thick was encountered in their Borings B-1 and B-3 which appeared to either support Building I or the cantilever retaining wall.

Inside the adjacent Barnes & Noble retail store, the floor slab was underlain by 2½ to 4-inches of sand and 5 to 9 inches of gravel, (Kleinfelder, 2017). The As-Built Subdrain Plan (sheet C3-30) dated June 2, 1998, indicates that floor slabs in Buildings G, H, and I were to be underlain by 4 inches of sand (with visqueen) and 4 inches of gravel and gravel trenches spaced 25 to 30 feet apart that extend 4 inches deeper. The gravel drains are not shown on the referenced plan below Building I. It is likely that Building I is underlain by sand and gravel with or without the deepened gravel trenches. The gravel drainage trenches may have 4-inch diameter pipes within them.

The underlying natural materials encountered consisted of hard, moist to very moist, siltstone bedrock to the depths explored. According to reports by others (Kleinfelder, 2009 and Kleinfelder, 2017) the bedrock is mapped as the Modelo Formation. The bedrock materials have moderate to high strength and low compressibility characteristics. The moisture content of the siltstone bedrock materials was consistently moist to very moist within our explorations ranging from 17 to 35 percent with an average of 27 percent, which is about 4 percent above the optimum moisture content. Expansion Index testing on a representative remolded sample of the bedrock indicates the materials (when remolded) have a medium potential for expansion. Additional discussion regarding the expansion potential of the bedrock materials is presented in the following section.

The fill and bedrock materials encountered in our current explorations are comparable to materials reportedly encountered in prior nearby borings by others (Kleinfelder, 2009 and Kleinfelder, 2017).

Corrosivity testing of the upper site soils and bedrock materials indicates they are severely corrosive to buried metal and concrete elements. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. If corrosion recommendations are required, a corrosion engineer such as HDR should be consulted.

3.3 BEDROCK EXPANSION POTENTIAL

Prior testing of the bedrock materials (Kleinfelder, 2017) indicated that when remolded, the bedrock materials had a low to medium expansion potential and that the in-situ bedrock had a slight potential for moisture induced heave when the confining pressures were less than 200 pounds per square foot (psf). The in-situ bedrock materials had a relatively minimal potential for moisture induced heave when confining pressures were at least 1,500 to 2,000 psf.

Mineralogical testing was also performed previously on the bedrock materials within the upper 10 feet at the site by others (Kleinfelder, 2017) to determine the mineral constituents in the bedrock. Their testing concluded that all the samples tested contained over 2 percent pyrite (iron sulfide). Oxidation of pyrite minerals present in the bedrock will form gypsum crystals within the exposed bedrock fractures and surfaces that can result in ground expansion (Bryant, 2003).

The performance of the bedrock underlying adjacent buildings has been linked to significant differential movement (heave) of floor slabs at the center (Kleinfelder, 2017). Significant heave of floor slabs has resulted in cracked and distressed floors, partition walls, door jams, and other distress to lightly loaded fixtures and racks supported on floors. Differential heave of the bedrock has been attributed to a combination of unloading of the bedrock (by site grading), minor swelling due to changes in moisture content, and mineralogical changes in the bedrock. The oxidation of pyrite minerals to form gypsum within bedrock fractures and exposed surfaces was identified as the primary cause of differential floor slab movement (up to 5 inches) in the nearby Barnes and Noble store (Building G) located immediately south of the project area (Kleinfelder, 2017). Differential heave of floor slabs has also been documented in retail units east and west of the existing Barnes and Noble Store.

3.4 GROUNDWATER CONDITOINS

Groundwater was not encountered in our explorations to depths of 51 feet. However, moist to very moist bedrock materials were encountered at depths starting at approximately 2 feet below existing grades.

Published data by the California Geologic Survey (CGS 1998) does not map the historical high groundwater at the site, however it does indicate historical high groundwater at a depth of approximately 10 feet below the ground surface to the south and east of the site. Additionally, groundwater was encountered as shallow as 3 to 9½ feet below existing grades in prior borings by others (Kleinfelder, 2017) in the walkway south of the existing movie theatre building and between the existing movie theatre building and the Barnes and Noble Store.

Caving was not noted in the small diameter borings performed and is not expected to be a constraint during construction.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed, provided the geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed buildings are as follows:

- The bedrock materials encountered in our explorations for the proposed buildings are comparable to the materials identified in prior nearby explorations by others at the center that have historically been associated with localized differential heave of slab on grade floors. The heave has been attributed to unloading of the bedrock (site grading), changes in moisture content of the bedrock, and oxidation of pyrite minerals exposed in bedrock surfaces and fractures. The latter mechanism (oxidation of pyrite minerals) appears to be the predominant factor causing the observed heave related distress to slab on grade floors at the center. The formation of gypsum crystals results in expansion (swelling) of the underlying bedrock and the upward forces have been sufficient to heave slab on grade floors. Floor slab heave impacts partition walls, doors, fittings, and mechanical equipment in addition to other distress and nuisances caused by non-level and distressed floor slabs. If a subterranean parking floor slab were to heave, there is concern that reduced height limits in parking level would result.

- Options are provided in the report to mitigate the potential distress of floor slabs that could be caused by heave of the underlying bedrock. The optional recommendations are based on using design measures to allow for movement of the bedrock and/or reducing access of air (oxygen) to the freshly exposed bedrock and bedrock fractures. For Option 1, we recommend the building floor slabs be structurally supported on spread foundations and suspended above the bedrock a minimum of 12 inches. The space between the bottom of the floor slab and bedrock could be filled with compressible fill material that will support construction of the floor slab and compress as the bedrock materials expand. For Option 2, we recommend the floor slab be underlain by a select non-expansive and low-permeability compacted fill material placed over the exposed bedrock to reduce the potential for future exposure of the bedrock to moisture changes and oxygen.

- By observation, heave of foundations has not been historically noted at the center. This could potentially be attributed to the higher pressures imposed by foundations either counteracting potential uplift forces from the bedrock or impeding the oxidation process of the pyrite minerals.

- To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on subterranean walls, we recommend walls below grade be backfilled with granular soils following temporary sloped excavations in the bedrock made at an inclination of 1:1 to the outside edge of the spread foundation supporting the wall. Alternatively, if there is insufficient space for sloped excavations, we recommend the vertical excavation required to construct the subterranean wall be set back 3 feet from the back of the wall to the top of wall footing and the subterranean wall be backfilled with select low-permeability and non-expansive compacted fill material. The 3-foot recommended setback could be reduced to 8 inches provided the vertical excavation sidewalls are coated with a water-based membrane/vapor barrier (such as Liquid Boot) to seal the bedrock and reduce its exposure to oxygen and compressible fill material is placed between the back of wall and bedrock surface.

- We recommend the proposed building be supported on shallow spread foundations underlain uniformly by bedrock materials. We recommend that stepped footings be avoided. Minor structures such as site walls, canopies, and short retaining walls may be supported on shallow foundations underlain by compacted fill.

- The on-site clay soils have a low to medium expansive potential and will shrink and swell with changes in moisture content. We recommend concrete pedestrian hardscape be supported on select non-expansive fill.

- Undocumented fills were reported to depths of up to 2 feet below existing grade in the parking lot north of the building sit. Based on the referenced 1998 rough grading plans, significant fill materials other than the sand and gravel making up the existing capillary break and possible trench drains are not anticipated within the building footprint. Although the fills were likely placed during original rough grading in 1998, the fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. For the proposed improvements, we recommend removal and recompaction of the fill to provide uniform support for floor slabs. Below new pavements, we recommend the existing fill be scarified, moisture conditioned and recompacted in place prior to placement of new fill or aggregate base and paving.

- Current moisture contents of the bedrock and overlying fill soils are moist to very moist and at and above optimum moisture content. As such, mixing/discing and extensive moisture conditioning will be required to achieve suitable moisture contents of the onsite fill soils. The onsite bedrock materials are not considered suitable for use as compacted fill within 3 feet of the building footprint. The earthwork contractor should evaluate the moisture content of the existing soils when planning the earthwork.

- With the observed shallow groundwater encountered in some of the borings drilled by others adjacent to the existing cinema building, there is a likely potential for localized water seepage to impact the proposed subterranean parking level. Accordingly, provisions should be implemented to protect the structure from hydrostatic pressures and minimize the lateral migration of groundwater and/or seepage into the structure. Recommendations are presented in this report for a perimeter wall drain and subfloor drain system for the subterranean parking level.

- Corrosivity testing of the upper site soils and bedrock materials indicates they are severely corrosive to buried metal and concrete elements. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. If corrosion recommendations are required, a corrosion engineer such as HDR should be consulted.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

The site is in a seismically active area of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2019 or 2022 California Building Code (CBC) criteria. We do not anticipate significant changes with respect to Site Class and seismic design parameters discussed herein. For the 2019 CBC, a Site Class C may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate seismic design maps, the corresponding seismic design parameters from the CBC are as follows:

2019 CBC:

$S_S = 1.619g$	$S_{MS} = F_a * S_S = 1.942g$	$S_{DS} = 2/3 * S_{MS} = 1.295g$
$S_1 = 0.57g$	$S_{M1} = F_V * S_1 = 0.815g$	$S_{D1} = 2/3 * S_{M1} = 0.543g$

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the Malibu Coast Fault, which is located 7.6 miles from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.80 for a mean magnitude 6.8 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2019 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture at this site due to faulting is considered unlikely.

4.2.4 Liquefaction and Seismic Settlement

Soil liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is not located within an area mapped by the State of California as having a potential for soil liquefaction (Calabasas Quadrangle, CGS 1997). Due to the bedrock materials beneath the proposed structures, the potential for liquefaction and associated settlement at the site is considered to be low.

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. Due to the bedrock materials beneath the proposed structures, the potential for seismic ground subsidence to adversely affect the site is considered to be low.

4.3 EARTHWORK

The earthwork for the planned improvements is anticipated to consist of clearing and excavation of undocumented fill and bedrock materials, as necessary, subgrade preparation, importing select soils for placement beneath floor slabs, pedestrian hardscape and behind subterranean walls, and the placement and compaction of fill. Site grading is anticipated to include cuts up to approximately 15 feet for the subterranean level and fills up to 3 feet.

With favorable weather, we anticipate active mechanical drying using earthwork equipment such as a disc will be a feasible option to lower the soil moisture content. In the rainy season, we would anticipate significantly longer drying times or the need for drying with cement treatment.

4.3.1 Clearing

Prior to grading, performing excavations or constructing the proposed improvements, the areas to be developed should be cleared of existing structures, debris, and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during clearing should be removed from the site. Existing vegetation should not be mixed into the soils. Inert demolition debris, such as concrete and asphalt, may be crushed for reuse in engineered fills in accordance with the criteria presented in the “Material for Fill” section of this report.

Although not encountered in our explorations, and unlikely at the site, if cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill, excavation of the proposed subterranean level, footing excavations, and trenching for proposed utility lines.

Removals and Overexcavation

To provide uniform support for the planned improvements and to mitigate the potential impacts of expansive materials, prior to placement of fills or construction of the building, the existing fill soils and a portion of the bedrock within the proposed building pad should be removed and replaced with select materials. Removals for the building pad should extend to the minimum depths presented below for the applicable floor slab support options.

- For (Option 1) floor slabs that will be structurally supported on spread foundations and suspended above exposed bedrock materials, removals should extend to a depth below floor slabs that will be sufficient to place at least 12 inches of compressible fill materials between the exposed bedrock and bottom of floor slab.

- For (Option 2) floor slabs that will be supported on select compacted fill, removals should extend at least 3 feet below bottom of floor slab. For this option, the floor slab is recommended to be underlain by at least 3 feet of 'non-expansive’ compacted fill with at least the lower 24 inches placed on the exposed bedrock consisting of ‘low-permeability’ fill. These materials are further defined in Section 4.3.4 of this report.

For planning purposes, if the foundations are embedded in bedrock materials, removals below the proposed foundations are not required unless the bedrock materials are disturbed. The actual depths of removal should be determined in the field during grading by a representative of GPI.

For minor at-grade supported structures, such as site walls, canopies, and short retaining walls, the existing fills should be removed and the footings should be underlain by at least 2 feet of properly compacted, non-expansive fill or the undisturbed bedrock materials. Removals below pedestrian hardscape should extend at least 18-inches below finished subgrade so that at least 18 inches of imported non-expansive fill can be placed below pedestrian hardscape. For pavement outside the building, removals are not required unless the depth of disturbance exceeds 6-inches.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally at least 5 feet beyond the building lines or edge of foundations, or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls.

Excavation of the soils and bedrock at the site should be readily achieved using conventional methods. Difficulties in drilling our borings in the upper bedrock were not encountered.

We recommend excavations of the bedrock be conducted with the least possible disturbance of the bedrock below grade and behind basement walls. Fracturing of the bedrock provides increased access to air and promotes the expansion of the bedrock by the pyrite oxidation process.

Existing Utility Trenches

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pads. This is especially important for deeper fills associated with existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities that are 5 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan. Wet utilities left in-place outside building areas should be capped to reduce the potential for water to infiltrate into the building pad.

Temporary Sloped Excavations

The upper soils and bedrock at the site are expected to have a low caving potential when exposed in open cuts. Excavations in bedrock should be evaluated by a geologist of GPI to evaluate the excavations for the presence of adverse bedding conditions. Temporary construction excavations may be made vertically into the existing fill to depths of 4 feet and undisturbed bedrock to a depth of 5 feet below adjacent grade, without shoring. For cuts up to 12- and 20- feet deep, the slopes should be properly shored or sloped back to at least 1:1 and 1½:1 (H:V), respectively, or flatter. The allowable slope inclinations are measured from the toe to the top of the cut. Even at these inclinations, some raveling should be anticipated. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards. Recommendations for temporary shoring are provided in Section 4.6 of this report.

Temporary Dewatering

Groundwater was not encountered in our explorations to depths of 51 feet. However, moist to very moist bedrock materials were encountered at depths starting at approximately 2 feet below existing grades in recent borings by GPI, and groundwater was encountered as shallow as 3 to 9½ feet below existing grades in prior borings by others (Kleinfelder, 2017) in the walkway south of the existing movie theatre building and between the existing movie theatre building and the Barnes and Noble Store.

Accordingly, there is a reasonable potential that groundwater seepage could be encountered during site excavations. We anticipate the groundwater seepage, should it manifest in the open excavation, will occur from local areas at the soil/ bedrock contact and/or from local fractures in the bedrock. During site excavation, we recommend the contractor be prepared to construct shallow gravel-filled temporary trenches to direct groundwater seepage to a sump and pump system to discharge the collected water.

4.3.3 Subgrade Preparation

Loose or disturbed soils should be removed from the subgrade prior to placement and compaction of the overlying fill soils. Scarification of the bedrock subgrade is not required.

In areas to receive pavements (outside of the structures), and where fill is to be placed over existing fill evaluated to be suitable to leave in place, the upper 12 inches of the exposed subgrade soils should be scarified, moisture-conditioned, and compacted to a minimum of 90 percent of the maximum dry density.

4.3.4 Material for Fill

The on-site soils and bedrock materials are, in general, suitable for use as compacted fill with the exception of retaining wall backfill or placed within 36 inches of the finished subgrade for floor slabs and within 18 inches of the finished subgrade for pedestrian hardscape. In general, we recommend the onsite bedrock materials not be used as compacted fill within 3 feet of the building footprint.

We recommend the soils placed within 36 inches of the finished subgrade for floor slabs and within 18 inches of the finished subgrade for pedestrian hardscape consist of imported, non-expansive (EI< 20) soils. In addition, the lower 24-inches of select materials placed under the floor slabs and directly over the exposed bedrock should consist of low-permeability (contain no less than 60 percent fines – portion passing No. 200 sieve) soils to reduce the potential exposure of the bedrock to changes in moisture and to oxygen. The low-permeability soils may be considered as part of the non-expansive fill provided it meets the applicable criteria.

Soils used for general wall backfill should be predominately granular (contain no more than 40 percent fines – portion passing No. 200 sieve) and non-expansive (E.I. less than 20). To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on the subterranean walls, the subterranean walls supporting bedrock materials should either be backfilled with select low-permeability and non-expansive compacted fill material that extends at least 3 feet laterally beyond the back of wall or the walls should be constructed adjacent to bedrock that is sealed immediately after excavation.

Suitable 'non-expansive' and 'low-permeability/non-expansive' soils are not anticipated to be available on-site and will need to be imported. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of debris and pieces larger than 6 inches in greatest dimension. If approved by the client and regulatory agencies, the on-site portland cement concrete and asphalt concrete can be crushed/pulverized and mixed with the on-site soils prior to performing the overexcavation. The material should be crushed so that the resulting particle size is less than 3 inches in diameter if used for stabilization, and it should be mixed with the on-site soils if used for general fill. If used to support pavements, it should be crushed to meet the specifications of Caltrans Class II or Greenbook crushed miscellaneous base (CMB).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. The aggregate base material should be compacted to a relative compaction of at least 95 percent. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field.

The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	3-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-9 inches
Scrapers, Heavy loaders, and large vibratory rollers	9-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content for the sandy soils and 1 to 4 percent over the optimum moisture content for the fine-grained soils in order to readily achieve the required compaction. Current moisture contents of the upper soils are generally well above the optimum moisture content; moisture conditioning (drying) will be required. Compacted fills should not be allowed to dry out prior to covering. If the fills are allowed to dry out, additional moisture conditioning and processing will be required.

4.3.6 Trench/Wall Backfill

Utility trench backfill consisting of the on-site materials or imported soil, or wall backfill consisting of granular material should be mechanically compacted in lifts. The on-site fine-grained soils and bedrock derived fill should not be placed as retaining wall backfill (fill placed within a distance of the wall equal to the height of the wall). Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning (drying) of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry (controlled low strength material, CLSM) may be substituted for compacted backfill. The slurry should contain two sacks of cement per cubic yard and have a maximum slump of 5 inches. When placed against retaining structures, the Project Structural Engineer should be consulted to determine the maximum wet slurry lift height of the wet slurry.

It is important that where utilities are placed within the layer of low-permeability fill, the trench backfill consists of the comparable low-permeability backfill. Alternatively, the trench backfill could consist of CLSM.

If open-graded rock is used as backfill, the material should be placed in lifts and mechanically densified. Open-graded rock should be separated from the on-site soils by a suitable filter fabric (Mirafi 140N or equivalent).

4.3.7 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.4 SUBDRAINS

With the observed shallow groundwater encountered in some of the borings drilled by others adjacent to the existing cinema borings, there is a likely potential for localized water seepage to impact the proposed subterranean parking level. Accordingly, provisions should be implemented to protect the structure from hydrostatic pressures and minimize the lateral migration of groundwater and/or seepage into the structure. Preliminary recommendations are provided below for a perimeter wall drain and subfloor drain system. Alternatively, the basement walls and floor slab would need to be designed to resist hydrostatic pressures and thoroughly waterproofed.

<u>Perimeter Wall Drain</u>

We recommend that a perimeter drainage system be installed behind the subterranean basement walls. We recommend that a perforated drain line be placed behind walls at the base of the retaining walls leading to sump areas equipped with automatic pumping units. The drain lines should be surrounded by at least 6 inches of Class II Permeable Base material. The backfill placed within 6 inches of the wall and more than 2 feet below the surface of the retained materials should also consist of Class II permeable materials. A proprietary drainage wall board could be used in lieu of the 6-inch Class II Permeable Base material behind the wall. Adjacent to exterior building walls, where the surface is not paved, a 2-foot-thick cap of relatively impermeable soils should be placed to restrict surface water from entering the wall drain system.

Subfloor Drains

Depending on the actual conditions exposed following excavation, a permanent subdrain system may be required below basement floor slab. A subfloor subdrain system (if required) could consist of a layer of filter material, approximately 12 inches in thickness, drained by subdrain pipes leading to sump areas equipped with automatic pumping units. The filter material should consist of Class II Permeable Base. The drain lines should consist of perforated pipe placed with the perforations at the bottom of the filter layer. The drain lines should extend around the perimeter of the building and should be spaced approximately 50 feet apart within the interior of the building. The locations of the drain lines may be adjusted following evaluation of the exposed subgrade conditions.

If a structurally supported floor is used (Option 1), we recommend the filter materials and subdrain pipes be placed below the compressible fill materials. If the floor slab is to be supported on grade (Option 2), we recommend that the filter layer be placed above the 24-inch-thick layer of select 'low-permeability' and 'non-expansive fill' to be placed on the bedrock surface. The filter material would serve as the upper 12 inches of the recommended 36 inches of 'non-expansive' fill below floor slabs.

System Review

The means of draining the soils outside the basement walls will also depend on the selected method of shoring and the method of constructing the basement walls. We can provide additional information for design of the subdrain system as features for the system and the building plans are developed. In addition, we should be provided a copy of the design for review after the excavation has been completed. If necessary, the system can be modified as indicated by the observed conditions,

4.5 FOUNDATIONS

4.5.1 Foundation Type

The proposed structures may be supported on conventional spread footings founded in undisturbed bedrock materials. Minor structures such as site walls, canopies, and short retaining walls may be supported on shallow foundations underlain by compacted fill or undisturbed bedrock. To reduce the potential for moisture migration under the buildings from adjacent planters, we recommend continuous footings, extending at least 24 inches below grade, be constructed around the perimeter of the building.

4.5.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the bedrock materials, a static allowable net bearing pressure of up to 6,000 pounds per square foot (psf) may be used for continuous and isolated column spread footings for the proposed building embedded at least one foot into competent underlying bedrock materials. A static allowable net bearing pressure of up to 3,500 psf may be used for both continuous and isolated spread footings for buildings and minor structures underlain by engineered fill. These bearing pressures are for dead-plus-live-loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

For minor structures, such as site walls and property line screen walls, where lateral limits of the overexcavation may be limited, we recommend a maximum allowable bearing capacity of 1,500 pounds per square foot be used.

4.5.3 Minimum Footing Width and Embedment

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure.

Building Foundations Embedded in Competent Bedrock

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
6,000	48	36
5,000	30	36
4,000	18	36

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction. If interior footings are not fully loaded before the slab is in-place, the depth of interior footings may be taken from the top of the floor slab.

Foundations for Structures Underlain by Compacted Fill

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
3,500	36	24
2,500	24	24
1,500	18	24

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction. If interior footings are not fully loaded before the slab is in-place, the depth of interior footings may be taken from the top of the floor slab.

A minimum footing width of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

4.5.4 Estimated Settlements

Total static settlement of isolated pad or continuous wall footings (up to 900 kips for columns and 12 kips per lineal foot for walls) underlain by competent bedrock and minor structure foundations underlain by properly compacted fill is expected to be on the order of ¾-inch or less. Differential static settlement between similarly loaded column footings or along a 40-foot span of a continuous footing is expected to be on the order of ½-inch or less.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

4.5.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying materials and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill and/or bedrock. These values may be used in combination without reduction.

4.5.6 Foundation Inspection

Prior to placement of concrete and reinforcing steel, a representative of GPI should observe and approve foundation excavations.

4.5.7 Foundation Concrete

Laboratory testing by HDR (Table 1 in Appendix B) on selected samples soil and bedrock indicates that the near surface soils exhibit a soluble sulfate content ranging from 497 to 9,860 mg/kg. For the 2019 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for severe levels of soluble sulfate exposure from the on-site soils, (Category S2). Chloride levels in the samples tested were found to range from 11 to 35 mg/kg, which is considered to be low. Considering this and that the foundation concrete will be exposed to moisture, we recommend a chlorine exposure level of C1 as outlined in ACI 318. The bedrock materials are generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3.

4.6 RETAINING STRUCTURES AND SHORING

The following recommendations are provided for basement and retaining walls up to 15 feet tall and shoring that does not extend more than 18 feet in height. We recommend that conventional retaining walls be backfilled as recommended in Section 4.3 of this report. The onsite clay soils and bedrock materials are considered to have medium to high expansion potential and should not be used as retaining wall backfill.

To reduce the potential for increased lateral forces due to expansion of exposed bedrock materials on subterranean walls, we recommend walls below grade be backfilled with granular soils following temporary sloped excavations in the bedrock made at an inclination of 1:1 to the outside edge of the spread foundation supporting the wall. Alternatively, if there is insufficient space for sloped excavations, we recommend the vertical excavation required to construct the subterranean wall be set back 3 feet from the back of the wall to the top of wall footing and the subterranean wall be backfilled with select low-permeability and non-expansive compacted fill material. The 3-foot recommended setback could be reduced to 8 inches provided the vertical excavation sidewalls are coated with a water-based membrane/vapor barrier (such as Liquid Boot) to seal the bedrock and reduce its exposure to oxygen, and compressible fill material is placed between the back of wall and bedrock surface.

Shoring, such as soldier piles constructed in drilled holes, may be used to support vertical excavation for the proposed basement level.

4.6.1 Basement and Retaining Walls

Active pressure may be used in the design of the subterranean walls if the total movement of the wall is sufficient to mobilize the active pressure (yielding at least ½-inch laterally in 10 feet of wall height). For cantilever walls with level, drained backfill comprised of imported granular soils, active pressures equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf) may be used in design. For cantilever walls with level backfill comprised of non-expansive and low-permeability fill soils or the wall is constructed against compressible fill material as discussed above, we recommend an active pressure equivalent to the pressures imposed by a fluid weighing 50 pounds per cubic foot (pcf) may be used in design.

At-rest pressures should be used for restrained walls or basement walls that remain rigid enough to be essentially non-yielding. At-rest earth pressures imposed by a fluid with an equivalent unit weight of 55 pounds per cubic foot may be used for drained, level, and granular backfill conditions. At-rest earth pressures imposed by a fluid with an equivalent unit weight of 70 pounds per cubic foot may be used for drained and level backfill comprised of non-expansive and low-permeability fills soils, or compressible fill material as discussed above, in close proximity to an exposed bedrock surface.

To account for seismic loads, an additional lateral earth pressure equal to 23 pcf (equivalent fluid pressure distribution) should be added to the above active pressure. If walls are designed using at-rest pressures, a total lateral earth pressure may be limited to the 70 pcf and 80 pcf, respectively, for the conditions sated above, when considering seismic loads.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders supported on the ground adjacent to the walls can impose lateral surcharge loads if they are supported adjacent to the basement walls (or shoring). Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed, the walls locally reinforced to support the surcharge from such loads.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. For retaining walls constructed adjacent to temporary shoring, a composite geotextile drain may be used with a manifold-type collection drain at the base of the wall. A composite geotextile drain system could also be used if the walls are backfilled with select non-expansive and low-permeability fills soils, or compressible fill material as discussed above.

The recommended perimeter wall drain outlined in Section 4.4 will also serve to prevent the build-up of hydrostatic pressures on the basement walls.

If the wall backfill is not drained, the combined earth and water pressures could be much higher. For undrained backfill conditions or for the portion of the wall backfill below bottom of the wall drain system, we recommend the cantilever and restrained (basement) walls be designed to resist an additional hydrostatic pressure equivalent to a fluid with a density of 42 and 32 pounds per cubic foot, respectively. This pressure should be added to the lateral earth pressures provide above.

As a minimum, if the walls below grade are drained, we recommend that they be damp-proofed to reduce the adverse effects of moisture intrusion into the structure. If additional protection is desired, the walls below grade should be water-proofed. Building walls with retained earth, basement walls with a perimeter wall drain, and walls designed for undrained conditions should also be waterproofed.

The Structural Engineer should specify the use of select and/or granular wall backfill on the plans for walls that are to be backfilled. Wall footings should be designed as discussed in the "Foundations" section.

Once preliminary design of the basement walls is completed and information regarding adjacent building foundations are known, we recommend GPI review the plans and provide lateral surcharge recommendations due to adjacent building foundations.

4.6.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes and backfilled with concrete. We do not anticipate that tie-back earth anchors or rakers will be required to laterally support the soldier piles. Utilities in the adjacent streets should be considered when planning the shoring.

<u>Lateral Earth Pressures</u>

For cantilever shoring with level backfill consisting of the on-site materials, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). It should be noted that the provided lateral earth pressure assumes a fully drained condition and do not include hydrostatic pressures.

Shoring subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained conditions, respectively. In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders supported on the ground adjacent to the shoring can impose lateral surcharge loads if they are supported adjacent to the shoring. Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed.

Soldier Piles and Lagging

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 600 pounds per square foot at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

Soldier piles are recommended to be installed in drilled holes. Driven/vibrated soldier piles are not recommended.

Continuous lagging will be required between the soldier piles where there is existing fill. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper.

Where bedrock is exposed, the excavation sidewalls between the soldier piles could be coated with a water-based membrane/vapor barrier such as Liquid Boot to seal the bedrock and reduce its exposure to oxygen. Lagging could then be placed between the soldier piles and the basement wall could then be constructed against the shoring system. We recommend that the voids between the lagging and bedrock be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper.

The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

Shoring Deflection and Monitoring

It is difficult to accurately predict the amount of deflection of the shored excavation. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limit deflection to 1-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing buildings and/or other settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures. We recommend limiting the lateral deflection of shoring adjacent to any buildings to ½-inch or less.

We recommend performing a detailed survey of the improvements and existing structures to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the basement walls.

4.7 BUILDING FLOOR SLABS

The bedrock materials at the site have the potential to heave causing distress to proposed slab on grade floors. We recommend the following options for building floor slabs based on using design measures to allow for movement of the bedrock and/or reducing access of air to the freshly exposed bedrock.

Option 1: We recommend the building floor slabs be structurally supported on spread foundations and suspended above the bedrock a minimum of 12 inches. The space between the bottom of the floor slab and bedrock could be filled with compressible fill material that will support construction of the floor slab and compress as the bedrock materials expand. By providing space between the expansive bedrock and the floor slab, the material is allowed to expand without structural damage.

Option 2: We recommend the floor slab be underlain by a select non-expansive and low-permeability compacted fill material placed over the exposed bedrock to reduce the potential for future exposure of the bedrock to moisture changes and oxygen. The intent of the select material is to reduce bedrock access to air and provide sufficient non-expansive material to distribute localized uplift forces from the bedrock surface. Other impermeable coatings such as use of grout or bitumen products could also be placed on the bedrock to further reduce the potential exposure to oxygen. With Option 2, there is still a potential for floor slab heave. The underlying select compacted fill combined with a strengthened floor slab is intended to reduce and/or distribute the potential effects of localized heave occurring in the underlying bedrock.

For Option 2, the slab-on-grade floors should be supported on at least 36 inches of imported, non-expansive (EI< 20) soils. In addition, at least 24-inches of low-permeability (contain no less than 60 percent fines – portion passing No. 200 sieve) soils should be placed over the exposed bedrock to reduce the potential exposure of the bedrock to changes in moisture and to oxygen. The low-permeability soils may be considered as part of the non-expansive fill provided it meets the applicable criteria. The non-expansive and low-permeability soils should be placed as compacted fill soils as discussed in the "Placement and Compaction of Fills" section.

It is important that where utilities are placed within the 2-foot layer of low-permeability fill, the trench backfill consists of the comparable low-permeability backfill. Alternatively, the trench backfill could consist of CLSM.

For Option 2, we recommend a minimum floor slab on-grade thickness of 6 inches with reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Both the slab-on-grade thickness and reinforcing should be confirmed by the Structural Engineer, as structural loads on the floor slab may govern these items. Option 1 includes a structurally supported slab.

For elastic design of slabs-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 75 pounds per cubic inch (pounds per square inch per inch of deflection) may be used for imported non-expansive soils. The structural design should consider both long-term loads related to building operations and short-term construction loads.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.) or will be storing moisture sensitive supplies. Currently, common practice is to use a 15-mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water to cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations), as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

4.8 EXTERIOR CONCRETE FLATWORK (PEDESTRIAN HARDSCAPE)

Exterior concrete pads and pedestrian hardscape should be supported on an 18-inch-thick layer of non-expansive soil. This includes exterior sidewalks, stamped concrete, non-traffic pavement, and concrete ramps and stairs. Prior to placement of concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report. We suggest minimum reinforcement of No.3 rebar spaced at 18-inches on center be used in concrete pads and pedestrian hardscape to help reduce the potential distress due to expansive materials.

If landscape planters are planned adjacent to building slab or pedestrian hardscape areas, we recommend the planters include a cut-off barrier (perimeter building footings may provide a suitable cut-off barrier for the building floor slab) to reduce the potential for landscape water to migrate beneath the floor slab or pedestrian hardscape, saturate the expansive materials, and cause swelling.

4.9 PAVEMENTS

A test on the upper soils resulted in an R-value of 13. Due to variability in subsurface conditions, we have used an R-value of 10 in our design. The following pavement sections are recommended for planning purposes only. These recommendations assume that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended for typical traffic uses:

ASPHALT CONCRETE PAVEMENT ON UNTREATED SUBGRADE

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT CONCRETE	AGGREGATE BASE COURSE
Auto Parking	4	3	6
Auto Drives	5.5	3.5	10
Truck Traffic	7	4	14

PORTLAND CEMENT CONCRETE PAVEMENT ON 3-INCHES OF AGGREGATE BASE OVER SUBGRADE

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)		
		f'c = 3,000 psi PCC	f'c = 3,500 psi PCC	f'c = 4,000 psi PCC
Auto Parking/Drives	5.5	7	6.5	6
Truck Traffic	7.0	8	7.5	7

Because of the clay soils anticipated in the finished subgrade within the planned pavement areas, we recommend portland cement concrete (PCC) pavement be underlain by 3 inches of aggregate base. Besides improving overall support, the aggregate base will serve to maintain the moisture content of the properly compacted clays and provide a working surface prior to the placement of the PCC.

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation". The subgrade soils should not be allowed to dry-out prior to covering with the aggregate base or pavement, and a representative from GPI should test the subgrade moisture content immediately prior to covering. If the soils are allowed to dry-out, additional processing and moisture conditioning will be required to achieve the moisture contents discussed previously in the Placement and Compaction of Fills section of this report.

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except processed miscellaneous base).

The above recommendations assume that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.10 CORROSION

Resistivity and soluble sulfate testing of representative samples of the on-site soils and bedrock indicates that they are severely corrosive to buried ferrous metals and concrete. The bedrock materials are considered to be generally acidic; each of the bedrock samples tested had pH values of 4.6 to 6.3. Soil corrosion with regards to foundation concrete was addressed in a prior section of this report. GPI does not practice corrosion protection engineering. If corrosion protection recommendations are required, a corrosion engineer such as HDR should be consulted to provide recommendations to protect these elements from corrosion.

4.11 DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings. If planters are planned adjacent to pedestrian hardscape or pavement, we recommend that the planters be lined and drained to reduce the potential for water to infiltrate into the adjacent expansive soils.

4.12 STORMWATER INFILTRATION

To the depth explored (approximately 50 feet below existing grade) the site materials consist of low permeable soils and relatively non-permeable bedrock. Accordingly, we do not recommend stormwater infiltration at the site.

4.13 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for The Commons at Calabasas, LLC c/o Caruso and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on projects other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If others perform the construction phase services, they must accept full responsibility for all geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER
PATRICK I.F. MCGERVEY
No. C 90770
CIVIL
STATE OF CALIFORNIA

Patrick I.F. McGervey, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
JUSTIN J. KEMPTON
No. 2385
GEOTECHNICAL
STATE OF CALIFORNIA

Justin J. Kempton, G.E.
Principal

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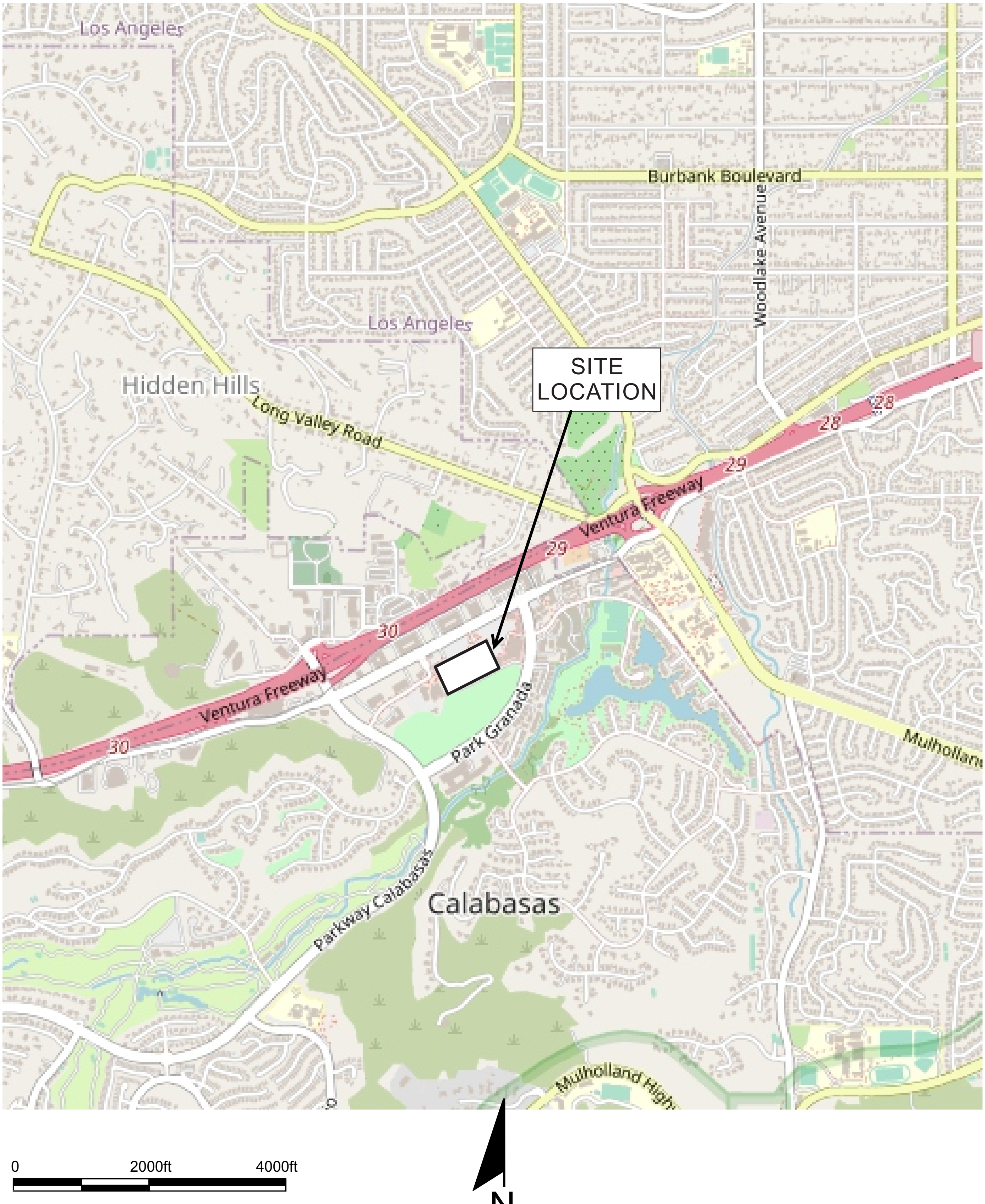


BASE MAP REPRODUCED FROM © CALTOPO

GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS

GPI PROJECT NO. 3063.I

SCALE: 1" = 2000 FEET

SITE LOCATION

FIGURE 1

EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

APPROXIMATE LOCATION OF SUBSURFACE CROSS SECTION (FIGURE 4)

0 150 300 FEET

N

BASE MAP REPRODUCED FROM SITE PLAN
BY ELKUS MANFREDI ARCHITECTS, UNDATED

GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS - RESIDENTIAL BUILDING

GPI PROJECT NO.: 3063.I

SCALE: 1” = 150’

SITE PLAN
(Proposed Conditions)

FIGURE 2

EXPLANATION

B-7 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2017)

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY OTHERS (KLEINFELDER, 2009)

A A' APPROXIMATE LOCATION OF SUBSURFACE CROSS SECTION (FIGURE 4)

0 150 300 FEET

N

BASE MAP REPRODUCED FROM GOOGLE EARTH @ 2021

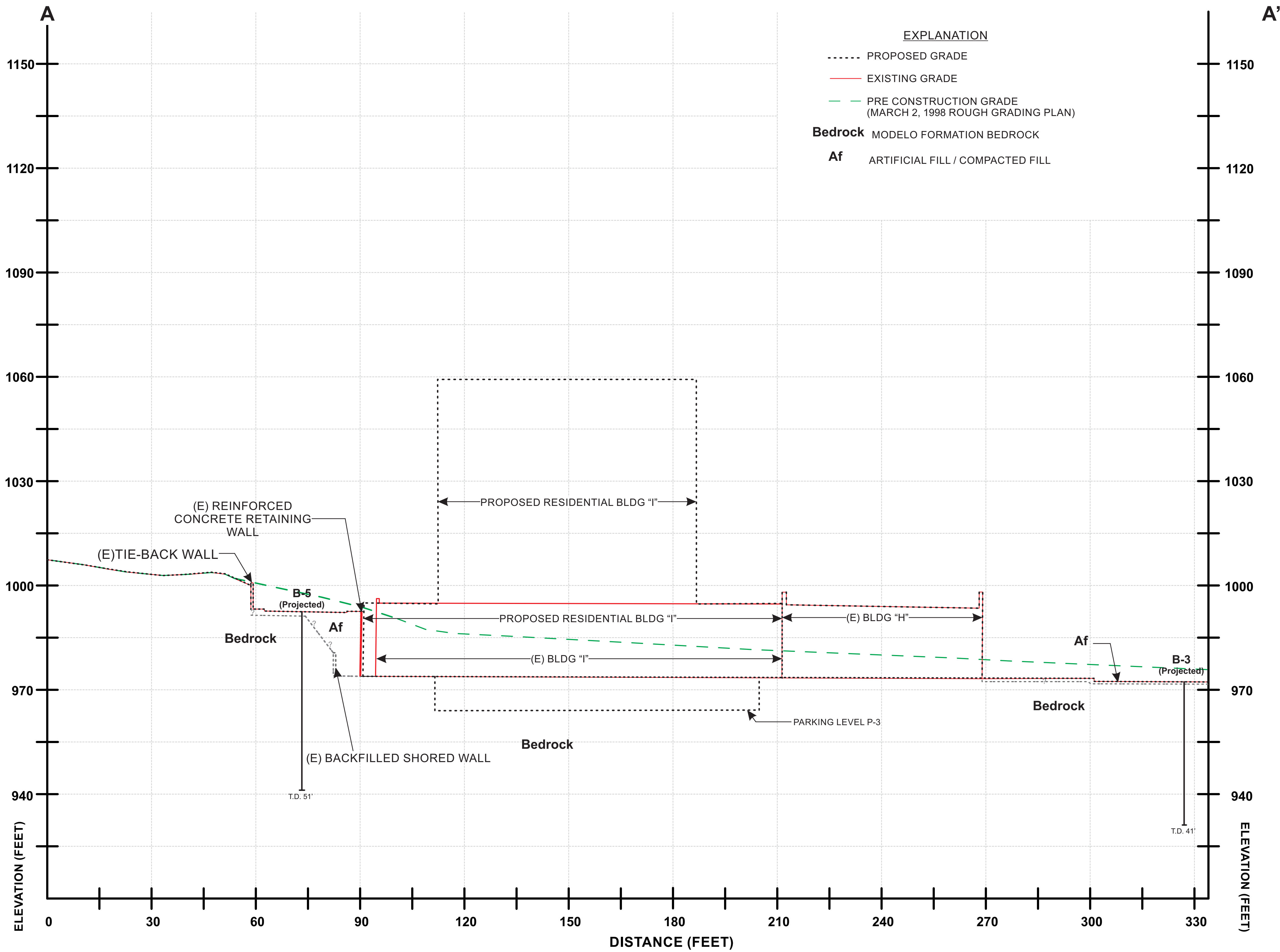
GPI GEOTECHNICAL PROFESSIONALS, INC.

THE COMMONS AT CALABASAS - RESIDENTIAL BUILDING

GPI PROJECT NO.: 3063.I	SCALE: 1" = 150'

SITE PLAN
(Existing Conditions)

FIGURE 3



Note: This section is based upon information obtained at borings obtained during geotechnical investigation. The section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

GPI GEOTECHNICAL PROFESSIONALS, INC.

COMMONS CALABASAS

GPI PROJECT NO.: 3063.I | SCALE: 1" = 30'

SUBSURFACE CROSS SECTION

FIGURE 4

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling seven exploratory borings. Six of the borings (B-1 through B-6) were drilled in August of 2021, and one of the borings (B-7) was drilled in October of 2022. The borings were advanced to depths ranging from approximately 21 to 51 feet below the existing ground surface. The locations of the explorations are shown on Figures 2 and 3, Site Plan (Proposed Conditions and Existing Conditions).

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-7 in this appendix.

The borings were backfilled with drill cuttings and patched with cold patch asphalt. Drilling permits (permit nos. PW210025 and PW2200307) were obtained from the City of Calabasas.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from ALTA/NSPS Land Title As-Built Survey of 4710-4799 Commons Way plans by Hennon Surveying and Mapping, Inc. dated September 18, 2020.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	4-Inch AC over 6-Inch BASE	
						Fill: **SANDY CLAY (CL)** light brown, slightly moist	
	29.2	80	80/11"	D		Natural: **SILTSTONE** light reddish brown, moist to very moist, hard, with fine grained sand, friable	970
	17.2	118	50/5"	D	5		
	21.5	93	82/11"	D			965
	27.5	87	72/11"	D	10	@ 10 feet, dark brown	
							960
	25.1	91	74/10"	D	15		
							955
	25.8	91	50/6"	D	20		
						Total Depth 21 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	3-Inch AC over 5-Inch BASE	
					Fill: **SANDY CLAY (CL)** brown, slightly moist	
35.2	83	89	D		Natural: **SILTSTONE** light reddish brown, moist to very, hard, friable	
33.7	82	67	D	5		965
28.5	79	62/9"	D		@ 7 feet, with fine sand	
		50/4"	D	10	@ 10 feet, no recovery	960
26.3	89	50/5"	D	15	@ 15 feet, dark brown	955
25.0	94	59/11"	D	20		950
					Total Depth 21 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-27-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-2

FIGURE A-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	5-Inch AC over 6-Inch BASE	
						Fill: **SANDY CLAY (CL)** dark brown, moist	970
	29.4	85	50	D		Natural: **SILTSTONE** dark brown, moist to very moist, hard, friable	
	33.1	83	68	D	5		
	34.9	79	83	D			965
	26.6	81	50/6"	D	10	@ 10 feet, with sand	
							960
	32.2	75	70	D	15	@ 15 feet, dark brown	
							955
	21.1	79	77	D	20		
							950
					25		
							945
	22.0	96	50/5"	D	30		
							940
					35		
							935

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	25.3	69	50/5"	D	40		**SILTSTONE** dark brown, moist to very moist, hard, friable	
							Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-3

DESCRIPTION OF SUBSURFACE MATERIALS

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	3-Inch AC over 4-Inch BASE	980
19.5	98	18	D		Fill: **SILTY CLAY (CL)** mottled light brown and dark brown, moist, stiff	
23.9	97	17	D	5		975
31.6	88	71	D		Natural: **SILTSTONE** light red brown, moist to very moist, hard, friable	
31.7	88	78/9"	D	10		970
22.1	96	50/5"	D	15	@ 15 feet, dark brown	965
21.4	100	75/10"	D	20		960
				25		955
21.9	90	50/5"	D	30		950
				35		945

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-27-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
20.5	80	50/5	D	40	**SILTSTONE** dark brown, moist to very moist, hard, friable	940
					Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 8-27-21

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-4

FIGURE A-4

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	4-Inch AC over 6-Inch BASE	
						Fill: **SILTY CLAY (CL)** brown, slightly moist, hard, trace sand	
	24.0	95	98/9"	D			
						Natural: **SILTSTONE** red brown, moist to very moist, hard, friable, trace sand	
	23.8	95	50/5"	D	5	@ 5 feet, dark brown	985
	23.3	88	505"	D			
	22.0	103	50/5"	D	10		980
	19.2	93	50/5"	D	15		975
	21.9	85	62/9"	D	20		970
					25		965
	21.1	79	50/5"	D	30		960
					35		955
							950

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-5

FIGURE A-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
21.0	84	50/6"	D	40	**SILTSTONE** dark brown, moist to very moist, hard, friable, trace sand	
				45		945
21.4	82	88/11"	D	50		940
					Total Depth 51 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 8-26-21
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				B	0	6-Inch AC over 8-Inch BASE	
						Fill: **SILTY CLAY (CL)** light brown, slightly moist	
	23.7	94	90/9"	D		Natural: **SILTSTONE** black, wet, hard, friable	
	22.7	95	87/11"	D	5		980
	21.7	98	50/6"	D			
	22.4	97	50/6"	D	10		975
	23.0	86	80/10"	D	15		970
	23.2	91	50/6"	D	20		965
					25		960
						Refusal @ 29 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-26-21

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I

COMMONS AT CALABASAS

LOG OF BORING NO. B-6

FIGURE A-6

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	4-Inch AC OVER 6-Inch BASE	
					Fill: **SANDY CLAY** brown, wet, very dense, with silt	970
26.2	85	50/6"	D		Natural: **SILTSTONE** red brown, wet, hard, with sand	
35.4	79					
25.7	77	50/4"	D			
				5		
20.7		50/3"	D		@ 6 feet, very moist	965
13.3		50/4"	D		@ 8 feet, moist	
22.1	85	50/4"	D	10	@ 10 feet, very moist, dark brown	
						960
21.5	84	50/4"	D			
26.6	91	50/4"	D	15	@ 15 feet, wet	
						955
25.8	84	50/5"	D	20		
						950
26.2	90	50/5"	D	25		
						945
26.1	75	50/5"	D	30		
						940
23.5	83	50/5"	D	35	@ 35 feet, very moist	
						935

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
10-6-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-7

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	20.6	78	50/4"	D	40			
							Total Depth 41 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
10-6-22
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3063.I
COMMONS AT CALABASAS

LOG OF BORING NO. B-7

FIGURE A-1

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the tables and figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

ATTERBERG LIMITS

The liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 to B-5.

COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D1557 on representative bulk samples of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-3	0 – 5	Mixture of Sandy Clay (CL) and Siltstone	95	23
B-4	0 - 5	Silty Clay (CL)	111	16

EXPANSION INDEX TEST

Expansion index tests were performed on representative bulk samples of the site soils. The tests were performed in accordance with ASTM D4829 to assess the expansion potential of the on-site soils. The results of the tests are summarized below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX, EI	EXPANSION POTENTIAL
B-3	0-5	Mixture of Sandy Clay (CL) and Siltstone	70	Medium
B-4	0-5	Silty Clay (CL)	37	Low
B-7	0-5	Mixture of Sandy Clay (CL) and Siltstone	58	Medium

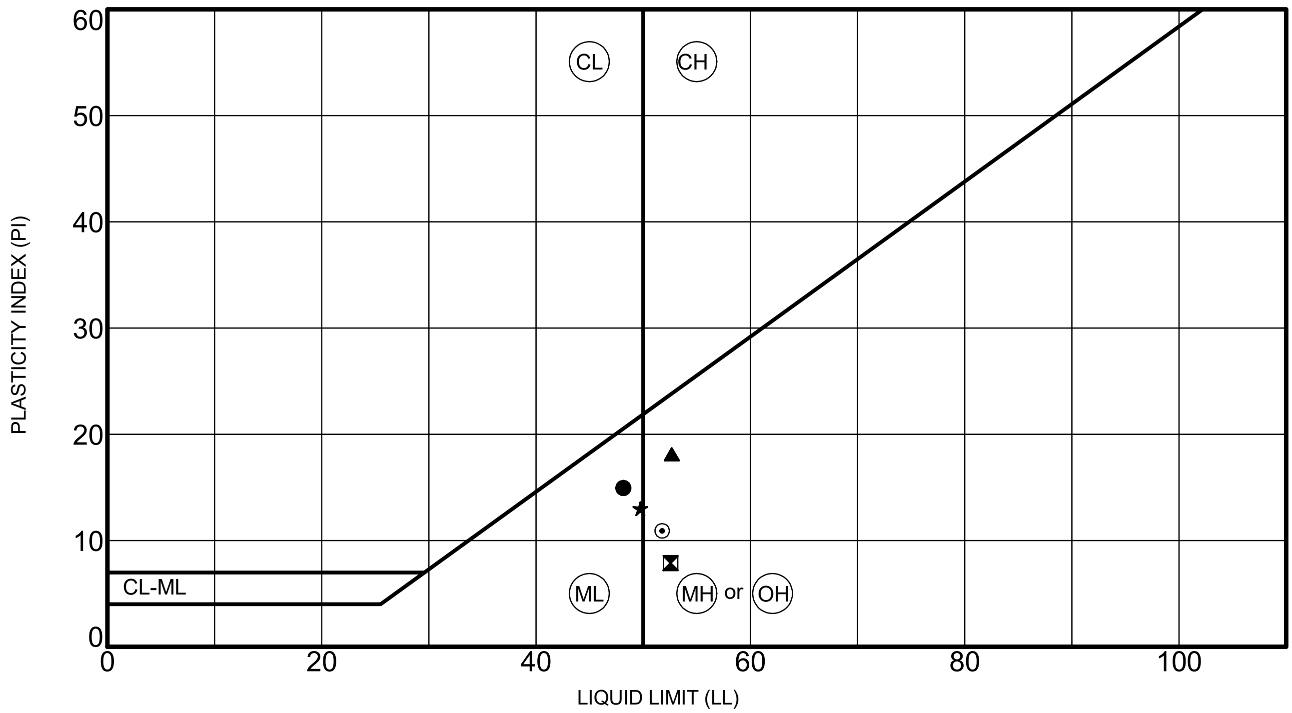
R-VALUE

Suitability of the near-surface soils for pavement was evaluated by conducting an R-value test. The test was performed in accordance with ASTM D2844 by GeoLogic Associates (GLA) under subcontract to GPI. The result of the test is as follows.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-3	0 – 5	Mixture of Sandy Clay (CL) and Siltstone	13

CORROSIVITY

Soil corrosivity testing was performed by HDR soil and bedrock samples provided by GPI. The test results are summarized in the tables by HDR included at the end of this Appendix.



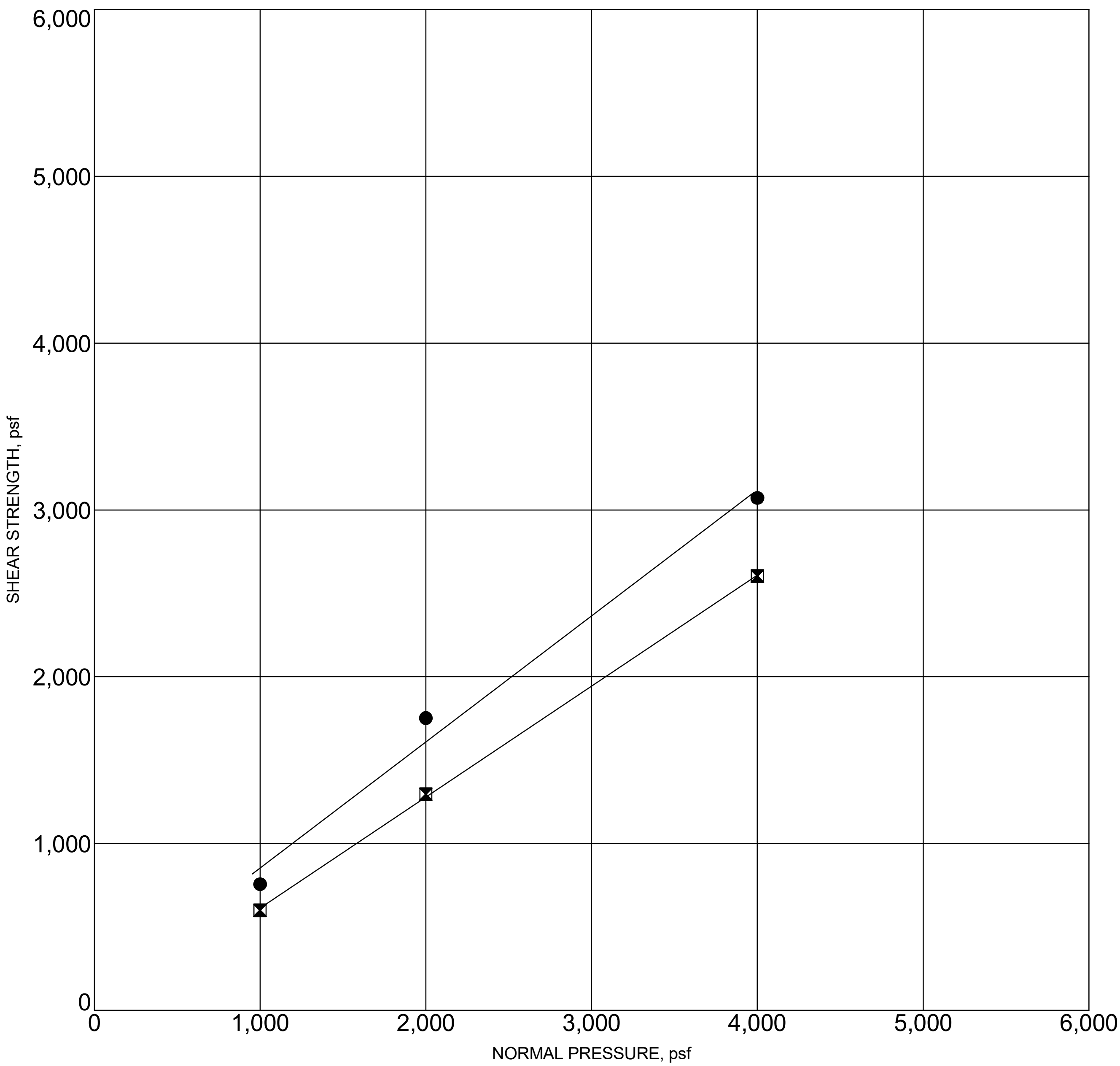
	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	0.0	48	33	15		SILTSTONE
⧗	B-2	5.0	53	45	8		SILTSTONE
▲	B-5	10.0	53	35	18		SILTSTONE
★	B-5	30.0	50	37	13		SILTSTONE
⊙	B-7	12.5	52	41	11		SILTSTONE

PROJECT: COMMONS AT CALABASAS PROJECT NO. 3063.I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 37 degrees
Cohesion= 96 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 33 degrees
Cohesion= 0 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	5.0	SILTSTONE	82	33.7

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

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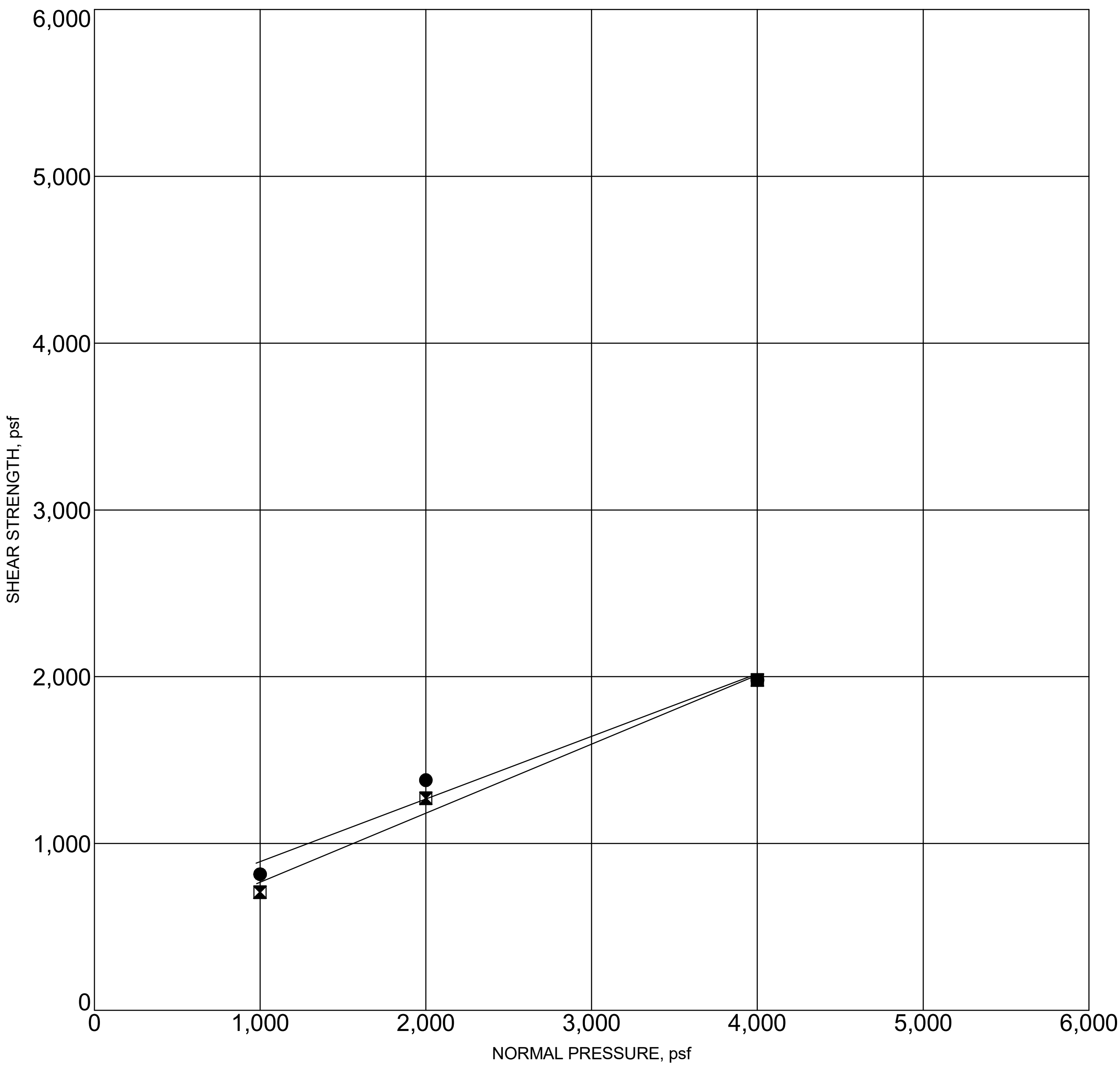
DIRECT SHEAR TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 21 degrees
Cohesion= 516 psf

ULTIMATE STRENGTH
Friction Angle= 22 degrees
Cohesion= 354 psf

Note: Samples remolded to 90% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-3	0-5	MIXTURE OF SANDY CLAY (CL) AND SILTSTONE	86	23.0

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

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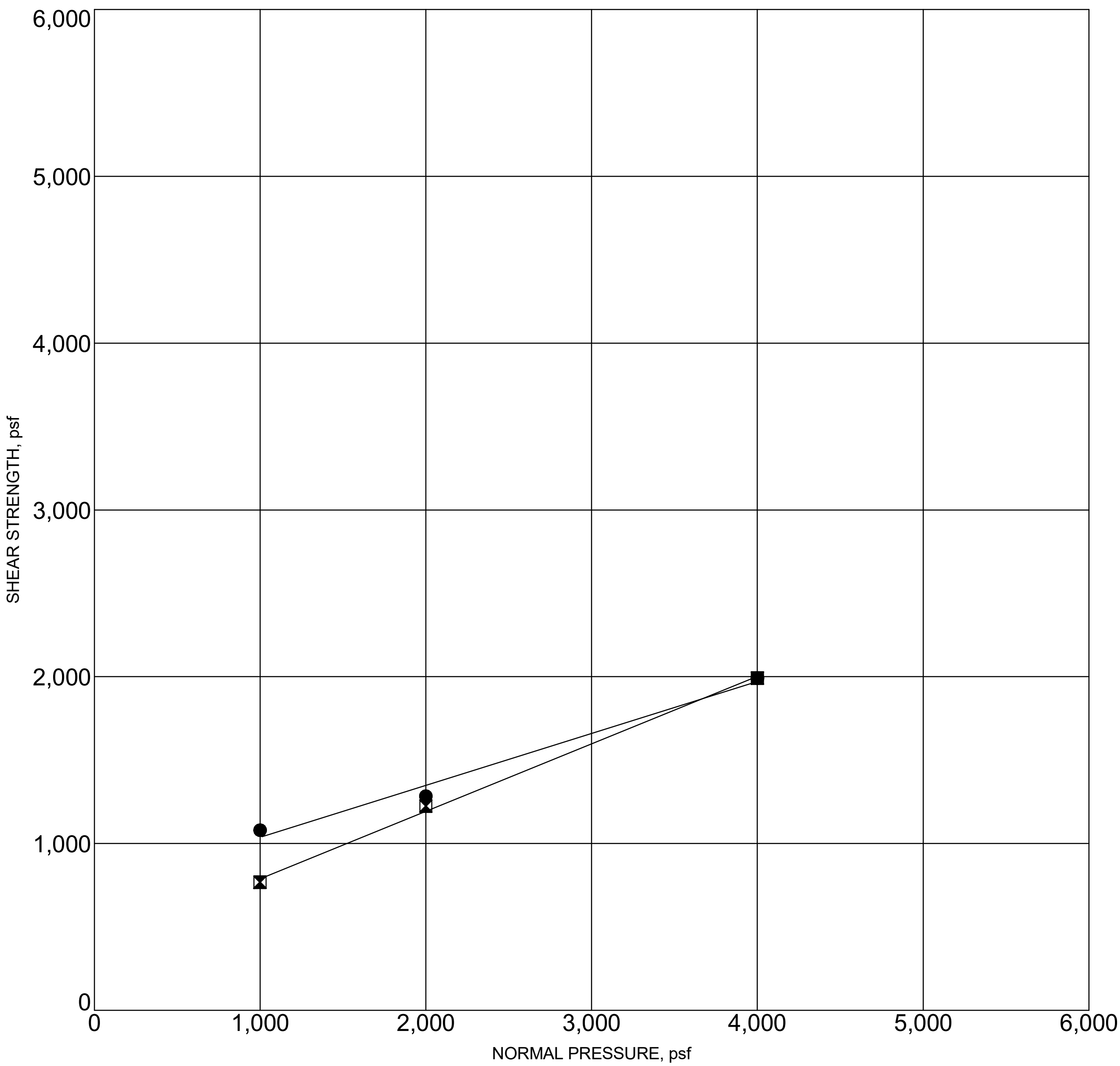
DIRECT SHEAR TEST RESULTS

FIGURE B-3



● **PEAK STRENGTH**
Friction Angle= 17 degrees
Cohesion= 726 psf

ULTIMATE STRENGTH
Friction Angle= 22 degrees
Cohesion= 384 psf

Note: Samples remolded to 90% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-4	0-5	SILTY CLAY (CL)	100	16.0

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

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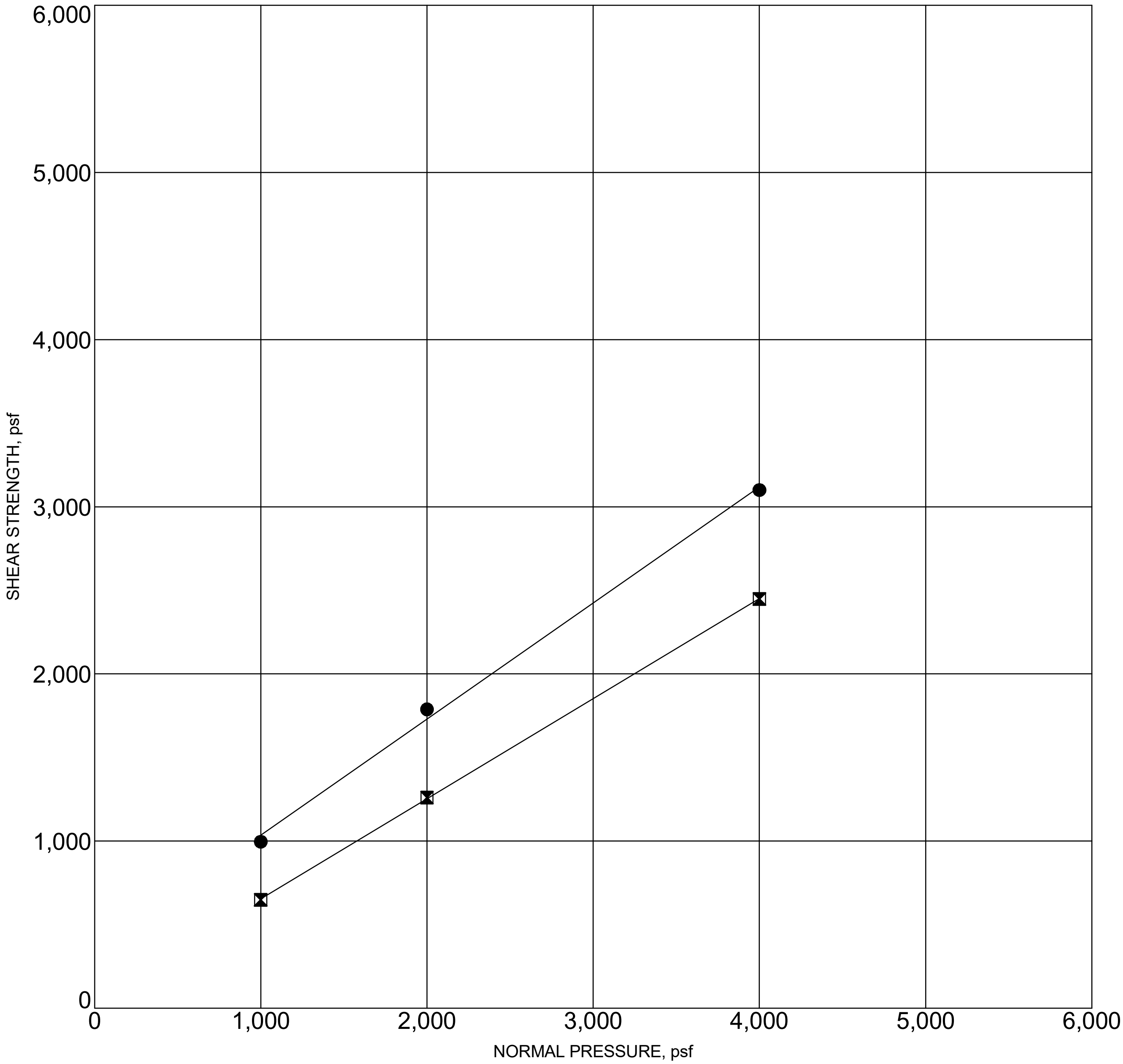
DIRECT SHEAR TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 35 degrees
Cohesion= 340 psf

ULTIMATE STRENGTH
Friction Angle= 31 degrees
Cohesion= 54 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	10.0	SILTSTONE	97	22.4

PROJECT: COMMONS AT CALABASAS PROJECT NO.: 3063.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-5

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Commons Calabasas
Your #3063.I, HDR Lab #21-0820LAB
17-Sep-21

Sample ID

			B-3 @ 0-5'	B-4 @ 0-5'	B-1 @ 0-5'	B-4 @ 30'	B-6 @ 20'
Resistivity		**Units**					
as-received		ohm-cm	9,200	12,800	13,200	60,000	56,000
saturated		ohm-cm	680	1,120	960	440	600
pH			6.2	8.8	6.3	5.0	4.6
Electrical Conductivity		mS/cm	2.14	0.44	0.63	1.20	1.66
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	3,190	185	247	537	1,550
magnesium	Mg^{2+}	mg/kg	153	0.8	21	246	601
sodium	Na^{1+}	mg/kg	139	127	183	233	92
potassium	K^{1+}	mg/kg	154	23	100	186	174
ammonium	NH_4^{1+}	mg/kg	84	ND	ND	84	75
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	77	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	275	46	214	186	79
fluoride	F^{1-}	mg/kg	42	14	20	16	15
chloride	Cl^{1-}	mg/kg	21	19	35	14	11
sulfate	SO_4^{2-}	mg/kg	9,860	866	1,520	3,460	5,990
nitrate	NO_3^{1-}	mg/kg	61	6.5	20	3.6	14
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND	0.6	ND
Other Tests							
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
COMMONS CALABASAS
Your #3063.I, HDR Lab #22-0988LAB
24-Oct-22

Sample ID			
			B-7 @ 6
Resistivity		**Units**	
as-received		ohm-cm	48,000
saturated		ohm-cm	1,720
pH			5.3
Electrical Conductivity		mS/cm	0.39
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	119
magnesium	Mg^{2+}	mg/kg	16
sodium	Na^{1+}	mg/kg	160
potassium	K^{1+}	mg/kg	89
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	107
fluoride	F^{1-}	mg/kg	3.1
chloride	Cl^{1-}	mg/kg	31
sulfate	SO_4^{2-}	mg/kg	497
nitrate	NO_3^{1-}	mg/kg	3.7
phosphate	PO_4^{3-}	mg/kg	20
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316