OAKMONT SENIOR LIVING-AGOURA HILLS

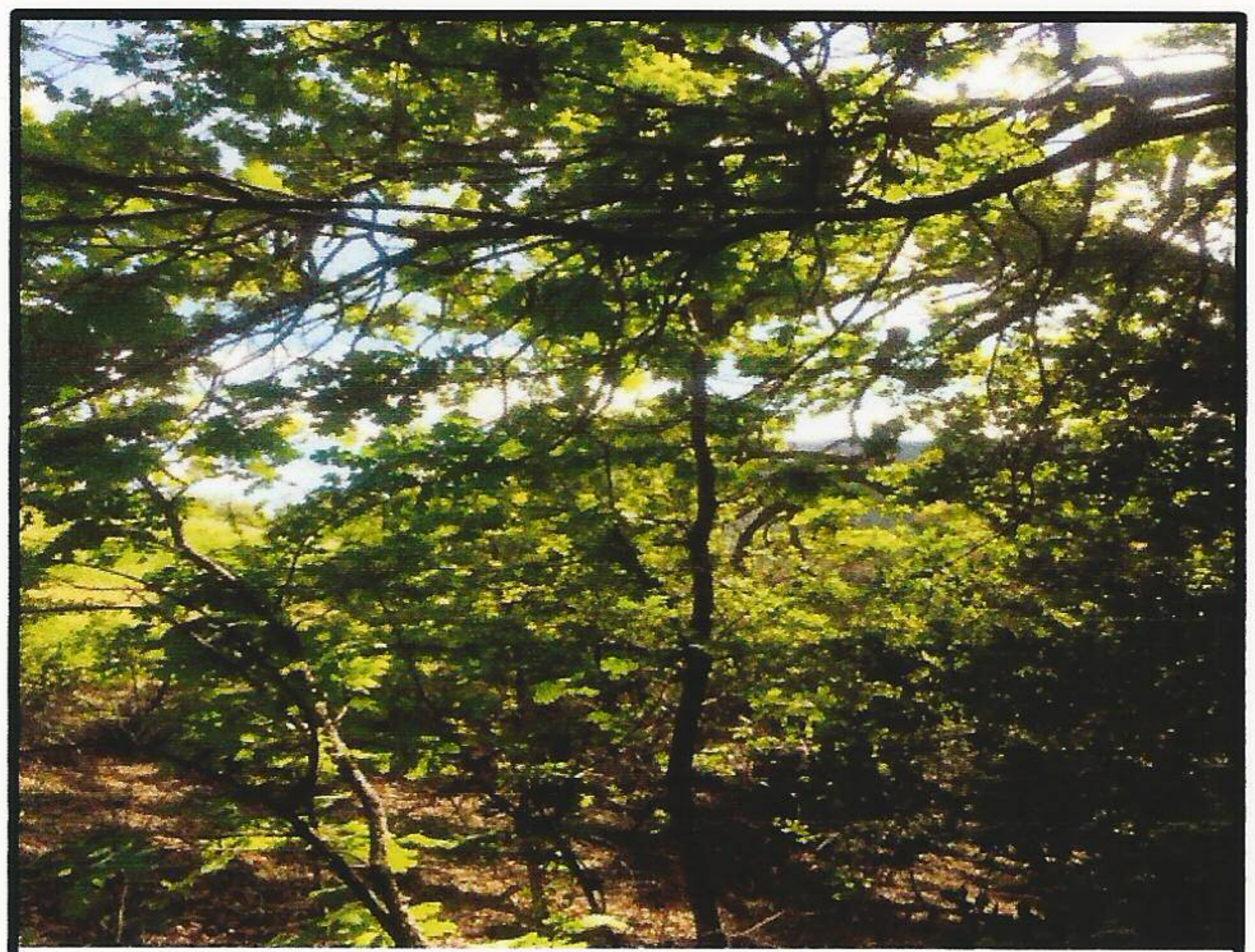
PHOTO LOG

OSL-35

OSL-36

OSL-37

OSL-37a



OSL-37b

OSL-38

OAKMONT SENIOR LIVING-AGOURA HILLS

PHOTO LOG

OSL-38a

OSL-39

OSL-40

OSL-41

OSL-42

OSL-43

OAKMONT SENIOR LIVING-AGOURA HILLS

PHOTO LOG

OSL-44

OSL-45

OSL-46

OSL-47

OSL-47a

OSL-48

OAKMONT SENIOR LIVING-AGOURA HILLS

PHOTO LOG


OSL-49


OSL-50


OSL-50a


OSL-51


OSL-52


OSL-53

OAKMONT SENIOR LIVING-AGOURA HILLS

PHOTO LOG

OSL-54

OSL-55

OSL-56

OSL-56a

OSL-57

OSL-58

OAKMONT SENIOR LIVING-AGOURA HILLS

PHOTO LOG

OSL-59

OSL-60

Greg Ainsworth

Memo

To: Allison Cook, Assistant Planning Director

From: Greg Ainsworth, Oak Tree Consultant

Date: May 26, 2016

Re: Oakmont Senior Living Oak Tree Assessment

I reviewed the Oak Tree Report and Site Plan packet (e.g., grading and landscape plans) for the proposed Oakmont Senior Living project. In addition, I visited the site on May 18, 2016 to verify the oak trees located on the property, which includes verifying if any oak trees could potentially be affected by the demolition and removal of a cement building foundation.

The Oak Tree report indicates that no oak trees would be removed, which appears to be true based on my review of the report, site plans, and field verification. As indicated in the Oak Tree Report, Oak Trees OSL-10, OSL-54, and OSL-55 would be encroached by the proposed retaining wall. The site plans (e.g., landscape plan) does not include tree numbers or the protective zones of the trees; therefore, I am unable to verify which trees would be encroached, and to what extent. The Oak Tree Report does not include a tree map, nor does it include ANY data on the trees, such as species, trunk diameter, height, canopy spread, protective zone, physical condition, health, etc.

The applicant must include the tree numbers and protective zones on a site plan map, so that the amount of encroachment to each tree can be estimated. Moreover, data on each tree, especially those that would be encroached, must be provided with the Oak Tree Report. The Oak Tree Report must be prepared in accordance with the guidelines set forth in the Oak Tree Preservation Guidelines.

Greg Ainsworth

Memo

To: Allison Cook, Assistant Planning Director

From: Greg Ainsworth, Oak Tree Consultant

Date: August 3, 2016

Re: Oakmont Senior Living Oak Tree Assessment

An initial review of the Oak Tree Report and Site Plan packet (e.g., grading and landscape plans) for the proposed Oakmont Senior Living project was conducted in May 2016, and a memorandum dated May 26, 2016 was prepared by Greg Ainsworth that outlined missing information needed to provide a thorough assessment of proposed impacts to protected oak trees. The missing information indicated in the May memorandum included the following:

- All tree numbers and protective zones must be depicted on a site plan map, so that the amount of encroachment to each tree can be estimated.
- Data on each tree, especially those that would be encroached, must be provided with the Oak Tree Report. The Oak Tree Report must be prepared in accordance with the guidelines set forth in the Oak Tree Preservation Guidelines.
- The Oak Tree Report must be prepared in accordance with the guidelines set forth in the Oak Tree Preservation Guidelines.

A revised Oak Tree Report for the Oakmont Senior Living project was submitted, which includes an Oak Tree Map that accurately depicts the tree numbers and protective zones of all City-protected trees located on the property. In addition, the revised tree report includes a summary of the oak tree data that was collected for each City-protected oak tree located on the property. In summary, the recently submitted Oak Tree Report has been prepared in accordance with the City's Oak Tree Preservation Guidelines.

The Oak Tree report indicates that no oak trees would be removed, which appears to be accurate based on my review of the report, site plans, and field verification. As indicated in the Oak Tree Report, Oak Trees OSL-10, OSL-54, and OSL-55 would be encroached by the proposed retaining wall. The following condition will be required to protect and preserve all oak trees located on the property.

Oak Tree Protection and Preservation

1. All oak trees located on the property that would be encroached or otherwise avoided shall be preserved in perpetuity.

2. An Oak Tree Permit Application and associated fees shall be submitted to the city, and approved, prior to the initiation of any ground disturbance activities.

3. All subsurface ground disturbance that will occur within the protective zone of an oak tree shall be performed using only hand tools under the direct observation of the applicant's oak tree consultant. If vegetation clearing or grading is not feasible within the protective zone with the use of hand tools, mechanical equipment may be allowed, so long as a certified arborist is present to ensure that no impacts occur to the oak tree.

4. Prior to the start of any work or mobilization at the site, protective fencing shall be installed at the protective zone of preserved oak trees that are located within a minimum of 100 feet of areas where ground disturbance will occur. The applicant or their consulting arborist shall consult the City's Oak Tree Consultant to determine the exact fencing configuration and appropriate fencing material, and submit a fencing plan subject to approval by the City's Oak Tree Consultant.

5. The applicant shall provide a minimum of 48 hours notice to the City Oak Tree Consultant prior to the start of any work within the protected zone of any oak tree.

6. No grading, scarifying or other soil disturbance shall be permitted within the portion of a protected zone of any oak tree except as specifically required to complete the approved scope of work.

7. No vehicles, equipment, materials, spoil or other items shall be used or placed within the protected zone of any oak tree at any time, except as specifically required to complete the approved work.

8. No irrigation or ground cover shall be installed within the Protective Zone of any existing oak tree unless specifically approved by the City Oak Tree Consultant and the Planning Director.

9. Prior to removal of the protective fencing, the applicant shall contact the City Oak Tree Consultant to perform a final inspection. The applicant shall proceed with any remedial measures the City Oak Tree Consultant deems necessary to protect or preserve the health of the subject oak tree at that time.

10. No pruning of live wood of an oak tree (including branches and roots) shall be permitted unless specifically authorized by the City Oak Tree Consultant and/or following an approved oak tree permit. Any authorized pruning shall be performed by a qualified arborist under the direct observation of the applicant's oak tree consultant. All pruning operations shall be consistent with ANSI A300 Standards – Part 1 Pruning and the most recent edition of the International Society of Arboriculture Best Management Practices for Tree Pruning.

11. No herbicides shall be used within 100 feet of the dripline of any oak tree unless the program is first reviewed and endorsed by the City Oak Tree Consultant.

Native American Heritage Commission Request and Response Letters

APPENDIX F

August 16, 2017

Native American Heritage Commission
1550 Harbor Boulevard, Room 100
West Sacramento, CA 95691

Subj: **Phase I Cultural Resources Assessment of the Oakmont of Agoura Hills Project *(Envicom Project #56-635-101*)**

Greetings,

Envicom is requesting a record search of the NAHC database for cultural resources for the attached Project area, plus a **0.25-mile buffer**. We also request a list of Tribal Group representatives for the area in case we need to contact their offices.

The Project is located at:

USGS Quad – Thousand Oaks, CA

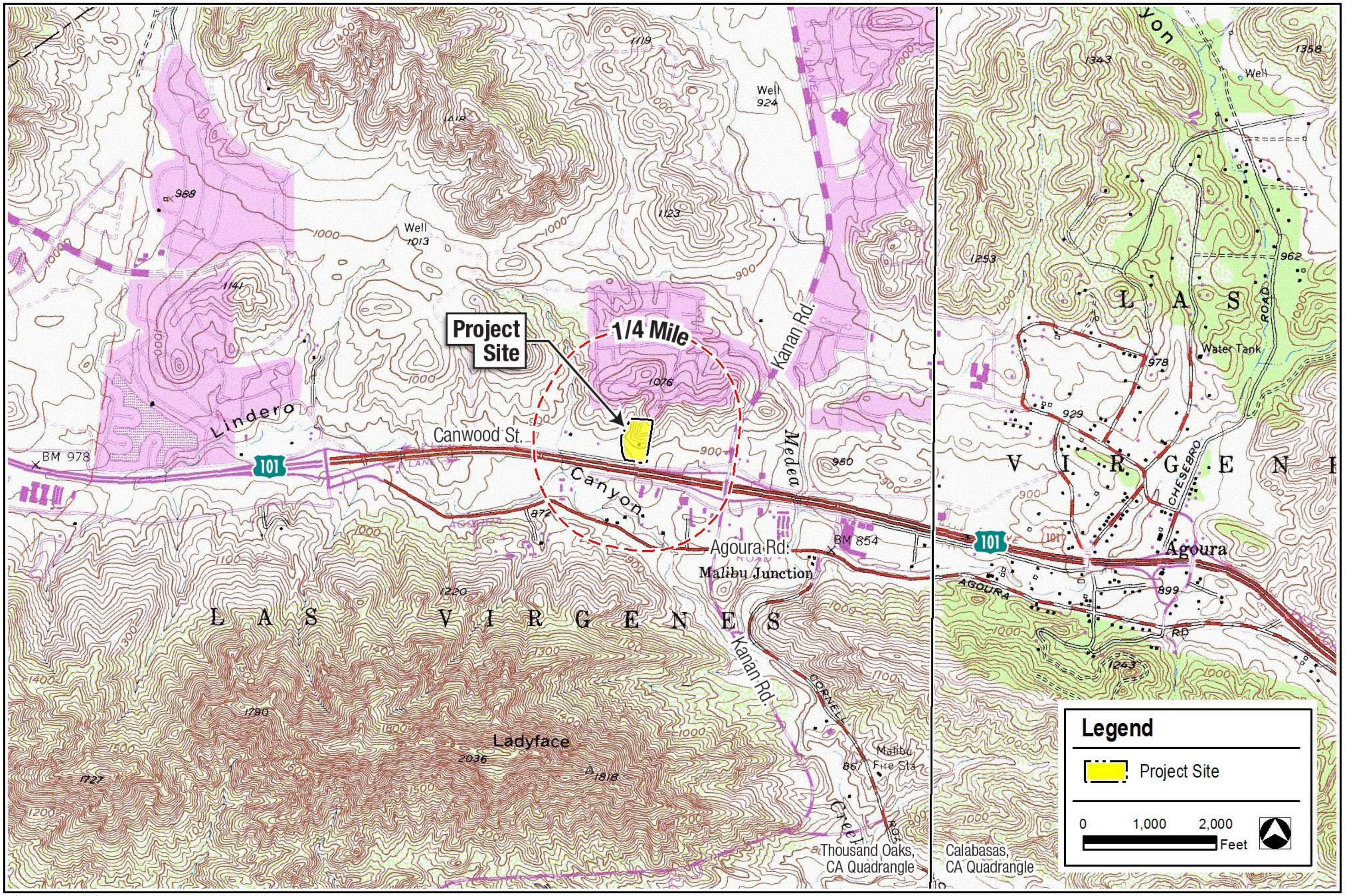
Please indicate if there are Native American cultural resources within the project area, or only in the project study area.

Envicom appreciates the NAHC's help with this request. For correspondence or questions regarding this Project, please contact Wayne Bischoff at 818-879-4700 (wbischoff@envicomcorporation.com).

Sincerely,

Dr. Wayne Bischoff
Director of Cultural Resources

Attachment:
Project vicinity map on 1:24,000 topographic map



Project Site
1/4 Mile
Canwood St.
Lindero
Kanan Rd.
Medea
Canyon
Agoura Rd.
Malibu Junction
BM 978
BM 854
Well 1013
Well 924
Well
Water Tank
LAS VIRGENES
Ladyface
Agoura
Chesebro Road
Agoura Rd
Malibu Fire Sta
Thousand Oaks, CA Quadrangle
Calabasas, CA Quadrangle
Legend
Project Site
0
1,000
2,000
Feet

STATE OF CALIFORNIA Edmund G. Brown, Jr., *Governor*

NATIVE AMERICAN HERITAGE COMMISSION
Environmental and Cultural Department
1550 Harbor Blvd., Suite 100
West Sacramento, CA 95691
(916) 373-3710

August 22, 2017

Dr. Wayne Bischoff
Envicom Corporation

Sent by E-mail: wbischoff@envicomcorporation.com
Cc: waynebischoff@gmail.com

RE: Proposed Oakmont of Agoura Hills Project, City of Agoura Hills; Thousand Oaks USGS Quadrangle, Los Angeles County, California

Dear Dr. Bischoff:

A record search of the Native American Heritage Commission (NAHC) *Sacred Lands File* was completed for the area of potential project effect (APE) referenced above with negative results. Please note that the absence of specific site information in the *Sacred Lands File* does not indicate the absence of Native American cultural resources in any APE.

Attached is a list of tribes culturally affiliated to the project area. I suggest you contact all of the listed Tribes. If they cannot supply information, they might recommend others with specific knowledge. The list should provide a starting place to locate areas of potential adverse impact within the APE. By contacting all those on the list, your organization will be better able to respond to claims of failure to consult. If a response has not been received within two weeks of notification, the NAHC requests that you follow-up with a telephone call to ensure that the project information has been received.

If you receive notification of change of addresses and phone numbers from any of these individuals or groups, please notify me. With your assistance we are able to assure that our lists contain current information. If you have any questions or need additional information, please contact via email: gayle.totton@nahc.ca.gov.

Sincerely,

Gayle Totton

Gayle Totton, M.A., PhD.
Associate Governmental Program Analyst

Native American Heritage Commission
Native American Contact List
Los Angeles County
8/23/2017

Barbareno/Ventureno Band of Mission Indians
Patrick Tumamait,
992 El Camino Corto — Chumash
Ojai, CA, 93023
Phone: (805) 216 - 1253

Barbareno/Ventureno Band of Mission Indians
Eleanor Arrellanes,
P. O. Box 5687 — Chumash
Ventura, CA, 93005
Phone: (805) 701 - 3246

Barbareno/Ventureno Band of Mission Indians
Julie Tumamait-Stennslie, Chairperson
365 North Poli Ave — Chumash
Ojai, CA, 93023
Phone: (805) 646 - 6214
jtumamait@hotmail.com

Barbareno/Ventureno Band of Mission Indians
Raudel Banuelos,
331 Mira Flores — Chumash
Camarillo, CA, 93012
Phone: (805) 427 - 0015

Coastal Band of the Chumash Nation
Mia Lopez, Chairperson
Phone: (805) 324 - 0135 — Chumash
cbcntribalchair@gmail.com

Fernandeno Tataviam Band of Mission Indians
Beverly Salazar, Councilmember
1931 Shady Brooks Drive — Tataviam
Thousand Oaks, CA, 91362
Phone: (805) 558 - 1154

Fernandeno Tataviam Band of Mission Indians
Kimia Fatehi, Tribal Historic and Cultural Preservation Officer
1019 Second Street, Suite 1 — Tataviam
San Fernando, CA, 91340
Phone: (818) 837 - 0794
Fax: (818) 837-0796
kfatehi@tataviam-nsn.us

Fernandeno Tataviam Band of Mission Indians
Alan Salazar, Chairman Elders Council
1019 Second St., Suite 1 — Tataviam
San Fernando, CA, 91340
Phone: (805) 423 - 0091

Fernandeno Tataviam Band of Mission Indians
Beverly Folkes, Elders Council
1019 Second St. Suite 1 — Tataviam
San Fernando, CA, 91340

San Fernando Band of Mission Indians
John Valenzuela, Chairperson
P.O. Box 221838 — Kitanemuk
Newhall, CA, 91322 — Serrano
Phone: (760) 885 - 0955 — Tataviam
tsen2u@hotmail.com

Santa Ynez Band of Mission Indians
Kenneth Kahn, Chairperson
P.O. Box 517 — Chumash
Santa Ynez, CA, 93460
Phone: (805) 688 - 7997
Fax: (805) 686-9578
kkahn@santaynezchumash.org

This list is current only as of the date of this document. Distribution of this list does not relieve any person of statutory responsibility as defined in Section 7050.5 of the Health and Safety Code, Section 5097.94 of the Public Resource Section 5097.98 of the Public Resources Code.

This list is only applicable for contacting local Native Americans with regard to cultural resources assessment for the proposed Oakmont of Agoura Hills Project, Los Angeles County.

Geotechnical Investigation and Addenda and City Geotechnical Consultant Memo

APPENDIX G

CHJ Consultants
1355 E. Cooley Drive, Suite C, Colton, CA 92324 ◆ Phone (909) 824-7311 ◆ Fax (909) 503-1136
15345 Anacapa Road, Suite D, Victorville, CA 92392 ◆ Phone (760) 243-0506 ◆ Fax (760) 243-1225
77-564A Country Club Drive, Suite 122, Palm Desert, CA 92211 ◆ Phone (760) 772-8234 ◆ Fax (909) 503-1136

October 21, 2015

Oakmont Senior Living Job No. 15473-3
9249 Old Redwood Highway, Suite 200
Windsor, California 95492
Attention: Mr. Wayne Sant, Vice President, Development

Dear Mr. Sant:

Attached herewith is the Geotechnical Investigation report prepared for the proposed Oakmont of Agoura Hills senior facility, to be located at 29353 Canwood Street, in the city of Agoura Hills, California.

This report was based upon a scope of services generally outlined in our proposal dated September 17, 2015, and other written and verbal communication.

We appreciate this opportunity to provide engineering geologic services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
CHJ CONSULTANTS

Maihan Noorzay
Project Engineer, P.E.

MN:lb

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OAKMONT OF AGOURA HILLS
29353 CANWOOD STREET
AGOURA HILLS, CALIFORNIA
PREPARED FOR
OAKMONT SENIOR LIVING
JOB NO. 15473-3

GEOTECHNICAL INVESTIGATION
OAKMONT OF AGOURA HILLS
29353 CANWOOD STREET
AGOURA HILLS, CALIFORNIA
PREPARED FOR
OAKMONT SENIOR LIVING
JOB NO. 15473-3

INTRODUCTION

During October of 2015, this firm performed a geotechnical investigation for the proposed Oakmont of Agoura Hills senior facility, which is to be located at 29353 Canwood Street (APN 2053-001-005), in the city of Agoura Hills, California. The purposes of this investigation were to explore and evaluate the geotechnical engineering/engineering geologic conditions of the site and to provide appropriate geotechnical engineering recommendations for the design and construction of the subject project.

The approximate location of the site is shown on the attached Index Map (Enclosure "A-1"). To orient our investigation, a site plan prepared by Landesign Group, Inc., showing the building location was provided for our use. The plan was utilized as a base map for our Site Plan (Enclosure "A-2").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this investigation included the following:

- Review of published and unpublished geologic literature and maps
- Field reconnaissance of the subject site and surrounding area and geologic mapping of the site
- Marking of exploration locations in the field and notification of Underground Service Alert
- Placement of four exploratory borings within the building pad area

- Placement of seven exploratory trenches within the site area
- Double-ring infiltrometer testing at two locations on the site
- Logging and sampling of the exploratory borings and test pits for testing and evaluation
- Laboratory testing on selected samples
- Evaluation of geologic hazards
- Seismic design parameters according to the 2013 California Building Code (CBC)
- Evaluation of the geotechnical data to develop site-specific recommendations for suitable foundation recommendations, including allowable bearing pressures, ultimate and allowable passive earth resistance and base friction, lateral earth pressures and mitigation of potential geotechnical concerns and hazards, such as expansive soils, liquefaction and seismic settlement, if encountered
- Preparation of this report summarizing our findings, professional opinions and recommendations for the geotechnical aspects of project design and construction

PROJECT CONSIDERATIONS

The proposed two- and three-story senior facility will include more than 80 units and will be approximately 80,000 square feet in plan area. We anticipate that the facility will be of wood frame and stucco or masonry construction. Light to moderate foundations loads are typically associated with structures of the type proposed.

Our review of furnished plans indicates that the site elevation varies approximately 120 feet, with the highest elevation of approximately 1,000 feet at the northeast corner and the lowest of approximately 880 feet at the southwest corner. The northern portion of the building pad (2-story portion) will be at elevation 912 feet and the southern portion of the building pad (3-story portion) will be at elevation 902 feet. Based on this information, we anticipate that the building pad and foundations will be stepped. Per our conversation with the client, post-tension slab foundations are anticipated. We expect that the slope on the north side of the building pad will be cut to provide a level building pad and that stepped retaining walls will be required for slope stability purposes. The slope cut will be on the order of 20 feet.

The final project grading plan should be reviewed by the geotechnical engineer to confirm that recommendations provided in this report have been properly implemented.

SITE DESCRIPTION

The site is located along a freeway frontage road on the north side of the 101 freeway, west of the Kanan Road off-ramp. At the time of our investigation, commercial buildings were located west of the site, and undeveloped land was located to the north and east. The site slopes up at a gentle grade north from Kanan Road to the toe of an approximately 2 horizontal (h) to 1 vertical (v) slope located north of the proposed building area. Debris and evidence of an abandoned structure and foundation area were present in the northeastern portion of the site.

Historic aerial imagery dating from 1947 was examined as part of this investigation. At the time of the 1947 aerial image, the site and surrounding area were undeveloped land. By the time of the 1959 aerial image, several structures were present on the north portion of the site. These structures remained on the site until the time of the 1980 aerial image, when the site appeared in its present condition, with debris in the northeastern portion of the site. Construction began on the commercial structures west of the site by the time of the image dated December 31, 2006, and was completed between the time of the image dated January 8, 2008, and May 24, 2009.

FIELD INVESTIGATION

Four exploratory borings were drilled to a maximum depth of 50-1/2 feet below the existing ground surface (bgs) using a limited-access (track mounted) hollow-stem auger drill rig equipped for soil sampling. In addition, seven trenches were excavated to depths ranging from 4 feet to 9-1/2 feet bgs. The exploratory trenches were used to evaluate the geologic structure of the bedrock. Two exploratory test pits were excavated in the proposed parking and driveway areas and were utilized to perform double-ring infiltrometer tests. The approximate locations of our exploratory borings, trenches and test pits are indicated on the attached Site Plan (Enclosure "A-2").

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. Both a standard penetration test (SPT) sampler (2-inch outer diameter and 1-3/8 inch inner diameter) and a modified California sampler (3-1/4-inch outer diameter and 2-3/8-inch inner diameter) were utilized in our investigation. Relatively undisturbed samples were obtained by driving the modified California sampler (a split-spoon ring sampler) ahead of the borings at selected levels. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The sampler is driven with an automatic hammer that drops a 140-pound weight 30 inches for each blow. After the required seating, the sampler is advanced up to 18 inches, providing up to three sets of blowcounts at each sampling interval. The recorded blows are raw numbers without any corrections for hammer type (automatic vs. manual cathead) or sampler size (California sampler vs. standard penetration test sampler). Both relatively undisturbed and bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

Our exploratory boring logs, together with our in-place blowcounts per 6-inch increment, are presented in Appendix "B". The stratification lines presented on the boring logs represent approximate boundaries between soil types, which may include gradual transitions.

LABORATORY INVESTIGATION

Included in our laboratory testing program were field moisture content tests on all samples returned to the laboratory and field dry density tests on all relatively undisturbed ring samples. The results are included on the boring logs. An optimum moisture content - maximum dry density relationship was established for a representative soil type. A direct shear test was performed on a selected remolded sample in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. No. 200 wash, sieve analysis, sand equivalent and plasticity index testing was performed on selected samples in order to classify the subsurface soils encountered. Expansion index testing was performed on a selected sample to evaluate the expansion potential of the subsurface soils. Since the on-site soils are expansive, a sample was set up in the consolidation testing machine to determine expansive deformation strain and expansive pressure.

A selected sample of material was delivered to HDR for chemical/corrosivity testing.

Summaries of the laboratory test results appear in Appendix "C". Soil classifications provided in our geotechnical investigation are generally per the Unified Soil Classification System (USCS).

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

Regionally, the site is located in a valley within the Santa Monica Mountains of the Transverse Ranges geomorphic province. This province includes several discreet mountain ranges and intervening valleys including the Santa Monica, San Gabriel and San Bernardino Mountains and is so named because structural trends, such as the Simi-Santa Rosa fault zone, are oriented east-west in relation to the dominant northwest-southeast trend of adjoining provinces. The Transverse Ranges province extends from the Channel Islands eastward to the Eagle and Cottonwood Mountains of the Mojave Desert. As depicted on published geologic mapping, the site is underlain by the Upper Topanga formation, which is a Miocene-age sedimentary bedrock consisting of interbedded shale, siltstone and sandstone, and Miocene-age Conejo Volcanics (Dibblee, and Ehrenspeck, 1993, Enclosure "A-3").

As encountered in the explorations, the site is mantled by colluvial fill to depths from approximately 3 to 5 feet below ground surface. The fill materials encountered consisted of medium dense to dense clayey sand (SC) and stiff to hard fat clay (CH). The bedrock was encountered at depths of 3 to 10 feet bgs and consisted of Topanga Formation Siltstone recovered as silty and clayey sands (SM, SC), clays (CL, CH) and silt (ML).

Groundwater or seepage was not encountered in the explorations. Refusal was not encountered in the explorations to the maximum 50-1/2 foot depth. Caving was not encountered upon removal of the drilling augers.

More detailed descriptions of the subsurface soil conditions encountered are presented on the attached boring logs (Appendix "B").

FAULTING

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting. The closest known fault is a segment of the Chatsworth fault that is located approximately 4.5 miles to the northeast. The Malibu fault, Santa Monica fault, Sierra Madre fault zone and San Gabriel fault zone are the nearest known faults to the site and are located 7.6 miles south, 9.5 miles southeast, 14 miles northeast and 22.5 miles northeast of the site, respectively. No faults are shown on or in the immediate vicinity of the site on published geologic maps.

SEISMICITY

A map of recorded earthquake epicenters is included as Enclosure "A-4" (Epi Software, 2000). This map includes a database maintained by the Southern California Earthquake Center (University of Southern California) for earthquakes with magnitudes of 4.0 or greater from 1932 through 2012. The following table summarizes earthquakes that have occurred in the region of the site.

Summary of Historic Earthquakes				
Event ID	**Date**	**Magnitude**	**Distance from Site (miles)**	**Direction from Site**
Lake Matthews Area	4/21/1918	6.6	79	SE
Long Beach	3/10/1933	6.4	58	SE
Fish Creek Mountains	10/21/1942	6.6	178	SE
Borrego Mountain	4/9/1968	6.5	164	SE
West Hollywood	9/9/2001	5.9	21.5	SE
Whittier Narrows	10/1/1987	5.9	39	SE
Upland	2/28/1990	5.4	61	E
Sierra Madre	6/28/1991	5.8	46	NE
Mojave	7/11/1992	5.7	85	NE
Landers	6/28/1992	7.3	133	NE
Big Bear	6/28/1992	6.4	111	E
Northridge	1/17/1994	6.7	14	NE
Hector Mine	10/16/1999	7.1	147	NE
Fort Tejon	1/9/1857	7.9	134	NW
Chino Hills	7/29/2008	5.4	59	SE
Kern County (Tehachapi)	7/21/1952	7.3	62	NW
Inglewood	5/17/2009	4.7	28	SE
Upland	6/26/1988	4.8	60	E
Yorba Linda	9/3/1992	4.8	59	SE
Sylmar	2/9/1971	6.6	28	NE

SEISMIC DESIGN PARAMETERS

Based on the geologic setting and blowcount data from subsurface explorations, the soils underlying the site are classified as Site Class "C", according to the 2013 CBC.

The seismic design parameters in accordance with Section 1613A of 2013 CBC are presented in Table 2.1. These values were determined using the web-based application http://earthquake.usgs.gov/designmaps/us/application.php and the site coordinates 34.1475, W118.7659. The deaggregated modal earthquake magnitude was determined from the USGS website http://geohazards.usgs.gov/deaggint/2008 for evaluation of soil effects due to earthquake ground shaking.

2013 CBC - Seismic Design Parameters	
Mapped Spectral Acceleration Parameters	S_s = 1.559 and S_1 = 0.600
Site Coefficients	F_a = 1.0 and F_v = 1.3
Adjusted Maximum Considered Earthquake Spectral Response Parameters	S_{MS} = 1.559 and S_{M1} = 0.780
Design Spectral Acceleration Parameters	S_{DS} = 1.039 and S_{D1} = 0.520
Geometric Mean Peak Ground Acceleration (PGA_M)	0.579g
Deaggregated Modal Magnitude	7.03

GROUNDWATER AND LIQUEFACTION

Depth-to-groundwater data from the State of California Water Resources Control Board (2015) and groundwater contour mapping by CGS (2000) were examined for the area of the site. These data are summarized in the following table.

Depth to Groundwater				
Well No./ID	**Date Measured**	**Depth to Water (feet)**	**Measuring Point Elevation (feet amsl)**	**Location**
T06037041688-W-05DD	8/25/2009	6	871	1/4 mile S
	1/22/2010	6		
T0603703142-MW-K	9/1/2002	12	900	1/3 mile E
	10/1/2006	8		
	7/6/2009	6		
	4//2012	11		
T-0603703449-W-14	1/14/2004	14	886	1/3 mile SE
	10/10/2006	16		
	12/27/2014	15		
Contour Mapping	Historic High	10	--	--

Groundwater was not encountered within the maximum 50-1/2-foot depth of the explorations. Based on historical data and a site elevation of 900 feet, the historic high depth to groundwater in the area of the site is estimated at approximately 10 feet bgs.

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid. Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth), 2) the presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is not included in a State of California Seismic Hazard Zone for liquefaction or earthquake-induced landslide (CGS, 2000). Based on the composition of the underlying soils encountered in our geotechnical investigation and the relatively shallow depths of bedrock encountered at the site, liquefaction is not considered a potential hazard, and further investigation is not warranted.

SEISMIC SETTLEMENT

Severe seismic shaking may cause dry and saturated sands to densify, resulting in settlement expressed at the ground surface. Seismic settlement in dry soils generally occurs in loose sands and silty sands, with cohesive and fine-grained soils being less prone to significant settlement. For saturated soils, significant settlement is anticipated if the soils are liquefied during seismic shaking. Soil types susceptible to liquefaction include sand, silty sand, sandy silt and silt, as well as clayey soils with clay content less than 15 percent.

Topanga Formation siltstone was encountered at depths of 3 to 10 feet below the existing ground surface. Little to no alluvial sands were encountered in our investigation. Therefore, seismic settlement at the site is considered negligible.

STATIC SETTLEMENT

Potential static settlement was evaluated utilizing field and laboratory data and foundation load assumptions. We anticipate a total static settlement of less than 1 inch beneath foundations. Differential settlement is anticipated to be less than one-half the total settlement in 40 feet. Most of the potential static settlement should occur during construction.

HYDROCONSOLIDATION

Based on the relatively dense nature of the underlying near-surface soils encountered in our investigation, the minimum mandatory removal requirements as provided in the "Recommendations" section of this report and the low potential for full saturation of the soil layers, it is our opinion that the potential for hydrocollapse settlement at the site is low.

SUBSIDENCE

The site is not located within an area identified by the State of California Seismic Hazard Zone as having a potential for subsidence. The potential for subsidence to affect the proposed structure is considered low.

SLOPE STABILITY AND LANDSLIDE POTENTIAL

Based on information provided by the project civil engineer, a finished floor elevation of approximately 912 feet above mean sea level (amsl) is estimated for the project. The slope located on the northern portion of the site consists of tight, well-bedded siltstone with sandstone interbeds. Bedding was measured to dip to the north. Landslides were not observed within the site. The site is not located within a State-designated area as having a potential for landslide, seismically induced landslide or lateral spreading (CGS, 2000). Therefore, the potential for landsliding or lateral spreading is considered low.

Grading of cut or fill slopes, if needed to achieve final site configurations, should be conducted in conformance with applicable grading codes. On-site soils may be considered Type "B" with regard to 2013 CAL/OSHA excavation standards.

FLOODING AND EROSION

The site is not located in an area designated by the Federal Emergency Management Agency (2008) as a flood hazard zone. A more accurate determination of the flood hazard to the site and the adequacy of existing flood and drainage improvements near the site is not within the scope of this investigation.

No large water storage facilities are known to exist within the area of the site. The site is not located within a coastal area; therefore, tsunami is not a potential hazard to the site.

EXPANSION POTENTIAL

ASTM D4829 test standard classifies expansion index (EI) of soils as follows:

Expansion Index	Expansion Potential
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Greater than 130	Very High

According to Section 1803.5.3 of the 2013 CBC, soils having an EI greater than 20 are considered "expansive" and require foundation design to mitigate these conditions as per Section 1808.6 of the 2013 CBC.

EI analysis according to the ASTM standard was performed by this firm. The result indicates EI values of 150 and 157 ("very high"). Based on these results, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary. Recommendations to mitigate expansive soil conditions are provided in the "Expansive Soils" section of this report.

DOUBLE-RING INFILTROMETER TESTS

Two double-ring infiltrometer tests were performed to evaluate the infiltration potential of the site soils located within the proposed water retention area. The test locations are indicated on Enclosure "A-2". The tests were performed in general conformance with ASTM D3385 at depths of 3 and 5 feet below the existing ground surface utilizing a rubber-tire backhoe to excavate the test pits. Exploratory test pit logs are provided in Appendix "B".

The data collected were used to calculate the infiltration rate of the soil. The infiltration test was performed until a steady-state infiltration velocity was reached. The steady-state infiltration velocity is presented as the infiltration rate.

The infiltration rates are presented in the following table and do not include safety factors.

Test Number/Depth	Infiltration Rate	
	cm. / hr.	in. / hr.
P-1	0.13	0.05
P-2	0.07	0.03

The measured infiltration rates are within the applicable range of the test method. The measured infiltration rate to use in design is discussed in the "Storm Water Infiltration" section of this report. It should be noted that infiltration rates determined by testing are ultimate rates based on short-duration field test results. The infiltration tests utilized clear water, and infiltration rates can be affected by buildup of silt, debris, the degree of soil saturation and other factors. An appropriate safety factor should be applied to measured infiltration rates prior to use in design to accommodate potential subsoil inconsistencies, possible compaction related to site grading and potential silting of the percolating soils. A safety factor should be determined with consideration to other factors in the storm water retention system design, particularly storm water volume estimates and the safety factors associated with those design components.

CONCLUSIONS

On the basis of our research and field and laboratory investigations, it is the opinion of this firm that the proposed project is feasible from a geological and geotechnical engineering standpoint, provided the recommendations contained in this report are implemented during design and construction.

As encountered in the explorations, the site is mantled by colluvial fill to depths from approximately 3 to 5 feet below ground surface. The fill materials encountered consist of medium dense to dense clayey sand (SC) and stiff to hard fat clay (CH). The bedrock was encountered at depths of 3 to 10 feet bgs and consisted of Topanga Formation Siltstone recovered as silty and clayey sands (SM, SC), clays (CL, CH) and silt (ML). Refusal to further advancement of the drilling augers was not experienced in the exploratory borings. Caving was not experienced within the exploratory borings utilized for this investigation.

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone designated by the State of California to include traces of suspected active faulting.

Moderate to severe seismic shaking can be expected at the site.

Groundwater was not encountered within the maximum 50-1/2-foot depth of the explorations. Historic high groundwater is estimated to be at 10 feet bgs in the area of the site. Based on the composition of the underlying soils encountered in our geotechnical investigation and the relatively shallow depths of bedrock encountered at the site, liquefaction is not considered a potential hazard to the site.

Settlement resulting from seismic shaking is considered negligible. Hydroconsolidation potential is considered low for the site.

The potential for subsidence to affect the proposed structure is considered low.

The potential for landsliding or lateral spreading is considered low.

Expansion index testing yielded "very high" potential for expansion. Based on the EI test result, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary.

Based on the classification, density and lack of significant soil cementation encountered in exploratory borings placed within the site, site grading and utility trenching are expected to be feasible with conventional heavy grading and trenching equipment, respectively.

RECOMMENDATIONS

The recommendations provided in this report assume that on-site expansive soils will be utilized and foundations and slabs-on-grade will be designed for expansive deformations and pressures provided herein. Retaining walls will require imported, very low expansive (EI<21), granular soils as backfill. If additional recommendations for use of imported soils or conventional foundations are required, this firm should be contacted.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the 2013 CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped or cleaned of significant vegetation, rocks greater than 6 inches in largest dimension and other deleterious materials. These materials should be removed from the site for disposal.

The cleaned soils may be reused as properly compacted fill if foundations, which include slabs-on-grade, are designed as indicated in the "Expansive Soils" section of this report.

If encountered, existing utility lines should be traced, removed and rerouted from areas to be graded.

Cavities created by removal of subsurface obstructions such as structures, individual effluent disposal systems and trees should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for compacted fill.

MINIMUM MANDATORY REMOVAL AND RECOMPACTION OF EXISTING SOILS:

All areas to be graded should have at least the upper 5 feet of existing soils removed or expose siltstone bedrock, and the open excavation bottoms observed by our engineering geologist to verify and document in writing that all undocumented fill is removed prior to refilling with properly tested and documented compacted fill. The removed soils may only be used as compacted fill if foundations are designed as recommended in the "Expansive Soils" section of this report.

Further subexcavation may be necessary depending on the conditions of the underlying soils. The actual depth of removal should be determined at the time of grading by the project geotechnical engineer/geologist. The determination will be based on soil conditions exposed within the excavations.

Compaction tests may be taken in the removal bottom areas where appropriate to provide in-place moisture/density data for potential relative compaction evaluations and to help support and document the engineering geologist's decision. As such, all areas to be graded should have any undocumented fill, topsoil or other unsuitable materials removed and replaced with properly compacted fill. Fill may consist of suitable on-site material, imported material or a combination thereof depending on foundation design.

PREPARATION OF FILL AREAS:

Prior to placing fill, and after the mandatory subexcavation operation with all loose native and/or undocumented fill removed, the surfaces of all areas to receive fill should be scarified to a depth of 6 inches or more. The scarified soils should be brought to between optimum moisture content and 2 percent above optimum moisture content and recompacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

PREPARATION OF FOUNDATION AREAS:

For foundations designed for expansive soils as recommended in the "Expansive Soils" section of this report, the thickness of compacted fill underneath footings should be at least 3 feet and the removed soils may be used as compacted fill. In areas where the required thickness of compacted fill is not accomplished by site rough grading, mandatory subexcavation operation and the undocumented fill removal, the footing areas should be further subexcavated to a depth of at least 3 feet below the proposed footing base grade. The required overexcavation should extend at least 10 feet laterally beyond the footing lines, where possible. The bottom of this excavation should then be scarified to a depth of at least 6 inches, brought to between optimum moisture content and 2 percent above optimum moisture content and recompacted to a minimum of 90 percent relative compaction in accordance with ASTM D1557 prior to refilling the excavation to the required grade as properly compacted fill.

Thickness of compacted fill underneath foundations should not be allowed to vary by more than 50 percent or 4 feet, whichever is less, for a single foundation system. In areas where, by virtue of grading, the fill thickness will exceed this maximum allowable differential, the subexcavation depths should be increased as necessary to reduce the differential fill thickness. This deepening of the subexcavation may involve additional removals of native soils. A determination of specific structural areas that require additional subexcavation should be performed at the time of grading.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for compacted fill.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material provided they are free from organic matter and other deleterious materials and foundations and slabs-on-grade are designed for expansive soils as indicated in the "Expansive Soils" section of this report. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 8 inches should not be buried or placed in fills.

If utilized, import materials should be inorganic, very low-expansive (EI<21), granular soil free from rocks or lumps greater than 6 inches in maximum dimension. The contractor shall notify the geotechnical engineer of import sources sufficiently ahead of their use so that the sources can be observed and approved as to the physical characteristic of the import material. For all import material, the contractor shall also submit current verified reports from a recognized analytical laboratory indicating that the import has a "not applicable" (Class S0) potential for sulfate attack based upon current American Concrete Institute (ACI) criteria and is not corrosive to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the job.

Fill should be spread in near-horizontal layers, approximately 8 inches thick. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 2 percent above optimum moisture content, and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557.

It is crucial that the geotechnical engineer or representative be present to observe the grading operations. Monitoring of the soil expansion potential by the geotechnical engineer during the grading operation should be performed regularly. Further recommendations may be made in the field, depending on the actual conditions encountered.

SLOPE CONSTRUCTION:

Slopes should be constructed no steeper than 2(h):1(v). Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

SLOPE PROTECTION:

Inasmuch as the native materials are susceptible to erosion by wind and running water, it is our recommendation that the slopes at the project be protected from erosion as soon as possible after completion. On permanent slopes the use of succulent ground covers, such as ice plant or sedum, is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the water system and to prevent overwatering.

Measures should be provided to prevent surface water from flowing over slope faces.

FOUNDATION DESIGN:

Foundations and slabs-on-grades should be designed to resist the effects of expansive soils. Structural design measures including design of slab-on-grade foundations in accordance with "WRI/CRSI Design of Slab-On-Ground Foundations" or "PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations of Expansive Soils" would be necessary. Foundations should also be designed to prevent uplift of the supported structure and resist forces exerted on the foundation due to soil volume change or shall be isolated from the expansive soil as indicated in Sections 1808.6.1 and 1808.6.2 of the 2013 California Building Code.

For foundations designed for expansive soils, bearing on a minimum of 3 feet of compacted fill, footings may be designed for a maximum safe soil bearing pressure of 1,000 pounds per square foot (psf) for dead plus live loads. The bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, we would anticipate a maximum static settlement of less than 1 inch. Differential static settlement between similarly loaded adjacent footings is expected to be approximately half the total settlement. Static settlement is expected to occur during construction or shortly after. Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for compacted fill.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and cohesion. For footings bearing against on-site compacted fill, allowable passive earth pressure may be considered to be developed at a rate of 100 psf per foot of depth. Passive earth pressure only applies to level, properly drained backfill with no additional surcharge loadings. Cohesion may be computed as 130 psf. Cohesion and passive earth pressure may be combined without reduction.

Cohesion value is to be multiplied by the contact area, as limited by Section 1806.3.2 of the 2013 CBC. The lateral passive earth pressure and cohesion values are provided from Table 1806.2 of the 2013 CBC.

The resistance values provided do not consider expansive pressures of the on-site soils. Expansive pressures should be taken into account during design of foundations.

For preliminary retaining wall design, lateral active earth pressures indicated in the table below should be utilized for properly drained backfill with no additional surcharge loadings.

Lateral Active Earth Pressures	
Backfill Inclination	**Active (psf/ft)**
Level	40
3(h):1(v)	55
2(h):1(v)	65

For restrained conditions, an at-rest earth pressure of 65 psf per foot of depth should be utilized for level, properly drained backfill with no additional surcharge loadings.

The "at-rest" condition applies toward braced walls that are not free to tilt. The "active" condition applies toward unrestrained cantilevered walls where wall movement is anticipated. The structural designer should use judgment in determining the wall fixity and may utilize values interpolated between the "at-rest" and "active" conditions where appropriate.

The values for earth pressures are based on imported backfills consisting of inorganic, very low-expansive (EI<21), granular, compacted fill, and assume that soils will have a phi angle of 30 degrees and a unit weight of 120 pounds per cubic foot. These values should be verified by an engineer from this firm when import materials are selected. These values do not include a factor of safety other than conservative modeling of the soil strength parameters.

RETAINING WALL BACKFILL:
Backfill behind retaining walls should consist of a soil of sufficient granularity that the backfill will properly drain. The granular backfill shall extend from the bottom of the wall at a 1(h):1(v) plane to the surface. The granular soil should be classified per the USCS as GW, GP, SW, SP, SW-SM or SP-SM and should have a minimum phi angle of 30 degrees and a unit weight of 120 pounds per cubic

foot. Surface drainage should be provided to prevent ponding of water behind walls. A drainage system should be installed behind all retaining walls consisting of either of the following:

1. A 4-inch-diameter perforated PVC (Schedule 40) pipe or equivalent at the base of the stem encased in 2 cubic feet of granular drain material per linear foot of pipe or

2. Synthetic drains such as Enkadrain, Miradrain, Hydraway 300 or equivalent.

Perforations in the PVC pipe should be 3/8 inch in diameter. Granular drain material should be wrapped with filter cloth such as Mirafi 140 or equivalent to prevent clogging of the drains with fines. Walls should be waterproofed to prevent nuisance seepage. Water should outlet to an approved drain.

SEISMIC LATERAL EARTH PRESSURE (CANTILEVERED WALL):

The seismic earth pressure acting on a cantilevered retaining wall was calculated using the Mononobe-Okabe ("M-O") method (Okabe, 1926; Mononobe and Matsuo, 1929). According to AASHTO (LRFD Bridge Design Specifications, Sixth Edition, 2012, Section C11.8.6.2 and A11.3.2), the resulting pseudostatic horizontal seismic coefficient, k_h, could be reduced by 50 percent when 1.0 to 2.0 inches of permanent ground deformation is permitted during the design seismic event, i.e., the pseudostatic horizontal seismic coefficient (k_h) can be taken as equal to one-half of the PGA, which equates to 0.29g. The pseudostatic vertical seismic coefficient (k_v) is usually taken as 0.0g. For retaining walls with imported backfills consisting of inorganic, very low-expansive (EI<21), granular, compacted fill, a unit weight of 120 pounds per cubic foot (pcf) and a friction angle of 30 degrees were utilized in the calculation. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to properly drained backfill with no additional surcharge loadings.

The total lateral active seismic earth pressures (including static active earth pressures) to be utilized for unrestrained conditions are provided in the following table.

Lateral Active Seismic Earth Pressures	
Backfill Inclination	**Active Seismic (psf/ft)**
Level	70
3(h):1(v)	125
2(h):1(v)	135

A triangular distribution of total seismic earth pressure should be used in the design (Atik and Sitar, 2010).

SLABS-ON-GRADE:

Slabs-on-grade should be designed to resist the expansive soils as provided in the "Expansive Soils" section of this report.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute 302.1R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder/barrier should comply with ASTM E1745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage. Per the Portland Cement Association (www.cement.org/tech/cct_con_vapor_retarders.asp), for slabs with vapor-sensitive coverings, a layer of dry, granular material (sand) should be placed under the vapor retarder/barrier. For slabs in humidity-controlled areas, a layer of dry, granular material (sand) should be placed above the vapor retarder/barrier.

A modulus of vertical subgrade reaction of 100 kips per cubic foot can be utilized in the design of slabs-on-grade for the proposed project.

EXPANSIVE SOILS:

The expansion index testing performed for this report indicated a "very high" potential for expansion (EI of 150 and 157) in the upper soil layers. Based on these results, construction procedures and/or special structural design to specifically mitigate the effects of expansive soil movements are necessary, as recommended below.

Structural design measures, including design of slab-on-grade foundations in accordance with "WRI/CRSI Design of Slab-On-Ground Foundations" or "PTI Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations of Expansive Soils", should be taken into consideration for this project. Foundations should also be designed to prevent uplift of the supported structure and resist forces exerted on the foundation due to soil volume change or shall be isolated from the expansive soil as indicated in Sections 1808.6.1 and 1808.6.2 of the 2013 California Building Code.

The expansive potential deformation within the upper 5 feet of clayey soils is expected to be approximately 1-1/2 inches (expansive strain of 2.4%). An expansive pressure of 7,000 psf should be used in the design of the foundations and slab-on-grade.

Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during grading in order to provide the geotechnical parameters required for the design. Utilities should also be designed for potential expansive deformation and pressure.

POTENTIAL EROSION AND DRAINAGE:

The potential for erosion should be mitigated by proper drainage design. The site should be graded in such a way that surface water flows away from structures. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be planted or otherwise protected from erosion by wind or water.

STORM WATER INFILTRATION:

Based on the measured infiltration rates, we recommend that a design infiltration rate of 0.03 inches per hour be used for the design of the storm water disposal system(s) on site. An appropriate safety factor should be applied to the recommended infiltration rate prior to use in design to accommodate potential subsoil inconsistencies, possible compaction related to site grading and potential silting of the percolating soils. A safety factor should be determined with consideration to other factors in the storm water retention system design, particularly storm water volume estimates and the safety factors associated with those design components.

As the design infiltration rate is very low, alternative measures to storm water abatement should be considered.

TRENCH EXCAVATION:

The soils encountered within our exploratory borings are generally classified as a Type "B" soil in accordance with the CAL/OSHA excavation standards. Unless specifically evaluated by our engineering geologist, all the trench excavations should be performed following the recommendation of CAL/OSHA (State of California, 2013) for Type "B" soil. Based upon a soil classification of Type "B", the temporary excavation should not be inclined steeper than 1(h):1(v) for maximum trench depth of less than 20 feet. For trench excavation deeper than 20 feet or for conditions that differ from those described for Type "B" in the CAL/OSHA excavation standards, this firm should be contacted.

TRENCH BEDDING AND BACKFILLS:

Trench Bedding - Pipe bedding material should meet and be placed according to the current edition of the Standard Specifications for Public Works Construction "Greenbook" or other project specifications. Pipe bedding should be uniform, free-draining, granular material with a sand equivalent of at least 30. The pipe bedding material should be evaluated to confirm sand equivalent values by this firm prior to use as pipe bedding material.

Backfill - The on-site expansive soils may be utilized for trench backfill if utilities are designed to accommodate the expansive deformations and pressures provided in the "Expansive Soils" section of this report. Rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in backfills.

Fill to be compacted by heavy equipment should be spread in near-horizontal layers, approximately 8 inches in thickness. For fill to be compacted by hand-operated equipment, thinner lifts, 4 to 6 inches in thickness, should be utilized. Each lift should be spread evenly, brought to between optimum moisture content and 2 percent above optimum moisture content and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D1557. To avoid pumping, backfill material should be mixed and moisture treated outside of the excavation prior to lift placement in the trench.

Soils required to be compacted to at least 95 percent relative compaction, such as pavement subgrade, should also be moisture treated to near optimum moisture content not exceeding 2 percent above optimum moisture content.

As an alternative, a controlled low-strength material (CLSM) could be considered to fill trenches, cavities, such as voids created by caving or undermining of soils beneath existing improvements or pavement to remain, or any other areas that would be difficult to properly backfill.

CHEMICAL/CORROSIVITY TESTING:

Selected samples of materials were delivered to HDR, Inc. for soil corrosivity testing. Laboratory testing consisted of pH, resistivity and major soluble salts commonly found in soils. The results of the laboratory tests performed by HDR, Inc. appear in Appendix "C".

These tests have been performed to screen the site for potentially corrosive soils. Values from the soil tested are considered "mildly corrosive" to ferrous metals at as-received moisture condition and "corrosive" at saturated condition. Specific corrosion control measures, such as coating of the pipe with non-corrosive material or alternative non-metallic pipe material, are considered necessary.

Ammonium and nitrate levels did not indicate a concern as to corrosion of buried copper.

Results of the soluble sulfate testing indicate a "not applicable" (Class S0) anticipated exposure to sulfate attack. Based on the criteria from Table 4.3.1. of the "American Concrete Institute Manual of Concrete Practice" (2011), no special measures, such as specific cement types or water-cement ratios, will be required.

The soluble chloride content of the soils tested was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

CHJ Consultants does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, is required, then a competent corrosion engineer could be consulted.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The geotechnical engineer's field representative will be present to provide observation and field testing and will not supervise or direct any of the actual work of the contractor, his employees or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

CHJ Consultants has striven to perform our services within the limits prescribed by our client and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of CHJ Consultants. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions that appear different from those described herein be encountered in the field by the client or any firm performing services for the client or the client's assign, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this firm at your convenience.

LICENSED PROFESSIONAL ENGINEER
MAIHAN NOORZAY
No. 77901
Exp. 06-30-2017
CIVIL
STATE OF CALIFORNIA

Respectfully submitted,
CHJ CONSULTANTS

Maihan Noorzay, R.C.E. 77901
Project Engineer

John S. McKeown

John S. McKeown, C.E.G. 2396
Project Geologist

REGISTERED GEOLOGIST
JOHN S. McKEOWN
NO. 2396
CERTIFIED ENGINEERING GEOLOGIST
Exp. 06-30-17
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
JAMES F. COOKE
No. 3012
Exp. 12-31-15
GEOTECHNICAL
STATE OF CALIFORNIA

James F. Cooke, G.E. 3012
Managing Engineer

Robert J. Johnson, P.E.
President

MN/JSM/JFC/RJJ:lb

REFERENCES

American Concrete Institute, 2011, Manual of Concrete Practice, Part 3, Table 4.3.1.

Atik, L., and Sitar, N., 2010, Seismic Earth Pressures on Cantilever Retaining Structures, Journal of Geotechnical and Geoenvironmental Engineering, Volume 136, No. 10, October 1, 2010, Pages 1324-1333.

California State Water Resources Control Board, 2010, http://geotracker.swrcb.ca.gov.

Coduto, D. P., Yeung, M. R., and Kitch, W. A., 2010, Geotechnical Engineering Principles and Practices, 2nd Edition, Pearson Higher Education, Inc., New Jersey.

Dibblee, T.W., Jr. and Ehrenspeck, H.E., 1993, Geologic Map of the Thousand Oaks quadrangle, Ventura and Los Angeles Counties, California, Dibblee Geological Foundation Map #DF-49.

Dickinson, W. R., 1996, Kinematics of trans-rotational tectonism in the California Transverse Ranges and its contribution to cumulative slip along the San Andreas transform fault system: Geological Society of America Special Paper 305.

Epi Software, 2000, Epicenter Plotting Program.

Federal Emergency Management Agency (FEMA), 2008, FIRM Map Panel No. 06037C1244F, dated August 28, 2008.

International Conference of Building Officials, 2013, California Building Code, 2013 Edition, Whittier, California.

Mononobe, N., and H. Matsuo (1929), "On the determination of earth pressures during earthquakes". Proceedings World Engineering Congress, Vol. 9.

Okabe, S. (1926), "General theory of earth pressure." Japan Society of Civil Engineers, Vol. 12, No. 1, Tokyo.

Pradel, D., 1998, "Procedure to Evaluate Earthquake-Induced Settlement in Dry Sand Soils", Journal of Geotechnical and Geoenvironmental Engineering, Vol 124, No. 4.

Seismic Hazard Evaluation of the Thousand Oaks 7.5 Minute Quadrangle, Ventura County, California: California Division of Mines and Geology, Open-File Report 2000-008.

Yi, F., 2015, "GeoSuite version 2.3.0.5", GeoAdvanced.

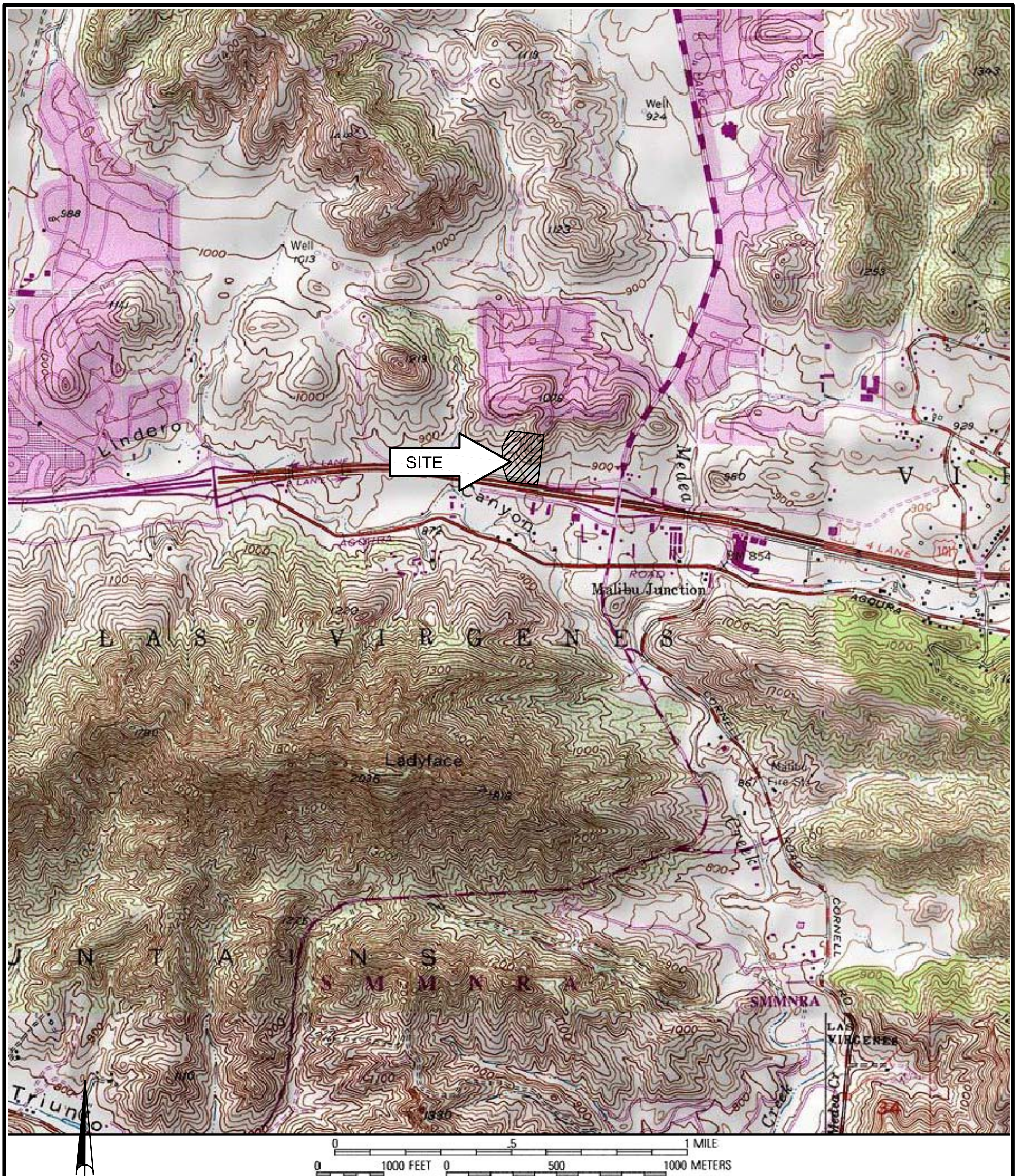
LIST OF AERIAL PHOTOGRAPHS

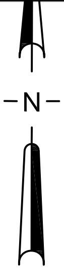
Google Earth web-based software application, aerial imagery dated July 17, 1989, May 31, 1994, June 9, 2002, December 21, 2005, March 15, 2006, December 31, 2006, August 31, 2007, January 8, 2008, May 24, 2009, July 23, 2014, and May 1, 2015.

Netroline, aerial imagery dated 1947, 1952, 1959, 1967, 1980 and 1989.

APPENDIX "A"

GEOTECHNICAL MAPS





SCALE: 1" = 2000'

INDEX MAP

FOR: OAKMONT SENIOR LIVING	GEOTECHNICAL INVESTIGATION OAKMONT OF AGOURA HILLS SENIOR FACILITY APN 2053-001-005 29353 CANWOOD STREET AGOURA HILLS, CALIFORNIA	ENCLOSURE "A-1"
DATE: OCTOBER 2015		JOB NUMBER 15473-3

CHJ Consultants