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LEGEND

BROWN	- BUILDING & SAFETY DIVISION COMMENTS
PURPLE	- PLANNING DEPARTMENT COMMENTS
GREEN	- ENVIRONMENTAL PROGRAMS DEPARTMENT COMMENTS
LIGHT ORANGE	- TRANSPORTATION DEPARTMENT COMMENTS
BLUE	- FLOOD CONTROL DISTRICT COMMENTS
LIGHT GREEN	- REGIONAL PARKS & OPEN SPACE DISTRICT COMMENTS
RED	- FIRE DEPARTMENT COMMENTS
YELLOW	- ENVIRONMENTAL HEALTH DEPARTMENT COMMENTS
LIGHT BLUE	- BUILDING & SAFETY PLAN CHECK COMMENTS
ORANGE	- WASTE MANAGEMENT DEPARTMENT COMMENTS

**Geotechnical Investigation Report Update
Proposed Extended Stay Lodges at:
25840 Idyllwild Road
Idyllwild, Riverside County, California**

Project No. GD19G508-1

July 7, 2020

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Attention: Mr. Wade Shuey
Principal - Owner

Subject: Geotechnical Investigation Report Update

Proposed Expansion of Extended Stay Lodges at:

25840 Idyllwild Road, Idyllwild, Riverside County, CA 92549

Reference: *Limited Geotechnical Investigation by Geodyne Engineering, Inc.*

Dated November 11, 2019, Geodyne Project No.GD19G508 (Attached as Appendix D)

Dear Mr. Shuey:

In accordance with your request, this report has been prepared to update our referenced limited geotechnical investigation report to 2019 CBC and to provide recommendations for the currently proposed development. It is our opinion that the recommendations provided in our referenced report are still applicable to the currently proposed development unless revised herein.

Based on this study, it is also our opinion that the currently proposed construction is feasible from the geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development.

We appreciate and wish to thank you for the opportunity to serve you on this project. Please do not hesitate to contact us if we can be of additional assistance during subsequent phases of this project.

Respectfully submitted,
Geodyne Engineering, Inc.



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Registration Exp. Date: 03-31-2022



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APPENDIX A

Vicinity Map
Geotechnical Map

APPENDIX B

Logs of Exploratory Test Pits

APPENDIX C

ASCE 7 Hazards Tools Detailed Output Report Per ASCE/SEI 7-16 Standard

APPENDIX D

Limited Geotechnical Investigation by Geodyne Engineering, Inc.
Dated November 11, 2019, Geodyne Project No.GD19G508

INTRODUCTION

Purpose

The purpose of this report update and supplemental Geotechnical Investigation is to provide preliminary geotechnical information to WSCS Design (“Client”) regarding the subject properties located in the unincorporated community of Idyllwild, Riverside County, California. There are three (3) noncontiguous properties in close proximity to each other that are the subject of this Limited Geotechnical Investigation (herein referred to as “Lot 1”, Lot “2” and Lot “3”). Geodyne Engineering, Inc. understands that the Client is considering developing the subject properties and requires a Limited Geotechnical Investigation.

The information gathered in this Limited Geotechnical Investigation is intended to provide the Client with an understanding of the physical conditions of site-specific subsurface soils, groundwater, and the regional geologic setting which could affect the cost or design of the proposed development at the subject properties (Vicinity Map, Site Plan – **Appendix A**).

This Limited Geotechnical Investigation has been conducted in general accordance with accepted geotechnical engineering principles and in general conformance with the discussed scope and verbal authorization by the Client.

Geodyne Engineering, Inc. conducted the first phase of our onsite field exploration for Lot “1” which was completed on November 5, 2019 that included excavation of four (4) backhoe test pits (designated TP-1 through TP-4) for the proposed development at the subject property. Geodyne Engineering, Inc. conducted the second phase of our onsite field exploration on June 23, 2020 that included the excavation of five (5) additional backhoe test pits (designated TP-5 through TP-9) for Lot “2”; and seven (7) additional backhoe test pits (designated TP-10 through TP-16) for Lot “3”. A total of sixteen (16) backhoe test pits were excavated for our Limited Geotechnical Investigation of the subject properties. This Limited Geotechnical Investigation has been prepared for the sole use of WSCS Design. Other parties, without the express written consent of Geodyne Engineering, Inc. should not rely upon this Limited Geotechnical Investigation.

Project Description

Based on information provided by the Client, we understand that the development of the subject properties will include:

- Lot “1” – Proposed development of Lot “1” will consist of the construction of three (3) new single-story buildings for a total of eleven (11) lodging units for seasonal stay lodging within the rear, northeast portion of the subject property. Each building will contain six (6),

three (3) and two (2) lodging units, respectively. Other proposed improvements include paved parking and drive areas and other related improvements. An existing shed and several trees will be removed prior to constructing the new development.

- Lot “2” – Proposed development of Lot “2” will consist of the construction of five (5) new single-story buildings for a total of sixteen (16) lodging units also for seasonal stay lodging spread across the subject property. Each building will contain seven (7), three (3), two (2), two (2) and two (2) lodging units, respectively. Other proposed improvements include paved parking and drive areas and other related improvements. An existing outbuilding and several trees will be removed prior to constructing the new development.
- Lot “3” – Proposed development of Lot “3” will consist of the construction of three (3) new single-story buildings for a total of thirteen (13) lodging units for more seasonal stay lodging, mainly spread over the western two-thirds of the subject property. Each building will contain six (6), four (4) and three (3) lodging units, respectively. Other proposed improvements include paved parking and drive areas and other related improvements. Lot “3” is currently undeveloped but has a moderate cover of large trees spread across the site, some of which will need to be removed prior to constructing the new development.

No detailed grading plans were provided to Geodyne Engineering, Inc. at the time of our preparation of this report; however, grading within each individual lot is anticipated to include cuts and fills of less than 5 feet across the subject property (exclusive of remedial grading). No foundation plans were provided to Geodyne Engineering, Inc. at the time of report preparation; however, foundation loads are assumed to be typical for the type of construction. No other information is known at this time.

Scope of Services

The scope of our services included:

- A review of readily available data pertinent to the subject property, including published and unpublished geologic reports/maps, and soils data for the area (**References**).
- Conducting a geotechnical reconnaissance of the subject property and nearby vicinity.
- Excavation and logging of sixteen (16) exploratory test pits (designated TP-1 through TP-16) by means of a backhoe excavator in readily accessible areas of the subject properties to depths ranging from approximately 1 foot to 8 feet below the existing ground surface (bgs) (**Appendix B**).

- An evaluation of seismicity and geologic hazards to include an evaluation of faulting and liquefaction/seismic-induced settlement potential.
- Completion of limited laboratory testing (moisture & density) of representative earth materials encountered onsite to ascertain their pertinent soils engineering properties
- The preparation of this report which presents our preliminary findings, conclusions, and recommendations.

BACKGROUND

Subject Properties Descriptions

Based on information provided by Client (WSCS Designs – Site Plan Concept, 2019), we understand that the proposed seasonal lodging development is composed of three (3) separate noncontiguous but adjacent properties east of Idyllwild Road (State Route 243) and southeast of Jameson Drive in the unincorporated community of Idyllwild, Riverside County, California. For clarity, the three subject properties are referred to as Lot “1” (APN: 563-250-028), Lot “2” (APN: 563-250-031) and Lot “3” (APN: 563-250-017). Lot “1” is associated with an address of 25840 Idyllwild Road, is located approximately 450 feet southeast of Jameson Drive, along and immediately east of Idyllwild Road. The boundary of Lot “2” is located about 20 feet east of the boundary of Lot “1”, separated by Oakwood Street. Lot “3” is located immediately northwest of Lot “1” along the east side of Idyllwild Road and also southeast of Jameson Drive. Lot “1” is essentially located between Lot “2” to the east and Lot “3” to the northwest.

Lot “1” - Based on our review of online aerial photography (GoogleEarth®, 2019) and our site reconnaissance, the irregularly-shaped subject property is currently partially developed. The subject property covers an area of approximately 1.45-acres and can be divided into two parcels or sections. The southwestern portion of the subject property covers an area of slightly less than 1-acre and is currently developed with three (3) single floor buildings and paved parking and drive areas comprising an old motel business that has been converted into apartments. The presently developed portion of Lot “1” is not a part of the subject proposed site development. The proposed seasonal lodging development is located on the northeastern portion of the subject Lot “1” property covering an area of approximately 0.5-acre, located approximately 200 feet northeast of Idyllwild Road. The subject parcel to be developed has generally bare ground that has been cleared with minor leveling for use as a parking area. This portion of the property also has an existing storage shed and scattered large evergreen trees with sparse forest cover. The surface gradient of the

subject property appears to be less than 5:1 (horizontal to vertical). The site is accessible via a driveway from Idyllwild Road and Oakwood Street.

The center of the subject Lot “1” property is approximately situated at 33.748788° degrees North latitude and 116.713981° degrees West longitude (Google Earth®, 2019). The property is bounded by Idyllwild Road (State Route 243) on the west, the Idyllwild Bible Church (54400 Pine Crest Avenue) with residential properties and open space forest to the north and northeast.

Lot “2” - Based on our review of online aerial photography (GoogleEarth®, 2019) and our site reconnaissance, the roughly rectangular-shaped subject property is generally undeveloped and covers an area of approximately 0.98-acres. Lot “2” is generally located immediately east of Lot “1” approximately 400 feet east of Idyllwild Road and 250 feet northwest of Pine Crest Avenue with Oakwood Street forming the southwestern boundary of the property. The subject parcel generally has a bare ground surface that appears to have been cleared with minor leveling of the ground surface. A small outbuilding and several parked fifth-wheel type trailers are present on the site. Scattered large evergreen trees with sparse forest cover is also present. The surface gradient of the subject property appears similar to the adjacent Lot “1” at less than approximately 5:1 (horizontal to vertical). The site is accessible through Lot “1” and from Oakwood Street via Pine Crest Drive.

The center of the subject Lot “2” property is approximately situated at 33.749283° degrees North latitude and 116.712648° degrees West longitude (Google Earth®, 2019). The property is bounded by Oakwood Street (across from Lot “1”) on the southwest with the remainder of the property surrounded by residential properties and open space forest.

Lot “3” - Based on our review of online aerial photography (GoogleEarth®, 2019) and our site reconnaissance, the irregularly-shaped subject property appears to be undeveloped and covers an area of approximately 1.3-acres. Lot “3” is generally located immediately northwest of the northwest corner of Lot “1” and is located along the east side of Idyllwild Road and approximately 150 feet southeast of Jameson Drive. The subject parcel is undeveloped with possible minor areas of clearing. No structures are present on the subject property. Scattered large evergreen trees with sparse forest cover is present across the property. The surface gradient of the subject property appears to be generally steeper than the adjacent Lot “1” at approximately 3:1 to 4:1 (horizontal to vertical). Low cut slopes of approximately 5 feet in height with exposed granitic bedrock are present along at the western edge of the property along Idyllwild Road.

The center of the subject Lot “3” property is approximately situated at 33.749747° degrees North latitude and 116.713876° degrees West longitude (Google Earth®, 2019). The property is bounded by Idyllwild Road on the west, unimproved forest land on the north and northwest, residential

properties and open space forest to the northeast and east, the existing “developed” portion of Lot “1” to the south and the “undeveloped” portion of Lot “1” to the southeast.

Topography

The subject properties are located in Section 7 of Township 5 South, Range 3 East within the Idyllwild, California 7.5-minute Quadrangle (USGS, 2018). For Lot “1”, the ground surface at the property gently slopes from northeast to southwest and ranges in elevation from roughly 5,490 feet to 5,465 feet above mean sea level (amsl). Surface drainage across the site is through sheetflow, generally to the southwest. For Lot “2”, the ground surface at the subject property also gently slopes to the southwest with surface elevations ranging from approximately 5,515 feet to 5,490 feet above mean sea level (amsl) with surface drainage across the subject property by sheetflow. For Lot “3”, the ground surface at the subject property also slopes relatively gently from northeast to southwest with surface elevations ranging from approximately 5,500 feet to 5,455 feet above mean sea level (amsl) with the lowest site elevations along Idyllwild Road.

FIELD EXPLORATION AND LABORATORY TESTING

Field Exploration

Field work for our Limited Geotechnical Investigation was conducted on November 1 and 5, 2019 (Lot “1”) and June 23, 2020 (Lots “2” and “3”). For Lot “1”, our initial field work on November 1, 2019 at the site consisted of attempting the excavation of hand-dug exploratory test pits at the subject property. Due to difficult hand digging conditions and shallow refusal, field work was terminated and, with the concurrence of the Client, it was decided to use a backhoe at a later date to excavate the test pits. On November 5, 2019, a total of four (4) exploratory test pits (designated TP-1 through TP-4) were excavated at the subject property. The test pit excavations ranged in depth from approximately 2.5 feet to 7.5 feet below the existing ground surface (bgs). For Lots “2” and “3”, a total of twelve (12) additional exploratory test pits (designated TP-5 through TP-16) were excavated at the subject properties on June 23, 2020. The test pit excavations ranged in depth from approximately 1 foot to 8 feet below the existing ground surface (bgs). All test pit excavations were logged under the supervision of a Registered Professional Engineer and Certified Engineering Geologist at Geodyne Engineering, Inc. The approximate locations of the test pits are shown in Figure B, **Appendix A**.

A small CAT, track backhoe excavator was used to excavate the exploratory test pits. Equipment refusal was encountered within test pit excavations due to the presence of hard, granitic bedrock at depth. The subsurface lithology is recorded on the test pit logs, which are presented in

Appendix B. The soils were classified in accordance with the Unified Soil Classification System (ASTM, 2015). Representative bulk samples were also collected for appropriate laboratory testing. Attempts were made to retrieve undisturbed ring samples. However, the samples were either disturbed or there was no recovery due to the friable nature of the underlying soil and bedrock. During our investigation on June 23, 2020, we were able to retrieve bedrock's ring samples generally from the lower depths only at lot 2.

Laboratory Testing

Selected samples obtained from our test pits were tested to evaluate pertinent soil classification and engineering properties and to enable development of geotechnical conclusions and recommendations. The laboratory tests of samples retrieved during our investigation on June 23, 2020 were limited to moisture and density determination. Due to the general similarities of soil and bedrock encountered in our geotechnical investigation on November 5, 2019, additional testing deemed unnecessary.

SUBSURFACE CONDITIONS

Geologic Setting

Regionally, the subject properties lie within the Peninsular Ranges Geomorphic Province of southern California. This province consists of a series of ranges separated by northwest trending valleys; sub parallel to branches of the San Andreas Fault (CGS, 2002). The Peninsular Ranges geomorphic province, one of the largest geomorphic units in western North America, extends from the Transverse Ranges geomorphic province and the Los Angeles Basin, south to Baja California. It is bound on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks (CGS, 2002). Major fault zones and subordinate fault zones found in the Peninsular Ranges Province typically trend in a northwest-southeast direction. Locally, the subject properties are located on the southwest flank of the San Jacinto Mountains.

Regional geologic maps of the subject properties and vicinity (Dibblee and Minch, 2008) indicate the properties are underlain by older surficial sediments mapped as Pleistocene-age alluvial fan gravel (map unit Qof), which is described as weakly indurated and dissected. Map unit Qof was not encountered within our exploratory test pits at the subject properties. However, the subject site properties are located near a geologic contact with Cretaceous-age plutonic rocks which compose the Peninsular Ranges Batholith. These plutonic rocks were encountered within our test pits and are described as Cretaceous-age quartz diorite or granodiorite (i.e. "granitic") that is grayish-white in color and massive to slightly gneissoid with minor xenoliths.

Subsurface Conditions

The materials encountered in our test pits consisted of undocumented artificial fill and weathered (“decomposed”) granitic bedrock. A brief description of the subsurface conditions is provided in the following section. Detailed descriptions of the subsurface conditions are provided on the test pit logs included in **Appendix B**.

Artificial Fill – For Lot “1” - Artificial fill was encountered in three (3) of our test pits at the ground surface and extended to depths ranging from 2 to 3 feet below the existing ground surface in test pits TP-1, TP-2 and TP-3. Artificial fill was not encountered within test pit TP-4. For Lot “2” – Artificial fill was encountered within all five test pits (TP-5 through TP-9) at the ground surface and extended to depths of 3.5 to 4 feet below the existing ground surface. For Lot “3” – Artificial fill was encountered within test pit TP-15 only at the ground surface and extended to a depth of approximately 3 feet below existing grade. As encountered within all three of the subject properties, the artificial fill generally consists of light grayish-brown or brownish-gray, dry to damp, medium dense, fine to coarse grained sand with pebbles and minor silt with tree roots up to 3-inches in diameter at the time of our subsurface exploration.

The artificial fill was likely placed in conjunction with the existing motel/apartment development (western portion of Lot “1”) and during minor site grading and re-leveling for parking areas and also the adjacent grading associated with the nearby existing residential developments.

Decomposed Granite (“DG”) – For Lot “1” - Decomposed granite bedrock (appears to be weathered granodiorite) was encountered beneath the existing artificial fill within test pits TP-1, TP-2 and TP-3 and was encountered at the ground-surface in test pit TP-4, and extended to the maximum explored depth of 7.5 feet below the existing ground surface. For Lot “2” - Decomposed granite bedrock (appears to be weathered granodiorite) was encountered beneath the existing artificial fill within all five test pits (TP-5 through TP-9), and extended to the maximum explored depth of 8 feet below the existing ground surface. For Lot “3” - Decomposed granite bedrock (appears to be weathered granodiorite) was encountered generally at the ground surface (except for a thin veneer of topsoil) within six of seven test pits (TP-10 through TP-14, TP-16) and extended to the maximum explored depth of 5 feet below the existing ground surface. In test pit TP-15, decomposed granite bedrock was encountered at a depth of 3 feet below the existing ground surface underlying the artificial fill. The decomposed granite materials at the time of our subsurface exploration can be described as tan or light brown to grayish-brown, highly weathered, friable with medium to coarse texture and moderately hard to hard becoming hard and relatively unweathered at approximate depths ranging from 2.5 to 8 feet below existing ground surface.

Groundwater

Static groundwater or seepage was not encountered in any of our exploratory test pits to a maximum depth of 8 feet bgs at the time of exploration. Based on the presence of shallow, hard granitic bedrock at the ground surface to very shallow depths, static groundwater is not anticipated beneath the site. It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation, and other factors that may not have been evident at the time of our subsurface exploration.

GEOLOGIC HAZARDS

California Building Code Seismic Design Parameters

Geodyne Engineering, Inc. utilized seismic design criteria provided in the CBC (2019) and ASCE 7-16. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism. The site coefficients and adjusted maximum considered earthquake spectral response accelerations in accordance with the 2019 California Building Code are presented in **Table 1**.

TABLE 1 2019 CBC Seismic Parameters and Peak Ground Acceleration	
Parameter	Value
Site Coordinates (Lot 1)	Latitude 33.748788° Longitude -116.713981°
Mapped Spectral Acceleration Value at Short Period: S_s	1.5g
Mapped Spectral Acceleration Value at 1-Second Period: S_1	0.6g
Site Classification	C
Short Period Site Coefficient: F_a	1.2
1-Second Period Site Coefficient: F_v	1.4
Site-Modified Spectral Acceleration Value: S_{MS}	1.8g
Site-Modified Spectral Acceleration Value: S_{M1}	0.84g
Design Spectral Response Acceleration at Short Periods: S_{DS}	1.2g
Design Spectral Response Acceleration at 1-Second Period: S_{D1}	0.56g
Peak Ground Acceleration adjusted for Site Class Effects: $PGAM$	0.651g

ASCE 7 Hazards Report per ASCE/SEI 7-16 Standard is attached in Appendix C. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism.

Faulting and Surface Rupture

The subject properties are located within an area of California known to contain a number of active and potentially active faults. There are no known active faults crossing the subject properties (Jennings and Bryant, 2010) and the properties are not within a State of California Earthquake Fault Zone (CDMG, 1974). The closest known active fault is the San Jacinto Valley segment of the San Jacinto Fault Zone, located approximately 7.41 miles (11.93 km) southwest of the subject properties (USGS, 2008). Therefore, the potential for surface rupture at the properties is considered low. Three of the closest faults along with their distance from the property (Lot “1”) and Maximum Magnitude are shown in **Table 2**.

TABLE 2 Nearby Active Faults		
Fault	Distance in Miles (Kilometers)¹	Maximum Magnitude¹
San Jacinto SJV+A+CC+B	7.41 (11.93)	7.70
San Jacinto SBV+SJV+A+CC	7.41 (11.93)	7.76
San Jacinto SJV+A	7.41 (11.93)	7.47

1. USGS Online Fault Search (2008)

Landslides and Slope Stability

The subject properties and surrounding areas are generally relatively flat with near-surface or surface exposures of granitic bedrock. As a result, we consider the potential for landslides or slope instabilities to occur at the properties to be negligible.

Liquefaction Potential and Lateral Spreading

Liquefaction occurs when loose, saturated, generally cohesionless fine sands and non-plastic silts are subjected to strong ground shaking. The soils lose shear strength and become liquid; potentially resulting in large total and differential ground surface settlements as well as possible lateral spreading during an earthquake. Seismically induced settlement can occur in response to liquefaction of saturated loose granular soils, as well as the reorientation of soil particles during strong shaking of loose, unsaturated sands.

Due to the lack of shallow groundwater at the subject site, and the presence of near-surface to surface granitic bedrock, the probability of liquefaction, seismically induced settlement, and lateral

spreading at the subject site is considered to be negligible.

Tsunamis, Flooding and Seiches

The subject property is not located within a Tsunami Evacuation Area; therefore, damage due to tsunamis is considered low.

Geodyne reviewed the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) online database (FEMA, 2019) to determine if the subject property was in a flood zone. According to Panel No. 06065C2155G - effective August 28, 2008, the subject property is located within Flood Zone X. FEMA defines Zone X as an area of minimal flood hazard, usually depicted on FIRMs as outside the 500-year flood plain.

Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The subject property is not located immediately adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered low.

FOUNDATION RECOMMENDATIONS

Foundation on Expansive Soils

Expansive soils change in volume with change in moisture content. Shrinking and swelling of the clays can cause heaving and cracking on slab-on-grade and structures founded on shallow foundations. The results of our exploration, laboratory testing and engineering tactile evaluation indicate the soils underlying this site have a Very Low Expansion Potential characteristics. As such, special measures per 2019 California Building Code (CBC) Section 1803.5.3 are not required to mitigate expansive soil. However, we recommend the expansion index to be verified during the rough grading/earthwork of the site.

SHALLOW FOUNDATION

Foundation support for the proposed construction could be derived by utilizing a shallow conventional continuous or spread foundation system extended below frost zone or insulated, embedded within the underlying bedrock or engineered fill compacted to 90%. An ultimate bearing capacity of 6000 psf and an allowable bearing capacity of 2000 psf may be used for foundation bearing on bedrock or compacted fill. The recommended values are for a footing 18" wide and 18 inch deep. It may be increased by 300 psf for every additional foot of width or depth (3500 psf Max). A one-third increase in bearing capacity may be used for resistance against seismic and wind loading.

The recommended passive earth pressure (equivalent fluid weight) is 250 pcf (Max. 3000 pcf). Minimum reinforcement for 18-inch-deep and 18-inch-wide continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom.

FRICTION PILES

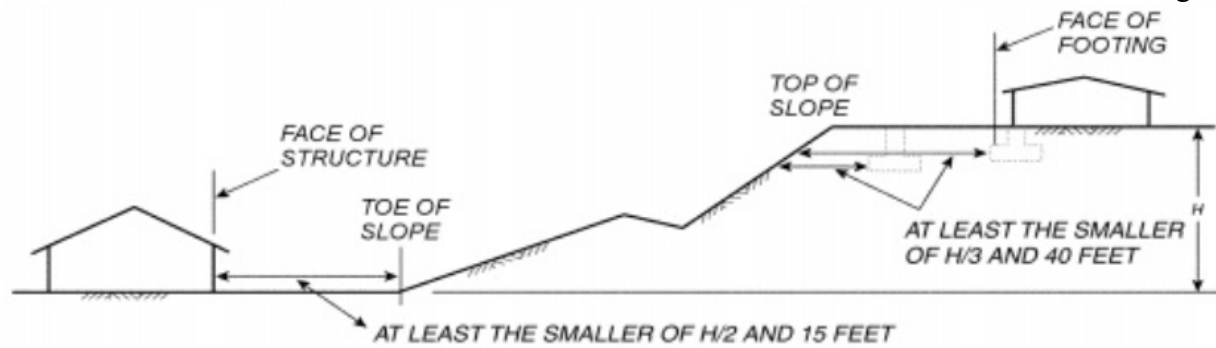
As an option, foundation support for the proposed construction could be derived by utilizing a friction pile foundation system. A minimum 24-inch diameter piers embedded a minimum of 5 ft into the underlying bedrock should be considered. The capacity of fill should be ignored unless certified. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces. An allowable skin friction resistance of 400 psf may be used for the design of friction pile system. The piers should not be spaced more than three diameters on center each way. For piers with closer spacing, a load capacity reduction factor of 0.75 due to pile group effects may be considered.

Retaining Wall

Embedded structural walls should be designed for lateral earth pressures exerted on the walls. The magnitude of these earth pressures will depend on the amount of deformation that the wall can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the active condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized, and the earth pressures will be higher. These walls such as basement walls and swimming pools should be designed for the at rest condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the passive resistance. For design purposes, the recommended equivalent fluid pressure for each case for walls constructed above the static groundwater table and backfilled with very low-expansive soils is provided below. Retaining wall backfill should be compacted to at least 95% relative compaction based on the maximum density of select import defined by ASTM D1557.

A conventional shallow minimum 24-inch square spread or minimum 24-inch diameter pier foundation system embedded within the underling compacted fill or bedrock may be used to support the proposed retaining wall. Spread or pier foundation system should be properly reinforced, embedded a minimum of 36" below the finished sub grade elevation. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces.

In cases where the ground surface slopes away from the foundation, passive resistance will be developed below an imaginary horizontal line that intersects the ground surface. As such, the requirements of 2019 CBC, Figure 1808.7.1 Foundation Clearances from Slopes should be adhered to as illustrated below:



For SI: 1 foot = 304.8 mm.

FIGURE 1808.7.1

FOUNDATION CLEARANCES FROM SLOPES

Retaining structure may be designed per the following geotechnical parameters:

- Allowable Bearing Pressure: 2000 psf

The bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3500 psf.

- Coefficient of Friction: 0.40
- Soil Unit Weight: 120 pcf
- Allowable skin friction resistance: 400 psf
- Passive Earth Pressure - equivalent fluid weight of 250 pcf (Max. 3000 pcf)

Frictional resistance and passive pressure resistance may be used in combination if friction coefficient is reduced by one-third. A one-third increase in passive pressure may be used for resistance against seismic and wind loading.

Active earth pressures behind walls depend on wall movement, backfill slope, surcharge loads and back fill material.

Table No. 4

Equivalent Fluid Density	
(PCF)	Level backfill
Active Condition	40
At-rest Condition	65

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls.

The backfill behind the wall should be drained properly. The simplest drainage system consists of a drain system located near the bottom of the wall. The drain collects the water that enters the backfill and this may be disposed of through outlets along the base of the wall. To ensure that the drains are not clogged by fine particles, they should be surrounded by a granular filter wrapped in a geofabric such as Mirafi 140N or equivalent. Despite a well-constructed toe drain, substantial water pressure may develop behind the wall if the backfill consists of clays or silts. A more satisfactory drainage system, consisting of a back drain of 12 inches to 24 inches width gravel may be provided behind the wall to facilitate the drainage. All back drains should be provided with outlet to suitable drainage facilities.

SEISMICALLY INDUCED LATERAL EARTH PRESSURES

Retaining wall design calculations for lateral earth pressure due to earthquake motions for walls higher than 6 feet, as required by section 1803.5.12 of the 2019 CBC is discussed below.

We have used $\frac{1}{2}$ of $\frac{2}{3}$ the PGA_M in our analysis ($\frac{2}{6}$) (0.651 g) = 0.217g . Evaluation of lateral earth pressures under static and seismic loading conditions is based on using the Coulomb (1776) and Mononobe-Okabe (1929) Methods for frictional backfill materials with little to zero cohesion. For a level backfill, we recommend using a granular frictional soil material which exhibits friction angle 30 degrees or better. If this material is used, we recommend using combined of static and dynamic active equivalent earth pressure 60 pcf. For walls with a retained height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall designs be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 22 pcf. Such walls that are to be designed in the static case assuming the at-rest condition should be checked seismically using this additive seismic EFP added to the active condition (i.e., the additive seismic EFP is not added to the at-rest EFP). The additive seismic EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

Slab on Grade

Slabs on grade should be underlain by a layer of four (4) inches free drainage 3/4" crushed rocks over firm native or approved structural fill. For the design of the slab on grade, modulus of subgrade reaction (k_1) of 100 kef is recommended. Slab thickness, reinforcement etc., should be selected by the structural engineer based on the analysis performed considering the loads anticipated, sub grade expansion potential and the modulus of subgrade reaction of the soil. As minimum, we recommend a 4.0-inch thick slab thickness, reinforced with #4 bars at 18 inch on center. The subgrade for the new slab should be prepared as recommended under Section 8.2 "Site Preparation." A vapor barrier over the crushed rock or approved subgrade soil should be considered in areas where the migration of moisture through the floor slab would be detrimental. To protect the vapor barrier (Visqueen) from punctures during placement, it is recommended that the Visqueen be placed over two-inch thick, clean sand layer. The vapor barrier should be at least 10-mil plastic (STEGO or Equivalent) and should be sealed at all splices, around plumbing, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier and care should be taken not to puncture the membrane. Some contractors exercising special care use heavier membranes or double layers of 10-mil plastic with splices staggered and sealed. All new slab design should be in compliance with the applicable sections of 2019 Cal Green.

SITE GRADING

As per 2019 CBC Section 1804.4, the ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet of horizontal distance, a 5-percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet of the building foundation. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building.

PLAN REVIEWS

It is recommended that copies of the final grading and foundation plans, when they become available, be provided to our office for review.

CLOSURE

This report may serve as an update to our referenced geotechnical report. Provided that the updated recommendations contained within this report are properly implemented, our referenced geotechnical investigation report is considered valid for the currently proposed development.

It is recommended that a complete copy of this update report, along with the original geotechnical report, be provided to all contractors and consultants associated with this project.

We sincerely appreciate the opportunity to be of continued service on this project. We look forward to providing additional consulting services during the course of the project.

REFERENCES

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads for Buildings and Other Structures, ASCE Document ASCE/SEI 7-16.

American Society for Testing and Materials (ASTM), 2015, Annual Book of ASTM Standards, Volume 04.08, Construction: Soil and Rock (I), Standards D 420 - D 5876.

California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, California Building Standards Commission, Based on 2018 International Building Code; 2019 California Historical Building Code, Title 24, Part 8; and 2016 California Existing Building Code, Title 24, Part 10, effective January 1, 2020.

California Green Code (CGC 2019)

California Geological Survey (CGS) Seismic Hazard Information
<https://maps.conservaion.ca.gov/cgs/EQZApp/app/#>

ASCE 7 Hazards Tools <https://asce7hazardtool.online/>

California Division of Mines and Geology (CDMG), 1974, State of California, Special Studies Zones, NW ¼ Idyllwild Quadrangle, Official Map – Effective: July 1, 1974.

California Geological Survey (CGS), 2002, California Geomorphic Provinces Note 36, Electronic Copy, Revised December 2002.

Dibblee, T.W. and Minch, J.A., 2008, Geologic Map of the Hemet & Idyllwild 15-Minute Quadrangles, Riverside County, California, Dibblee Geological Foundation Map DF-371, scale – 1:62,500.

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map 06065C2155G, Riverside County Unincorporated Areas, California, dated August 28, 2008.

GeoTracker Website, 2019, State Water Resources Control Board, website address - <http://geotracker.waterboards.ca.gov/>, accessed November 2019.

GoogleEarth, 2018, Version 7.1.8.3036.

Hart, E.W., and Bryant, W.A. (Hart and Bryant), 1997, Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.

Jennings, C.W., and Bryant, W.A., (Jennings and Bryant) 2010, Fault Activity Map of California and Adjacent Areas: California Geologic Survey, Map Sheet No. 6, scale 1:750,000.

Riverside County Integrated Project – Planning Home – TMLA website, http://planning.rctlma.org/Portals/0/genplan/content/gp/chapter06.html#TOC3_6, accessed November 2019.

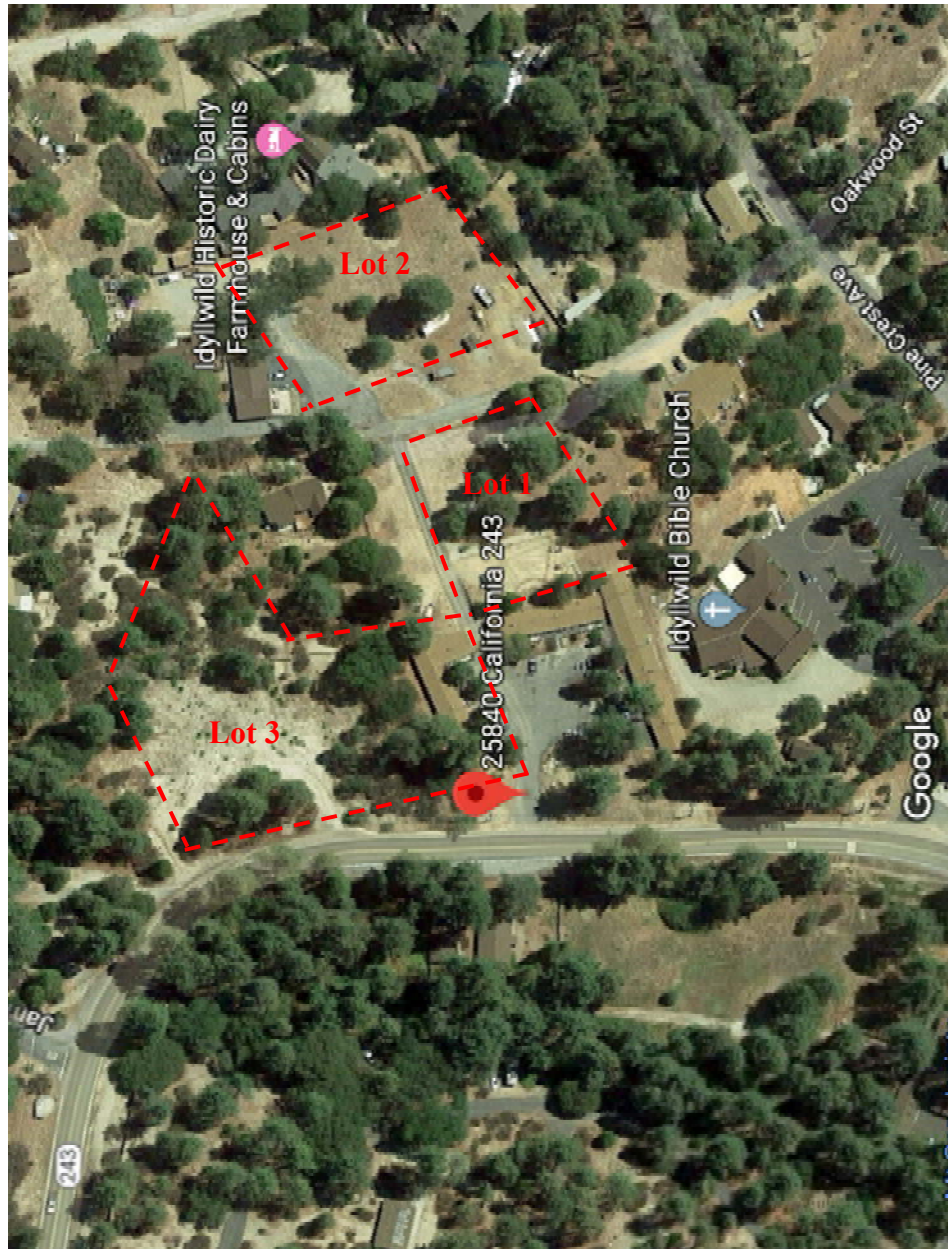
United States Geological Survey (USGS), 2018, 7.5-Minute Topographic Map, Idyllwild, California Quadrangle, scale 1:24,000.

United States Geological Survey (USGS), 2008, 2008 National Seismic Hazard Maps – Online Fault Database Search, web address < <http://earthquake.usgs.gov/hazards/products/conterminous/>>, accessed November 2019.

WSCS Designs, 2019, Site Plan Concept, APN: 563-250-017, -028, -031, Idyllwild Rd., Idyllwild, CA, Scale: 1"=40' (reduced), Project 19-324, dated May 13, 2019.

APPENDIX A

Vicinity Map
Geotechnical Map



NORTH
NTS

Proposed Expansion of Extended Stay Lodges at:
25840 Idyllwild Road, Idyllwild, California

GEODYNE ENGINEERING, INC.

Vicinity Map

Project No. GD19G508-1

Figure: A

Number	Revision	Description	Date

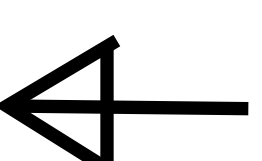


GEODNEYE ENGINEERING, INC.
Geotechnical Map
Geodyne Project No. GD19G508-1

Figure: B

LEGENDS:

- TP-4 Approximate Test Pit Locations by Geodyne Engineering, Inc. Excavated on 11-05-2019
- TP-16 Approximate Test Pit Locations by Geodyne Engineering, Inc. Excavated on 06-23-2020

NORTH 
Scale: 1" = 40'

APPENDIX B

Logs of Exploratory Test Pits

Unified Soil Classification System

Major Divisions			Letter	Symbol	Description
Coarse-grained Soils More than ½ retained on the No. 200 Sieve	Gravels More than ½ coarse fraction retained on the No. 4 sieve	Clean Gravels	GW		Well-graded gravels and gravel-sand mixtures, little or no fines.
			GP		Poorly-graded gravels and gravel-sand mixtures, little or no fines.
		Gravels With Fines	GM		Silty gravels, gravel-sand-silt mixtures.
			GC		Clayey gravels, gravel-sand-clay mixtures.
	Sands More than ½ passing through the No. 4 sieve	Clean Sands	SW		Well-graded sands and gravelly sands, little or no fines.
			SP		Poorly-graded sands and gravelly sands, little or no fines.
		Sands With Fines	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sandy-clay mixtures.
Fine-grained Soils More than ½ passing through the No. 200 Sieve	Silts and Clays Liquid Limit less than 50%		ML		Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL		Organic clays of medium to high plasticity.
	Silts and Clays Liquid Limit greater than 50%		MH		Inorganic silts, micaceous or diatomaceous fines sands or silts, elastic silts.
			CH		Inorganic clays of high plasticity, fat clays.
			OH		Organic clays of medium to high plasticity.
Highly Organic Soils		PT		Peat, muck, and other highly organic soils.	
Consistency Classification					
Granular Soils				Cohesive Soils	
Description - Blows Per Foot (Corrected)				Description - Blows Per Foot (Corrected)	
Very loose	MCS <5	SPT <4	Very soft	MCS <3	SPT <2
Loose	5 - 15	4 - 10	Soft	3 - 5	2 - 4
Medium dense	16 - 40	11 - 30	Firm	6 - 10	5 - 8
Dense	41 - 65	31 - 50	Stiff	11 - 20	9 - 15
Very dense	>65	>50	Very Stiff	21 - 40	16 - 30
			Hard	>40	>30
MCS = Modified California Sampler			SPT = Standard Penetration Test Sampler		

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 5
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube ☒ Bulk Sample ☒ 2.5" Ring Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Commets
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5	SW	☒	Bareground, <u>0.0'-3.5': Topsoil/Artificial FILL (Qaf)</u> Brownish gray fine-coarse SAND, slightly silty, with pebbles, loose, dry-damp, tree roots,		2.8	117.3				
			<u>3.5'-8.0': Decompsed granite (DG)</u> Brown to light olive brown, weathered, friable,							
			End of Test Pit @ 8.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,							

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 6
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Thin Wall Tube ☐ Standard Split Spoon Sample ☒ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
	SW			Bareground, <u>0.0'-4.0': Topsoil/Artificial FILL (Qaf)</u> Brown, fine-coarse SAND, with pebbles, loose, dry-damp,						
5		☒		<u>4.0'-6.0': Decompsed granite (DG)</u> Brownish olive gray, weathered, friable, massive,		3.8	113.7			
				End of Test Pit @ 6.0-feet No groundwater or caving Excavation backfilled with cutting spoil,						
										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 7
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube ☒ 2.5" Ring Sample ☒ Bulk Sample ☐ Standard Split Spoon Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
	SW			Bareground, <u>0.0'-3.5': Topsoil/Artificial FILL (Qaf)</u> Brownish gray, fine-coarse SAND, trace silt, with pebbles, loose, dry-damp, rootlets, <u>4.0'-6.0': Decompsed granite (DG)</u> White, weathered, friable, massive, End of Test Pit @ 6.0-feet No groundwater or caving Excavation backfilled with cutting spoil,						
5		☒			2.9	115.1				
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 8
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Thin Wall Tube ☐ Standard Split Spoon Sample ☒ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
	SOIL DESCRIPTION									
5	SW	☒	Bareground, <u>0.0'-3.5': Topsoil/Artificial FILL (Qaf)</u> Brownish olive gray, fine-coarse SAND, trace silt, with pebbles, loose, dry-damp,		2.8	117.2				
			<u>4.0'-6.0': Decompsed granite (DG)</u> Yellow, white, gray ,weathered, massive,							
			End of Test Pit @ 6.0-feet No groundwater or caving Excavation backfilled with cutting spoil,							
										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 9
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5	SW			Bareground, <u>0.0'-3.5': Topsoil/Artificial FILL (Qaf)</u> Brownish gray fine-coarse SAND, slightly silty, with pebbles, loose, dry-damp, tree roots,		1.8	119.5			
			<u>3.5'-7.0': Decompsed granite (DG)</u> Light gray, white, brown, weathered, massive,							
			End of Test Pit @ 7.0-feet No groundwater or caving Excavation backfilled with cutting spoil,							
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 10
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube ☒ Bulk Sample Standard Split Spoon Sample Static Water Table ☒ 2.5" Ring Sample	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
				Bareground/Topsoil <u>0.5'-2.0': Decompsed granite (DG)</u> Light gray, white, brown, hard, slightly weathered,						
				End of Test Pit @ 2.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,						



LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 11
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		SOIL DESCRIPTION	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments	
		Undisturbed	Bulk								
				☒ Bulk Sample ☐ Thin Wall Tube ☐ Standard Split Spoon Sample ☒ 2.5" Ring Sample ☐ Static Water Table							
				Bareground/Topsoil <u>0.0'-1.0' Decompsed granite (DG)</u> Light gray and brown, hard, slightly weathered, End of Test Pit @ 1.0-ft							Proposed Parking Area
											

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 12
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5				Bareground/Topsoil <u>0.5'-3.0': Decompsed granite (DG)</u> Light brownish gray, hard, slightly weathered,						
				End of Test Pit @ 3.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,						
										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 13
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5			Bareground/Topsoil <u>0.5'-2.0': Decompsed granite (DG)</u> White to Light gray, hard, slightly weathered,							
			End of Test Pit @ 2.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,							
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 14
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Thin Wall Tube ☐ Standard Split Spoon Sample ☐ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
SOIL DESCRIPTION										
				Bareground/Topsoil <u>0.5'-2.0': Decompsed granite (DG)</u> White to Light gray, hard, weathered, End of Test Pit @ 2.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,						
5										
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 15
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Standard Split Spoon Sample ☒ Thin Wall Tube ☒ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5				0.0'-3.0': Topsoil/Artificial FILL (Qaf) Brown, fine-coarse SAND, slightly silty, with pebbles, loose, dry-damp, tree roots,						
				0.3'-5.0': Decompsed granite (DG) White to Light gray, hard, slightly weathered,						
				End of Test Pit @ 5.0-feet No groundwater or caving Excavation backfilled with cutting spoil,						
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 6/23/2020

TP No: 16
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavation/Trenching

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Thin Wall Tube ☐ Standard Split Spoon Sample ☒ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
				Bareground/Topsoil <u>0.5'-2.0': Decompsed granite (DG)</u> White to Light gray, hard, slightly weathered, End of Test Pit @ 2.0-feet No groundwater or caving Excavation backfilled wih cutting spoil,						
5										
10										
15										

APPENDIX C

ASCE 7 Hazards Tools Detailed Output Report Per ASCE/SEI 7-16 Standard

ASCE 7 Hazards Report

Address:

25840 Idyllwild Rd
Idyllwild, California
92549

Standard:

ASCE/SEI 7-16

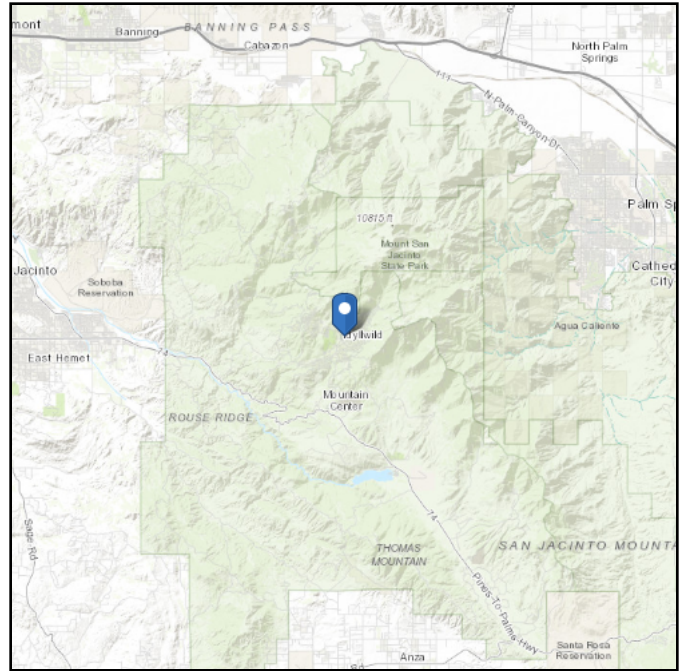
Risk Category: II**Soil Class:**

C - Very Dense
Soil and Soft Rock

Elevation: 5422.98 ft (NAVD 88)

Latitude: 33.748788

Longitude: -116.713981



Wind

Results:

Wind Speed:	96 Vmph
10-year MRI	67 Vmph
25-year MRI	73 Vmph
50-year MRI	78 Vmph
100-year MRI	82 Vmph

Data Source: ASCE/SEI 7-16, Fig. 26.5-1B and Figs. CC.2-1–CC.2-4

Date Accessed: Fri Jun 26 2020

Value provided is 3-second gust wind speeds at 33 ft above ground for Exposure C Category, based on linear interpolation between contours. Wind speeds are interpolated in accordance with the 7-16 Standard. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (annual exceedance probability = 0.00143, MRI = 700 years).

Site is not in a hurricane-prone region as defined in ASCE/SEI 7-16 Section 26.2.

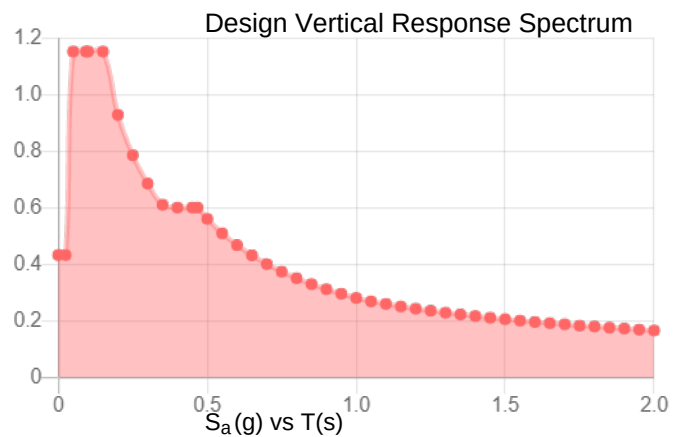
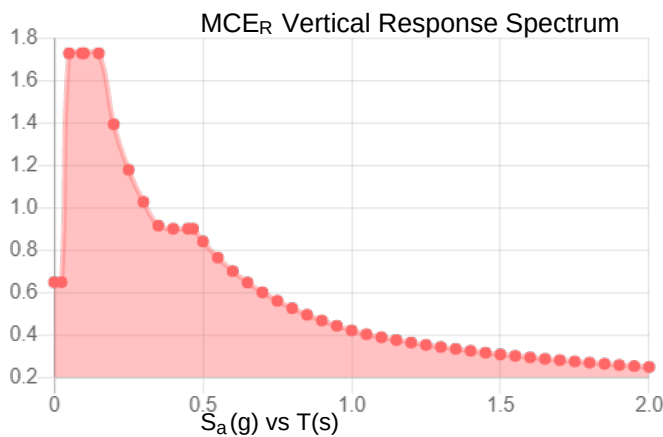
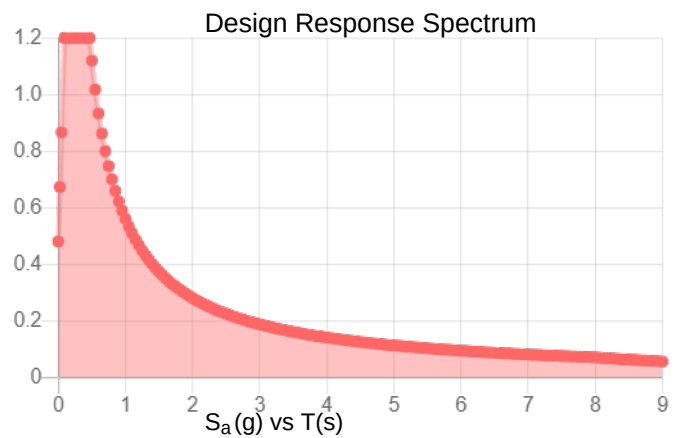
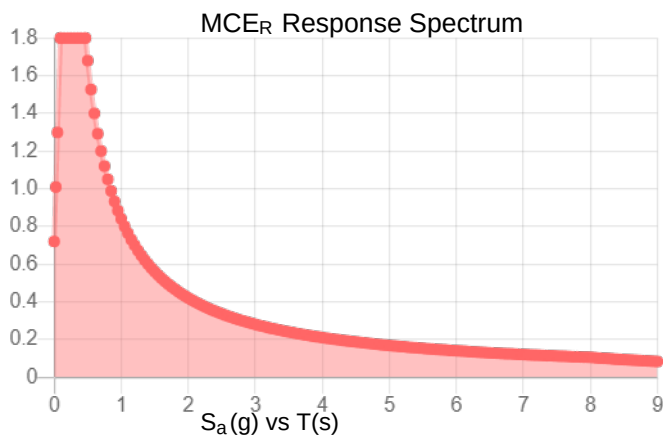
Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.

Site Soil Class: C - Very Dense Soil and Soft Rock

Results:

S_S :	1.5	S_{D1} :	0.56
S_1 :	0.6	T_L :	8
F_a :	1.2	PGA :	0.542
F_v :	1.4	PGA _M :	0.651
S_{MS} :	1.8	F_{PGA} :	1.2
S_{M1} :	0.84	I_e :	1
S_{DS} :	1.2	C_v :	1.2

Seismic Design Category D



Data Accessed:

Fri Jun 26 2020

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

Ice

Results:

Ice Thickness: 0.00 in.

Concurrent Temperature: 25 F

Gust Speed: 30 mph

Data Source: Standard ASCE/SEI 7-16, Figs. 10-2 through 10-8

Date Accessed: Fri Jun 26 2020

Ice thicknesses on structures in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Values provided are equivalent radial ice thicknesses due to freezing rain with concurrent 3-second gust speeds, for a 500-year mean recurrence interval, and temperatures concurrent with ice thicknesses due to freezing rain. Thicknesses for ice accretions caused by other sources shall be obtained from local meteorological studies. Ice thicknesses in exposed locations at elevations higher than the surrounding terrain and in valleys and gorges may exceed the mapped values.

Snow

Results:

Elevation: 5423.0 ft

Data Source: ASCE/SEI 7-16, Table 7.2-8

Date Accessed: Fri Jun 26 2020

In "Case Study" areas, site-specific case studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2 percent annual probability of being exceeded (50-year mean recurrence interval).

Values provided are ground snow loads. In areas designated "case study required," extreme local variations in ground snow loads preclude mapping at this scale. Site-specific case studies are required to establish ground snow loads at elevations not covered.

Rain

Results:

15-minute Precipitation Intensity: 3.08 in./h

60-minute Precipitation Intensity: 1.89 in./h

Data Source: NOAA National Weather Service, Precipitation Frequency Data Server, Atlas 14
(<https://www.nws.noaa.gov/oh/hdsc/>)

Date Accessed: Fri Jun 26 2020

Results:

Flood Zone Categorization: X (unshaded)

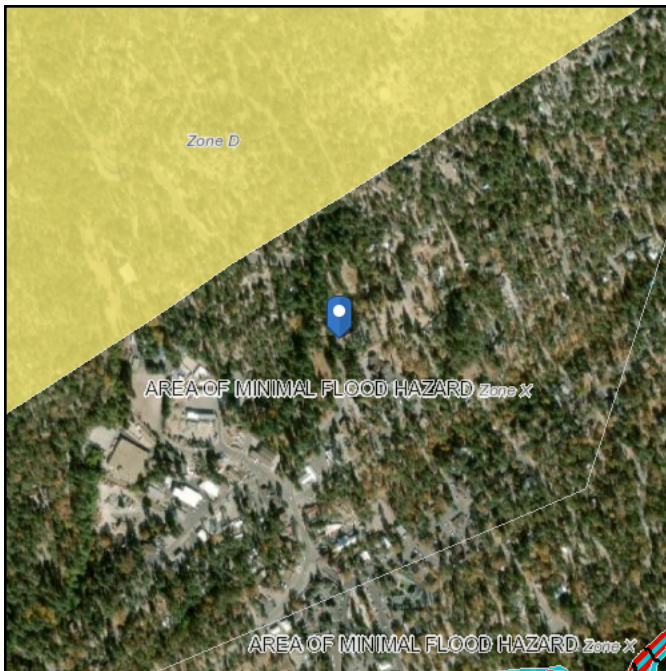
Base Flood Elevation:

Data Source: FEMA National Flood Hazard Layer - Effective Flood Hazard Layer for US, where modernized (<https://msc.fema.gov/portal/search>)

Date Accessed: Fri Jun 26 2020

FIRM Panel: If available, download FIRM panel [here](#)

Insurance Study Note: Download FEMA Flood Insurance Study for this area [here](#)



Tsunami

Results:

Tsunami: Not in mapped tsunami design zone.

Data Source: [ASCE Tsunami Design Geodatabase](#)

Date Accessed: Fri Jun 26 2020

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

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APPENDIX D

Limited Geotechnical Investigation by Geodyne Engineering, Inc.
Dated November 11, 2019, Geodyne Project No.GD19G508



**GEODYNE
ENGINEERING, INC.**

Geotechnical & Environmental Consultants

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**Limited Geotechnical Investigation
Proposed Extended Stay Lodges at:
25840 Idyllwild Road
Idyllwild, CA 92549**

Project No. GD19G508

November 11, 2019



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November 11, 2019

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Attention: Mr. Wade Shuey
Principal - Owner

Subject: **Limited Geotechnical Investigation**
Proposed Extended Stay Lodges at:
25840 Idyllwild Road, Idyllwild, Riverside County, CA 92549
Project No. GD19G508

Dear Mr. Shuey:

Per request, Geodyne Engineering, Inc. has performed a limited geotechnical investigation at the subject site. The purpose of our investigation was to evaluate the geotechnical conditions at the site in the areas of proposed construction and to provide geotechnical parameters for design and construction.

Based on our investigation, it is our opinion that the proposed construction is feasible from the geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development.

We appreciate and wish to thank you for the opportunity to serve you on this project.

Respectfully submitted,
Geodyne Engineering, Inc.



A. Wahab Noori., P. E.
Senior Engineer / (RCE C 081696)
Registration Exp. Date: 03-31-2020



Craig C. Chase, P.G., C.E.G.
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Registration Exp. Date: 06-30-2021

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APPENDIX A

Vicinity Map
Test Pit Locations Plan
USGS Topographic Map

APPENDIX B

Key to Classification Terms and Symbols
Test Pit Logs: TP-1, TP-2, TP-3, TP-4

APPENDIX C

Laboratory Tests

APPENDIX D

ATC-127 - Applied Technology Councils/USGS Site Specific Seismic Hazards by Location

EXECUTIVE SUMMARY

The soil conditions at the site of the proposed construction at 25840 Idyllwild Road, Idyllwild, County of Riverside, California, were explored by excavating two test pits up to a depth of 7.0-ft below existing grade. Laboratory tests were performed on selected specimens to evaluate the engineering characteristics of various soil strata encountered in our exploratory test pits. This report presents a description of subsurface conditions encountered at the site, recommended foundation systems, and design and construction criteria influenced by the subsurface conditions. It is based on data obtained from field investigations, laboratory test results and our previous experience with similar sites.

- Foundation support for the proposed construction could be derived by utilizing a shallow conventional continuous or spread foundation system extended below frost zone, embedded within the underlying bedrock or engineered fill compacted to 90%. An ultimate bearing capacity of 6000 psf and an allowable bearing capacity of 2000 psf may be used for foundation bearing on bedrock or compacted fill.
- As an option, foundation support for the proposed construction could be derived by utilizing a friction pile foundation system. A minimum 24 inch diameter piers, embedded a minimum of 5 ft into the underlying bedrock should be considered. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces. An allowable skin friction resistance of 300 psf may be used for the design of friction pile system. The piers should not be spaced more than three diameter on center each way. For piers with closer spacing, a load capacity reduction factor of 0.75 due to pile group effects may be considered.
- In cases where the ground surface slopes away from the foundation, passive resistance will be developed below an imaginary horizontal line that intersects the ground surface. As such, the requirements of 2016 CBC, Figure 1808.7.1 Foundation Clearances from Slopes should be adhered to as deemed appropriate.
- The project plans were not available during this investigation. Therefore, the recommendation of this report should be considered preliminary. Once the design phase proceeds to a more finalized plan, the recommendations within this report should be reviewed by our firm and revised as needed. Grading and earthwork remedial improvement recommendations will be provided based on project grading and structural plans.

Detailed descriptions of subsurface conditions, engineering analysis, and design recommendations are included in this report.

1.0 INTRODUCTION

PURPOSE

The purpose of this Limited Geotechnical Investigation is to provide preliminary geotechnical information to WSCS Design (“Client”) regarding the subject property located in the unincorporated community of Idyllwild, Riverside County, California. Geodyne Engineering, Inc. understands that the Client is considering developing the subject property and requires a Limited Geotechnical Investigation.

The information gathered in this Limited Geotechnical Investigation is intended to provide the Client with an understanding of the physical conditions of site-specific subsurface soils, groundwater, and the regional geologic setting which could affect the cost or design of the proposed development at the property (Vicinity Map, Site Plan – **Appendix A**).

This Limited Geotechnical Investigation has been conducted in general accordance with accepted geotechnical engineering principles and in general conformance with the discussed scope and verbal authorization by the Client.

Geodyne Engineering, Inc. conducted onsite field exploration that was completed on November 5, 2019, that included excavation of four (4) backhoe test pits (designated TP-1 through TP-4) for the proposed development at the subject property. This Limited Geotechnical Investigation has been prepared for the sole use of WSCS Design. Other parties, without the express written consent of Geodyne Engineering, Inc. should not rely upon this Limited Geotechnical Investigation.

2.0 PROPOSED CONSTRUCTION/PROJECT DESCRIPTION

Based on information provided by the Client, we understand that development of the rear (northeast) portion of the subject property will consist of three (3) new single story buildings for a total of eleven (11) lodging units for seasonal stay lodging. Each building will contain six, three and two lodging units, respectively. Other proposed improvements include paved parking and drive areas and other related improvements. An existing shed and several trees will be removed prior to constructing the new development.

No detailed grading plans were provided to Geodyne Engineering, Inc. at the time of our preparation of this report; however, grading is anticipated to include cuts and fills of less than 5 feet across the subject property (exclusive of remedial grading). No foundation plans were provided to Geodyne Engineering, Inc. at the time of report preparation; however, foundation loads are assumed to be typical for the type of construction. No other information is known at this time.

3.0 SCOPE OF SERVICES

The scope of our services included:

- A review of readily available data pertinent to the subject property, including published and unpublished geologic reports/maps, and soils data for the area (**References**).
- Conducting a geotechnical reconnaissance of the subject property and nearby vicinity.
- Excavation and logging of four (4) exploratory test pits (designated TP-1 through TP-4) by means of a backhoe excavator in readily accessible areas of the subject property to depths ranging from approximately 2.5 feet to 7.5 feet below the existing ground surface (bgs) (**Appendix B**).
- An evaluation of seismicity and geologic hazards to include an evaluation of faulting and liquefaction/seismic induced settlement potential.
- Completion of laboratory testing of representative earth materials encountered onsite to ascertain their pertinent soils engineering properties, including corrosion potential assessment (**Appendix C**).
- The preparation of this report which presents our preliminary findings, conclusions, and recommendations.

The Scope of Services does not include an environmental assessment of the presence or absence of wetlands and/or hazardous or toxic materials in the soil, surface water, groundwater, or air, in the proximity of this site. Any statements in this report or on the boring logs regarding odors, colors or unusual or suspicious items or conditions are strictly for the information of the client.

4.0 BACKGROUND

Subject Property Description

Based on information provided by Client (WSCS Designs – Site Plans, 2019), we understand that the proposed seasonal lodging development is associated with the address of 25840 Idyllwild Road (State Route 243), approximately 450 feet southeast of Jameson Drive, in the unincorporated community of Idyllwild, Riverside County, California. The subject property can also be identified as (APN) 563250028. Based on our review of online aerial photography (GoogleEarth®, 2018) and our site reconnaissance, the irregularly-shaped subject property is currently partially developed. The subject property covers an area of approximately 1.45-acres and can be divided

into two parcels or sections. The southwestern portion of the subject property covers an area of slightly less than 1-acre and is currently developed with three (3) single floor buildings and paved parking and drive areas comprising an old motel business that has been converted into apartments. The proposed seasonal lodging development is located on the northeastern portion of the subject property covering an area of approximately 0.5-acre, located approximately 200 feet northeast of Idyllwild Road. The subject parcel to be developed has generally bare ground that has been cleared with minor leveling for use as a parking area. This portion of the property also has an existing storage shed and scattered large evergreen trees with sparse forest cover. The surface gradient of the subject property appears to be less than 5:1 (horizontal to vertical). The site is accessible via a driveway from Idyllwild Road and Oakwood Street.

The center of the subject property is approximately situated at 33.747119° degrees North latitude and 116.711249° degrees West longitude (Google Earth®, 2018). The property is bounded by Idyllwild Road (State Route 243) on the west, the Idyllwild Bible Church (54400 Pine Crest Avenue) with residential land and open space forest to the north and east.

Topography

The subject property is located within the Idyllwild, California 7.5 minute Quadrangle (USGS, 2018). The ground surface at the property (within the subject northeast parcel) gently slopes from northeast to southwest and ranges in elevation from roughly 5,490 feet to 5,465 feet above mean sea level (amsl). Surface drainage across the site would be by sheetflow, generally to the southwest. Including the existing apartment development within the subject property, the lowest elevation is in the southwest corner of the overall property along Idyllwild Road at an elevation of approximately 5,430 feet above mean sea level (amsl).

4.1 SUBSURFACE SOIL AND GEOLOGIC CONDITIONS

The subject site is located in the central portion of the Peninsular Ranges Physiographic Province of California. A series of ranges is separated by northwest trending valleys, subparallel to faults branching from the San Andreas Fault. The trend of topography is similar to the Coast Ranges, but the geology is more like the Sierra Nevada, with granitic rock intruding the older metamorphic rocks. The Peninsular Ranges extend into Lower California and are bound on the east by the Colorado Desert. The Los Angeles Basin and the island group (Santa Catalina, Santa Barbara, and the distinctly terraced San Clemente and San Nicolas islands), together with the surrounding continental shelf (cut by deep submarine fault troughs), are included in this province.

Review of the available references (USGS, 1999), indicate that the site is located in an area underlain by Older (Pleistocene, younger than 500,000 years), marine and non-marine terrace

deposits (Qomt2-6sa), above marine wave-cut platform; moderately consolidated sediments. This unit is described as consisting of mostly of silty sand with clay and gravel. These marine and non – marine terrace deposits are underlain by the Capistrano formation. This formation consists mainly of siltstone, claystone and mudstone, typically expansive in nature and slide prone.

A description of the artificial fill and formational soils underlying this site located on a cut/fill lot is presented below from youngest to oldest:

Artificial Fill: Our exploratory excavations indicated the presence of artificial fill to the full depth explored (i.e., 6.0 feet). This material was noted to consist of silty sands and contains locally derived bedrock fragment fill. Based on review of available information, the fill is documented and ranges from 3 ft in thickness within the building pad area (lot cap) and increases to 15 ft within the rear slope as stabilized buttress fill.

Formation: Formational soils was not encountered in our exploratory excavations. Based on available information, this formation consists mainly of siltstone, claystone and mudstone, typically expansive in nature and slide prone.

GROUND WATER

Review of the available references (DWR, 2019), indicate that there are not groundwater wells in the general vicinity of the site and in a mile radius circle around the site.

It should also be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. Based on the permeability characteristics of the soil and the anticipated usage and development, it is our opinion that any seepage problems, which may occur, will be minor in extent. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

5.0 FIELD EXPLORATION AND LABORATORY TESTING

Field work for our Limited Geotechnical Investigation was conducted on November 1 and 5, 2019. Our initial field work on November 1, 2019 at the site consisted of attempting the excavation of hand-dug exploratory test pits at the subject property. Due to difficult hand digging conditions and shallow refusal, field work was terminated and, with the concurrence of the Client, it was decided to use a backhoe at a later date to excavate the test pits. On November 5, 2019, a total of four (4) exploratory test pits were excavated at the subject property. The test pit depths ranged from

approximately 2.5 feet to 7.5 feet below the existing ground surface (bgs) and were logged under the supervision of a Registered Professional Engineer and Certified Engineering Geologist at Geodyne Engineering, Inc. The approximate locations of the test pits are shown in **Appendix A**.

A small CAT, track backhoe excavator was used to excavate the exploratory test pits, designated TP-1 through TP-4. Equipment refusal was encountered within test pit excavations due to the presence of hard, granitic bedrock at depth. The subsurface lithology is recorded on the test pit logs, which are presented in **Appendix B**. The soils were classified in accordance with the Unified Soil Classification System (ASTM, 2015). Representative bulk samples were also collected for appropriate laboratory testing. Attempts were made to retrieve undisturbed ring samples. However, the samples were either disturbed or there was no recovery due to the friable nature of the underlying soil and bedrock.

5.1 Laboratory Testing Program

In addition to field exploration, a supplemental laboratory testing program was conducted to determine additional pertinent engineering characteristics of the subsurface materials that are necessary to evaluate the soil parameters. These tests include:

- 1) Moisture Content (ASTM D 2216)
- 2) Grain Size Distribution (ASTM D 422)
- 3) Corrosion Potential CT-417, CT-422, CT-532(643)
- 4) Maximum Density & Optimum Moisture (ASTM D 1557)
- 5) Direct Shear Tests (ASTM D3080 / D3080M)
- 6) Consolidation –Shear Test (ASTM 2435)

5.2 Corrosion Potential Screening Assessment

A near surface soil sample was tested to measure pH, soluble sulfate, soluble chloride and resistivity of the soil. The results are presented on Table No. 1.

Table No.1

Sample Location/ Depth, (ft)	pH	Soluble Sulfate (PPM)	Soluble Chloride (PPM)	Soil Resistivity (Ohm-cm)
TP-3/3.0'-7.0'	7.1	131	54	31,000

Sulfate Content

A representative near-surface soil sample was tested during our investigation for soluble sulfate content. The result of this test indicates a soluble sulfate content of (0.0131) percent by weight or negligible sulfate exposure. As such, the soils exposed are not expected to pose a potential sulfate reaction with concrete.

Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's pH level, electrical resistivity, and chloride content. In general, soil having a minimum resistivity less than 2,000 ohm-cm is considered corrosive. Soil with a chloride content of 500 ppm or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during our investigation to determine soil resistivity, chloride content, and pH level. The soil resistivity measurement of the sample was approximately (31,000) ohm-cm, chloride content of approximately (54) ppm, and the pH level of approximately (7.1). The results indicate that the soil at the site is considered mildly corrosive to ferrous metals. However, a standard corrosion potential protection measure (applying protective wrapping, tape, sealant, paint, protective lubricants etc.) is advisable to be considered in the design phase. It should be noted that Geodyne does not practice corrosion engineering. For any specific design or further assessment, a corrosion specialist should be consulted.

Concrete

Laboratory test indicated that the subject site contains soil sulfate content in the negligible range (i.e., less than 1000 part per million). However for convenience, it is recommended that concrete for all construction at the site utilize a wide and ready, commercially available Type-II Portland cement with a maximum 0.50 water/cement ratio and should comply with all of the requirements of current Code. The minimum compressive strength of concrete shall be 3000 psi at 28 days and maximum slump during placement shall be five inches. The minimum concrete cover should be 1.5-inches. Final selection of the appropriate concrete design should be made by the project structural engineer based on the applicable local laws and ordinances, and desired level of conservatism.

5.3 GENERAL SUBSURFACE CONDITIONS

Geologic Setting

Regionally, the subject property lies within the Peninsular Ranges Geomorphic Province of southern California. This province consists of a series of ranges separated by northwest trending valleys; sub parallel to branches of the San Andreas Fault (CGS, 2002). The Peninsular Ranges geomorphic province, one of the largest geomorphic units in western North America, extends from the Transverse Ranges geomorphic province and the Los Angeles Basin, south to Baja California. It is bound on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks (CGS, 2002). Major fault zones and subordinate fault zones found in the Peninsular Ranges Province typically trend in a northwest-southeast direction. Locally, the subject property is located on the southwest flank of the San Jacinto Mountains.

Regional geologic maps of the subject property and vicinity (Dibblee and Minch, 2008) indicate the property is underlain by older surficial sediments mapped as Pleistocene-age alluvial fan gravel (map unit Qof), which is described as weakly indurated and dissected. Map unit Qof was not encountered within our exploratory test pits at the site. However, the subject site is located near a geologic contact with Cretaceous-age plutonic rocks which compose the Peninsular Ranges Batholith. These plutonic rocks were encountered within our test pits and is described as Cretaceous-age quartz diorite or granodiorite (i.e. “granitic”) that is grayish-white in color and massive to slightly gneissoid with minor xenoliths.

Subsurface Conditions

The materials encountered in our test pits consisted of undocumented artificial fill and weathered (“decomposed”) granitic bedrock. A brief description of the subsurface conditions is provided in the following section. Detailed descriptions of the subsurface conditions are provided on the test pit logs included in **Appendix B**.

Artificial Fill - Artificial fill was encountered in three (3) of our test pits at the ground surface and extended to depths ranging from 2 to 3 feet below the existing ground surface in test pits TP-1, TP-2 and TP-3. Artificial fill was not encountered within test pit TP-4. As encountered, the artificial fill consists of light grayish-brown or brownish-gray, dry to damp, medium dense, fine to coarse grained sand with pebbles and minor silt with tree roots up to 3-inches in diameter at the time of our subsurface exploration. The artificial fill was likely placed in conjunction with the adjacent motel/apartment development during minor site grading and re-leveling into a parking area.

Decomposed Granite (“DG”) – Decomposed granite bedrock (appears to be weathered granodiorite) was encountered beneath the existing artificial fill within test pits TP-1, TP-2 and TP-3 and was encountered at the ground-surface in test pit TP-4, and extended to the maximum explored depth of 7.5 feet below the existing ground surface. The decomposed granite materials at the time of our subsurface exploration can be described as tan or light brown to grayish-brown, highly weathered, friable with medium to coarse texture and moderately hard to hard becoming hard and relatively unweathered at approximate depth below 7 feet.

Groundwater

Static groundwater or seepage was not encountered in any of our exploratory test pits to a depth of 7.5 feet bgs at the time of exploration. Based on the presence of shallow, hard granitic bedrock at the ground surface to very shallow depths of 3 feet, static groundwater is not anticipated beneath the site. It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation, and other factors that may not have been evident at the time of our subsurface exploration.

6.0 GEOLOGIC HAZARDS

California Building Code Seismic Design Parameters

Geodyne Engineering, Inc. utilized seismic design criteria provided in the CBC (2016) and ASCE 7-10. Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism. The site coefficients and adjusted maximum considered earthquake spectral response accelerations in accordance with the 2016 California Building Code are presented in **Table 2**.

TABLE 2 2016 CBC Seismic Parameters and Peak Ground Acceleration	
Parameter	Value
Site Coordinates	Latitude 33.747119° Longitude -116.711249°
Mapped Spectral Acceleration Value at Short Period: S_s	1.5g
Mapped Spectral Acceleration Value at 1-Second Period: S_1	0.6g
Site Classification	B
Short Period Site Coefficient: F_a	1
1-Second Period Site Coefficient: F_v	1
Site-Modified Spectral Acceleration Value: S_{MS}	1.5g

Site-Modified Spectral Acceleration Value: S_{M1}	0.6g
Design Spectral Response Acceleration at Short Periods: S_{Ds}	1g
Design Spectral Response Acceleration at 1-Second Period: S_{D1}	0.4g
Peak Ground Acceleration adjusted for Site Class Effects: PGAM	0.504g

Faulting and Surface Rupture

The subject property is located within an area of California known to contain a number of active and potentially active faults. There are no known active faults crossing the property (Jennings and Bryant, 2010) and the property is not within a State of California Earthquake Fault Zone (CDMG, 1974). The closest known active fault is the San Jacinto Valley segment of the San Jacinto Fault Zone, located approximately 7.41 miles (11.93 km) southwest of the subject property (USGS, 2008). Therefore, the potential for surface rupture at the property is considered low. Three of the closest faults along with their distance from the property and Maximum Magnitude are shown in **Table 3**.

TABLE 3 Nearby Active Faults		
Fault	Distance in Miles (Kilometers)¹	Maximum Magnitude¹
San Jacinto SJV+A+CC+B	7.41 (11.93)	7.70
San Jacinto SBV+SJV+A+CC	7.41 (11.93)	7.76
San Jacinto SJV+A	7.41 (11.93)	7.47

1. USGS Online Fault Search (2008)

Landslides and Slope Stability

The subject property and surrounding areas are generally relatively flat with near-surface or surface exposures of granitic bedrock. As a result, we consider the potential for landslides or slope instabilities to occur at the property to be negligible.

Liquefaction Potential and Lateral Spreading

Liquefaction occurs when loose, saturated, generally cohesionless fine sands and non-plastic silts are subjected to strong ground shaking. The soils lose shear strength and become liquid; potentially resulting in large total and differential ground surface settlements as well as possible lateral spreading during an earthquake. Seismically induced settlement can occur in response to

liquefaction of saturated loose granular soils, as well as the reorientation of soil particles during strong shaking of loose, unsaturated sands.

Due to the lack of shallow groundwater at the subject site, and the presence of near-surface to surface granitic bedrock, the probability of liquefaction, seismically induced settlement, and lateral spreading at the subject site is considered to be negligible.

Tsunamis, Flooding and Seiches

The subject property is not located within a Tsunami Evacuation Area; therefore, damage due to tsunamis is considered low.

Geodyne reviewed the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) online database (FEMA, 2019) to determine if the subject property was in a flood zone. According to Panel No. 06065C2155G - effective August 28, 2008, the subject property is located within Flood Zone X. FEMA defines Zone X as an area of minimal flood hazard, usually depicted on FIRMs as outside the 500-year flood plain.

Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The subject property is not located immediately adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered low.

7.0 FOUNDATION RECOMMENDATIONS

7.1 Foundation on Expansive Soils

Expansive soils change in volume with change in moisture content. Shrinking and swelling of the clays can cause heaving and cracking on slab-on-grade and structures founded on shallow foundations. The results of our exploration, laboratory testing and engineering tactile evaluation indicate the soils underlying this site have a Very Low Expansion Potential characteristics. As such, special measures per 2016 California Building Code (CBC) Section 1803.5.3 are not required to mitigate expansive soil. However, we recommend the expansion index to be verified during the rough grading/earthwork of the site.

SHALLOW FOUNDATION

Foundation support for the proposed construction could be derived by utilizing a shallow conventional continuous or spread foundation system extended below frost zone or insulated, embedded within the underlying bedrock or engineered fill compacted to 90%. An ultimate bearing capacity of 6000 psf and an allowable bearing capacity of 2000 psf may be used for foundation bearing on bedrock

or compacted fill. The recommended values are for a footing 18" wide and 18 inch deep. It may be increased by 300 psf for every additional foot of width or depth (3500 psf Max). A one-third increase in bearing capacity may be used for resistance against seismic and wind loading.

The recommended passive earth pressure (equivalent fluid weight) is 250 pcf (Max. 3000 pcf). Minimum reinforcement for 18 inch deep and 18 inch wide continuous footings should consist of four No. 4 steel reinforcing bars; two placed near the top of the footing and two near the bottom.

FRICTION PILES

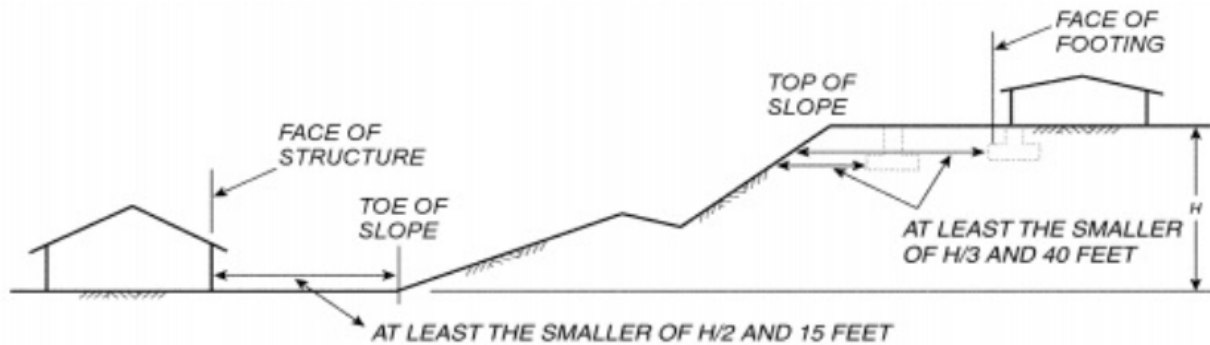
As an option, foundation support for the proposed construction could be derived by utilizing a friction pile foundation system. A minimum 24 inch diameter piers, embedded a minimum of 5 ft into the underlying bedrock should be considered. The capacity of fill should be ignored unless certified. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces. An allowable skin friction resistance of 300 psf may be used for the design of friction pile system. The piers should not be spaced more than three diameter on center each way. For piers with closer spacing, a load capacity reduction factor of 0.75 due to pile group effects may be considered.

7.2 Retaining Wall

Embedded structural walls should be designed for lateral earth pressures exerted on the walls. The magnitude of these earth pressures will depend on the amount of deformation that the wall can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the active condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized and the earth pressures will be higher. These walls such as basement walls and swimming pools should be designed for the at rest condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the passive resistance. For design purposes, the recommended equivalent fluid pressure for each case for walls constructed above the static groundwater table and backfilled with very low-expansive soils is provided below. Retaining wall backfill should be compacted to at least 95% relative compaction based on the maximum density of select import defined by ASTM D1557.

A conventional shallow 24 inch square spread or minimum 24 inch diameter pier foundation system embedded within the underling compacted fill or bedrock may be used to support the proposed retaining wall. Spread or pier foundation system should be properly reinforced, embedded a minimum of 36" below the finished sub grade elevation. Greater embedment may be necessary to resist lateral loads due to wind and seismic forces.

In cases where the ground surface slopes away from the foundation, passive resistance will be developed below an imaginary horizontal line that intersects the ground surface. As such, the requirements of 2016 CBC, Figure 1808.7.1 Foundation Clearances from Slopes should be adhered to as illustrated below:



For SI: 1 foot = 304.8 mm.

FIGURE 1808.7.1

FOUNDATION CLEARANCES FROM SLOPES

Retaining structure may be designed per the following geotechnical parameters:

- Allowable Bearing Pressure: 2000 psf

The bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3500 psf.

- Coefficient of Friction: 0.40
- Soil Unit Weight: 120 pcf
- Allowable skin friction resistance: 300 psf
- Passive Earth Pressure - equivalent fluid weight of 250 pcf (Max. 3000 pcf)

Frictional resistance and passive pressure resistance may be used in combination if friction coefficient is reduced by one-third. A one-third increase in passive pressure may be used for resistance against seismic and wind loading.

Active earth pressures behind walls depend on wall movement, backfill slope, surcharge loads and back fill material.

Table No. 4

Equivalent Fluid Density	
(PCF)	Level backfill
Active Condition	40
At-rest Condition	65

Lateral resistance parameters provided above are ultimate values. Therefore, a suitable factor of safety should be applied to these values for design purposes. The appropriate factor of safety will depend on the design condition and should be determined by the project Structural Engineer. In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls.

The backfill behind the wall should be drained properly. The simplest drainage system consists of a drain system located near the bottom of the wall. The drain collects the water that enters the backfill and this may be disposed of through outlets along the base of the wall. To insure that the drains are not clogged by fine particles, they should be surrounded by a granular filter wrapped in a geofabric such as Mirafi 140N or equivalent. In spite of a well-constructed toe drain, substantial water pressure may develop behind the wall if the backfill consists of clays or silts. A more satisfactory drainage system, consisting of a back drain of 12 inches to 24 inches width gravel may be provided behind the wall to facilitate the drainage. All back drains should be provided with outlet to suitable drainage facilities.

SEISMICALLY INDUCED LATERAL EARTH PRESSURES

Retaining wall design calculations for lateral earth pressure due to earthquake motions for walls higher than 6 feet, as required by section 1803.5.12 of the Building Code is discussed below.

We have used $\frac{1}{2}$ of $\frac{2}{3}$ the PGA_M in our analysis $(\frac{2}{6}) (0.504 g) = 0.17g$. Evaluation of lateral earth pressures under static and seismic loading conditions is based on using the Coulomb (1776) and Mononobe-Okabe (1929) Methods for frictional backfill materials with little to zero cohesion. For a level backfill, we recommend using a granular frictional soil material which exhibits friction angle 30 degrees or better. If this material is used, we recommend using combined of static and dynamic active equivalent earth pressure 55 pcf. For walls with a retained height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall

designs be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 18 pcf. Such walls that are to be designed in the static case assuming the at-rest condition should be checked seismically using this additive seismic EFP added to the active condition (i.e., the additive seismic EFP is not added to the at-rest EFP). The additive seismic EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

7.3 Slab on Grade

Slabs on grade should be underlain by a layer of four (4) inches free drainage 3/4" crushed rocks over firm native or approved structural fill. Slab thickness, reinforcement etc., should be selected by the structural engineer based on the analysis performed considering the loads anticipated, sub grade expansion potential and the modulus of subgrade reaction of the soil. As minimum, we recommend a 4.0 inch thick slab thickness, reinforced with #4 bars at 18 inch on center. The subgrade for the new slab should be prepared as recommended under Section 8.2 "Site Preparation." A vapor barrier over the crushed rock or approved subgrade soil should be considered in areas where the migration of moisture through the floor slab would be detrimental. To protect the vapor barrier (Visqueen) from punctures during placement, it is recommended that the Visqueen be placed over two-inch thick, clean sand layer. The vapor barrier should be at least 10-mil plastic (STEGO or Equivalent) and should be sealed at all splices, around plumbing, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier and care should be taken not to puncture the membrane. Some contractors exercising special care use heavier membranes or double layers of 10-mil plastic with splices staggered and sealed. All new slab design should be in compliance with the applicable sections of 2016 Cal Green.

Preliminary Pavement Design

Based on site soil conditions and laboratory test results, an R-value of 35 was assumed for the preliminary flexible asphaltic concrete pavement design shown in the table below. The R-value should be verified during grading of the pavement areas.

Table 5
ASPHALT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Asphaltic Concrete	Class II Aggregate Base*	Compacted Subgrade*
5.0 (Parking and Vehicle Drive Areas)	3.0"	4.5"	12.0"
6.0 (Heavy Truck Areas)	3.0"	7.0"	12.0"

*95% compaction based on ASTM D1557 Test Method

The following tabulated recommendations are for light-duty and heavy-duty Portland Cement Concrete pavement sections.

Table 6
PORTLAND CEMENT CONCRETE PAVEMENT THICKNESSES

Traffic Index	Portland Cement Concrete*	Class II Aggregate Base**	Compacted Subgrade**
5.0 (Light Duty)	5.0"	4.0"	12.0"
6.0 (Heavy Duty)	6.0"	6.0"	12.0"

* Minimum Compressive Strength of 4,000 psi

** 95% compaction based on ASTM D1557 Test Method

8.0 CONSTRUCTION GUIDELINES

8.1 Construction Monitoring

As Geotechnical Engineer of Record for this project, Geodyne Engineering, should be involved in monitoring the foundation installation and earthwork activities. The performance of any foundation system is not only dependent on the foundation design, but it is strongly influenced by the quality of construction. Prior to construction, please contact our office so that a Foundation and Earthwork Monitoring Plan can be incorporated into the Project Quality Control Program.

8.2 Preliminary Site Preparation Recommendations

Prior to the start of grading operations, utility lines within the project area, should be located and marked in the field so they can be rerouted or protected during site development. All debris, vegetation, tree roots, organic matters and perishable material including remains of previous structures (if any) should be removed from the site.

When excavations deeper than five feet are made, temporary construction slopes should be no steeper than 1.5:1 (horizontal to vertical). Sheeting and bracing should be provided by the contractor, as necessary, to protect workers in the excavation. Where excavations undermine existing improvements, such as the existing walls, etc., temporary structural support should be provided to reduce the risk of damage resulting from undercutting. Permanent cut and fill slopes should not be constructed steeper than 2:1 and should be considered subject to review by the geotechnical consultant at the time of grading. These slopes should possess sufficient compacted fines to limit erosion risk. If upon construction, relatively clean, cohesionless sands are encountered, reconstruction by blending in fines to compacted fill and/or flattening of slopes will be advised. Where fill is to be placed including the bottom excavation, the upper 12 inches of the surface exposed by the excavation should be scarified, moisture-conditioned up to three (3) percent over optimum moisture content and compacted to 95 percent relative compaction. If localized areas of relatively loose soil prevent proper compaction, over-excavation, and re-compaction will be necessary.

Excavated soils generated from cut operations at the site are suitable for use as general Engineered Fill in structural areas, provided they do not contain deleterious matter, organic material, or rock material larger than 3 inches in maximum dimension. All earthwork should be conducted in accordance with the applicable codes, agency requirements, the aforementioned recommendations, and the standard grading guidelines. The minimum required compaction for the building pads is 90 percent of the maximum dry density as determined by ASTM D1557, with moisture content of up to three percentage points above the optimum moisture content of the soil.

Site grading

The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet of horizontal distance, a 5-percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet of the building foundation. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building.

8.3 Temporary Drainage Measures

Temporary drainage provisions should be established to minimize water runoff into construction areas. If standing water does accumulate, it should be removed by pumping as soon as possible.

Adequate protection against sloughing of soils should be provided for workers and inspectors entering the excavations. This protection should meet Cal OSHA and other applicable building codes.

8.4 Select Structural Fill (IMPORT)

Any select structural fill used at the site for general construction use should have a Liquid Limit less than 35 and a Plasticity Index between 5 and 15. The fill should contain no particles greater than one (1) inch in diameter. The percent passing U.S. Standard Sieve No. 4 should be between 40 and 80 percent and passing Sieve No. 40 between 10 and 50 percent. The percent passing Sieve No. 200 should be less than 20 percent.

Pit-run gravels (with some clay binders) and crushed limestone (with sufficient fines to bind the aggregate together) are examples of suitable select structural fill materials. The fill materials should be placed in loose lifts not to exceed 8 inches thick and compacted to 95 percent of the maximum dry density as determined by ASTM D1557, with moisture content of up to (2) percentage points above optimum.

8.5 Groundwater Seepage

In areas where significant cuts (2-feet or more) are made to establish final grades for building pads, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. Subsurface drains may be required to intercept seasonal groundwater seepage. The need for these, or other dewatering devices, on building pads should be carefully addressed during construction. Our office could be contacted to visually inspect final pads to evaluate the need for such drains.

Groundwater seepage may occur several years after construction if the rainfall rate or drainage changes in the vicinity of the project site. If seepage runoff occurs towards the building, an engineer from our office should be called on to evaluate its' effect and determine whether French drains are required at the location.

8.6 Control Testing and Field Observation

Subgrade preparation and structural fill placement should be monitored by the project geotechnical engineer or his representative. Field-tests for moisture content and relative compaction of the fill soils shall be performed by Geodyne Engineering, Inc. Location and frequency of tests shall be at our field representative(s) discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction. Any areas not meeting the required compaction should be re-compacted and retested until compliance is met.

8.7 Temporary Excavations

Temporary slopes on the order of 1.5H to 1V may be provided for excavations. These recommendations may require modification where loose soil or ground water is encountered. The contractor is responsible for the safety of the workers and should observe the federal and local regulations including Cal-OSHA excavation and trench safety guidelines. During wet weather, runoff water should be prevented from entering the excavation.

9.0 SITE DRAINAGE AND MAINTENANCE

Final drainage is important for the performance of the proposed construction. Landscaping, plumbing, and downspout drainage is also important. It is vital that all roof drainage be transported away from the structures so that water does not pond around it, which can result in a soil volume change underneath the structures. Plumbing leaks (if any) should be repaired as soon as possible in order to minimize the magnitude of a moisture change under the slab. Large trees and shrubs should not be planted in the immediate vicinity of the structures, since root systems can cause a substantial reduction in soil volume in the vicinity of the trees during dry periods.

Adequate drainage should be provided to reduce seasonal variations in moisture content of foundation soils. All pavement and sidewalks within 10-feet of the structures should be sloped away from the structures to prevent ponding of water around the foundations. Final grades within 10-feet of the structure should be adjusted to slope away from structures preferably at a minimum slope of 5 percent. Maintaining positive surface drainage throughout the life of the structure is essential.

In areas with pavement or sidewalks adjacent to the new structure, a positive seal must be provided and maintained between the structures and the pavement or sidewalk to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and flat-work is not uncommon. Maximum grades practical should be used for paving and flatwork to prevent areas where water can pond. In addition, allowances in final grades should take into consideration post construction movement of flatwork, particularly if such movement would be critical. Normal maintenance should include inspection of all joints in paving and sidewalks, etc. as well as re-sealing where necessary.

Trench backfill for utilities should be properly placed and compacted, as outlined in this report, and in accordance with the requirements of local City, County and/or State Standards. Since granular bedding backfill is used for most utility lines, the backfilled trench should be prevented from becoming a conduit and allowing an access for surface or subsurface water to travel toward the new structures. Concrete cut-off collars or clay plugs should be provided where utility lines

cross building lines to prevent water from traveling in the trench backfill and entering beneath the structures.

Plan Review

The project geotechnical engineer should review the final plans for the proposed structures so that he may determine if changes in the foundation recommendations are required.

10.0 LIMITATIONS

Only a portion of subsurface conditions have been reviewed and evaluated. Conclusions, recommendations, and other information contained in this report are based upon the assumption that the subsurface conditions do not vary appreciably between and adjacent to the observation points. Although no significant variation is anticipated, it must be recognized that variations can occur. This report has been prepared for the sole use and benefit of our client. The intent of the report is to advise our client on geotechnical matters involving the proposed improvements. It should be understood that the geotechnical consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report and/or any other geotechnical aspect of the project should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive the advice. Subsequent use of this report can only be authorized by the client. Any transferring of information or other-directed use by the client should be considered "advice by the client."

Geotechnical engineering is characterized by uncertainty. Geotechnical engineering is often described as an inexact science or art. Conclusions and recommendations presented herein are partly based upon the evaluations of technical information gathered, partly on experience, and partly on professional judgment. The conclusions and recommendations presented should be considered "advice." Other consultants could arrive at different conclusions and recommendations. Typically, "minimum" recommendations have been presented. Although some risk will always remain, lower risk of future problems would usually result if more restrictive criteria were adopted. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project.

REFERENCES

American Society of Civil Engineers (ASCE), 2010, Minimum Design Loads for Buildings and Other Structures, ASCE Document ASCE/SEI 7-10.

American Society for Testing and Materials (ASTM), 2015, Annual Book of ASTM Standards, Volume 04.08, Construction: Soil and Rock (I), Standards D 420 - D 5876.

California Building Code (CBC), 2016, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, California Building Standards Commission, Based on 2015 International Building Code; 2016 California Historical Building Code, Title 24, Part 8; and 2013 California Existing Building Code, Title 24, Part 10, effective January 1, 2017.

California Division of Mines and Geology (CDMG), 1974, State of California, Special Studies Zones, NW ¼ Idyllwild Quadrangle, Official Map – Effective: July 1, 1974.

California Geological Survey (CGS), 2002, California Geomorphic Provinces Note 36, Electronic Copy, Revised December 2002.

Dibblee, T.W. and Minch, J.A., 2008, Geologic Map of the Hemet & Idyllwild 15-Minute Quadrangles, Riverside County, California, Dibblee Geological Foundation Map DF-371, scale – 1:62,500.

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map 06065C2155G, Riverside County Unincorporated Areas, California, dated August 28, 2008.

GeoTracker Website, 2019, State Water Resources Control Board, website address - <http://geotracker.waterboards.ca.gov/>, accessed November 2019.

GoogleEarth, 2018, Version 7.1.8.3036.

Hart, E.W., and Bryant, W.A. (Hart and Bryant), 1997, Fault-Rupture Hazard Zones in California: California Department of Conservation, Division of Mines and Geology, Special Publication 42.

Jennings, C.W., and Bryant, W.A., (Jennings and Bryant) 2010, Fault Activity Map of California and Adjacent Areas: California Geologic Survey, Map Sheet No. 6, scale 1:750,000.

Riverside County Integrated Project – Planning Home – TMLA website, http://planning.rctlma.org/Portals/0/genplan/content/gp/chapter06.html#TOC3_6, accessed November 2019.

United States Geological Survey (USGS), 2018, 7.5-Minute Topographic Map, Idyllwild, California Quadrangle, scale 1:24,000.

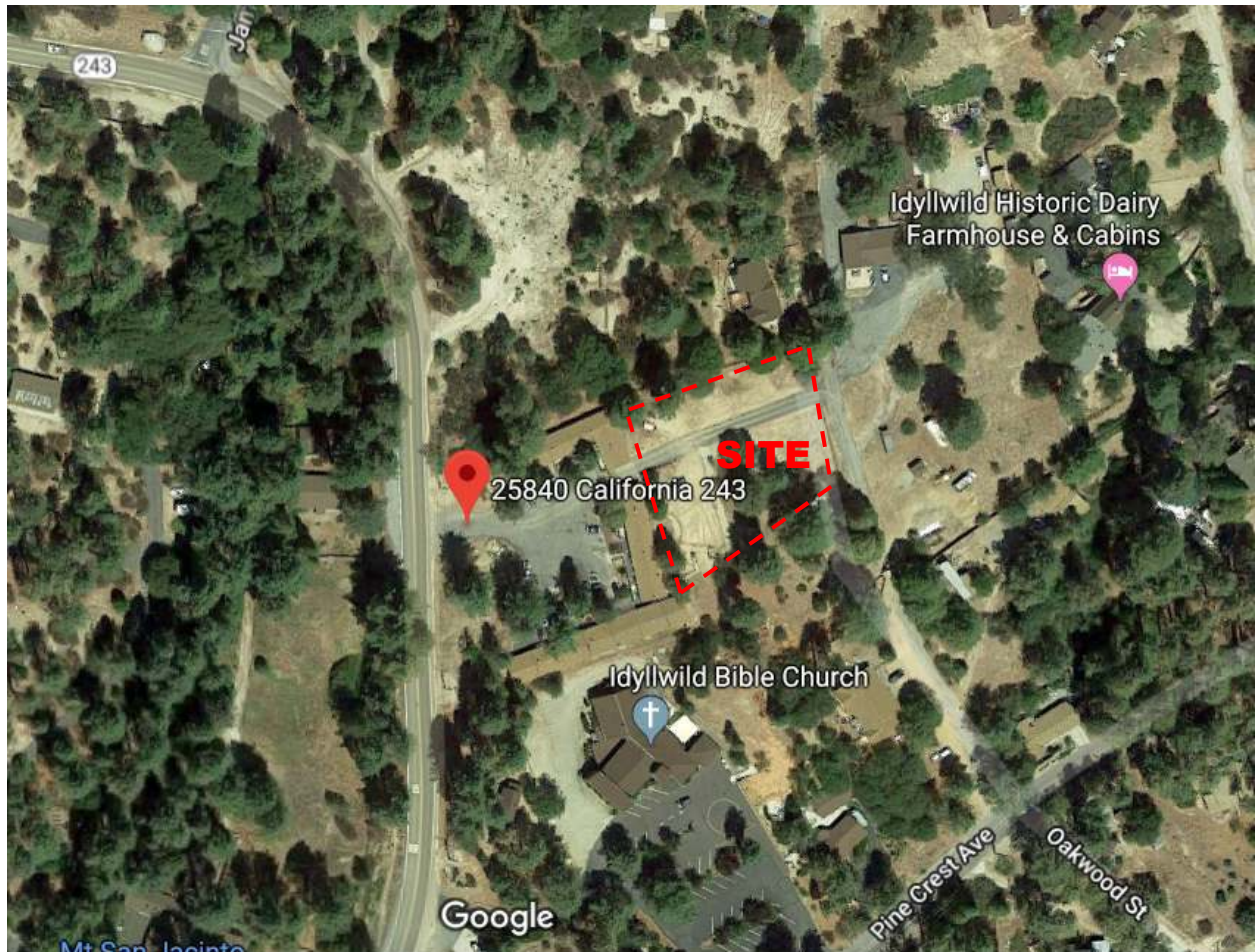
United States Geological Survey (USGS), 2008, 2008 National Seismic Hazard Maps – Online Fault Database Search, web address < <http://earthquake.usgs.gov/hazards/products/conterminous/>>, accessed November 2019.

ATC-127 - Applied Technology Councils/USGS Site Specific Seismic Hazard information <https://hazards.atcouncil.org/#/seismic>

WSCS Designs, 2019, Design Review for: Idyllwild Stewart Extended Stay Lodges, 25840 Idyllwild Road, Idyllwild, CA 92549, Project 19-324, dated September 12, 2019.

NAVFAC DM 7.1 Soil Mechanics, U.S. Department of the Navy 1982 Edition, May 1982
NAVFAC DM 7.2, Foundation and Earth Structures, U.S. Department of the Navy 1984

APPENDIX A



North
NTS

Limited Geotechnical Investigation
Proposed Extended Stay Lodges at:
25840 Idyllwild Road, Idyllwild, Riverside County, CA

GEODYNE ENGINEERING, INC.

Vicinity Map

Project No. GD19G508

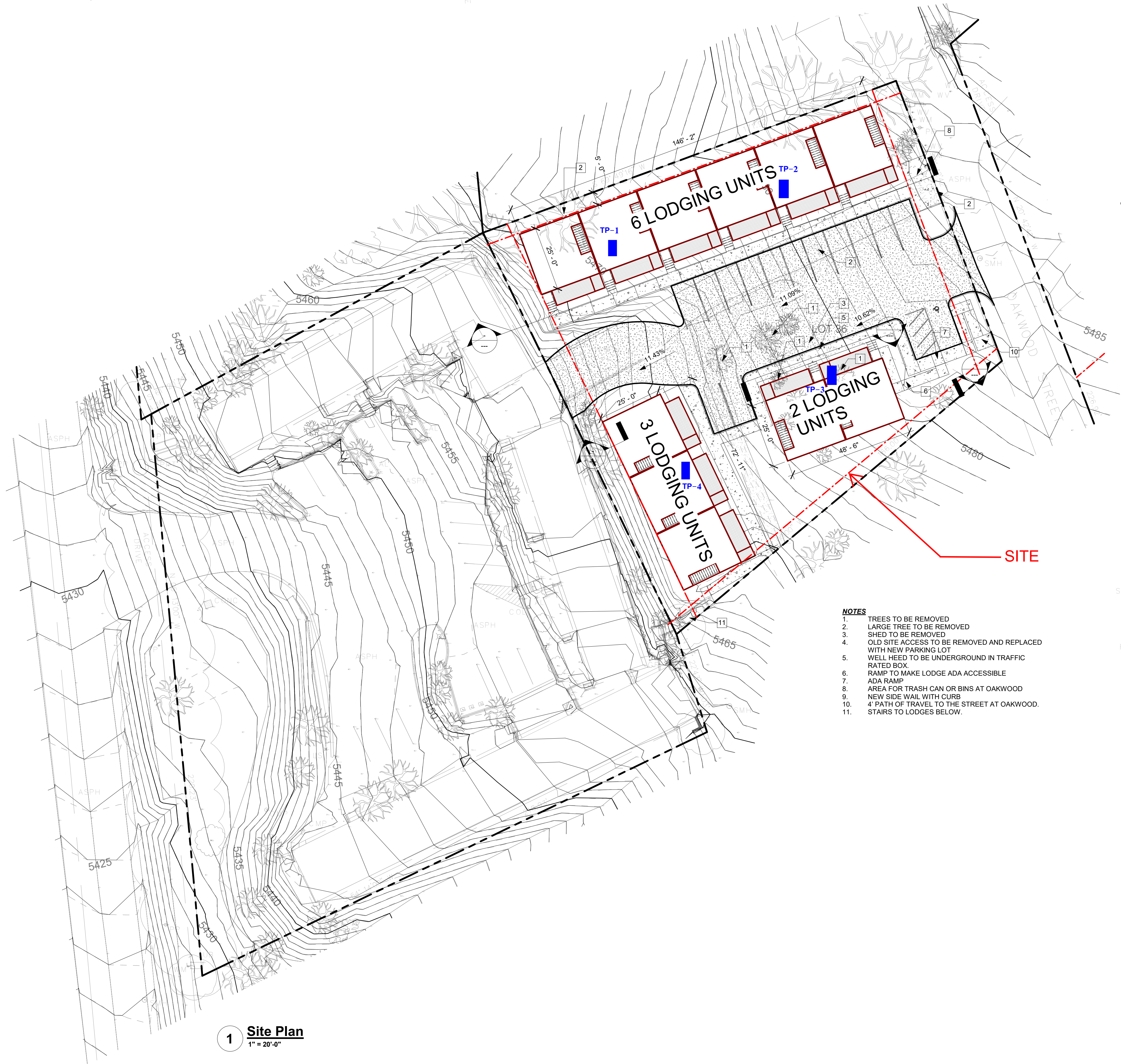
Figure: A

Design Review For:

Idyllwild Stewart Extended Stay Lodges

25840 Idyllwild Rd. Idyllwild,
CA.92549

Revision		
Number	Description	Date



- NOTES**
1. TREES TO BE REMOVED
 2. LARGE TREE TO BE REMOVED
 3. SHED TO BE REMOVED
 4. OLD SITE ACCESS TO BE REMOVED AND REPLACED WITH NEW PARKING LOT
 5. WELL NEED TO BE UNDERGROUND IN TRAFFIC RATED BOX
 6. RAMP TO MAKE LODGE ADA ACCESSIBLE
 7. ADA RAMP
 8. AREA FOR TRASH CAN OR BINS AT OAKWOOD
 9. NEW SIDE WALK WITH CURB
 10. 4' PATH OF TRAVEL TO THE STREET AT OAKWOOD
 11. STAIRS TO LODGES BELOW

1 **Site Plan**
1" = 20'-0"

- Legend:**
- TP-4
- Approximate Test Pit Locations

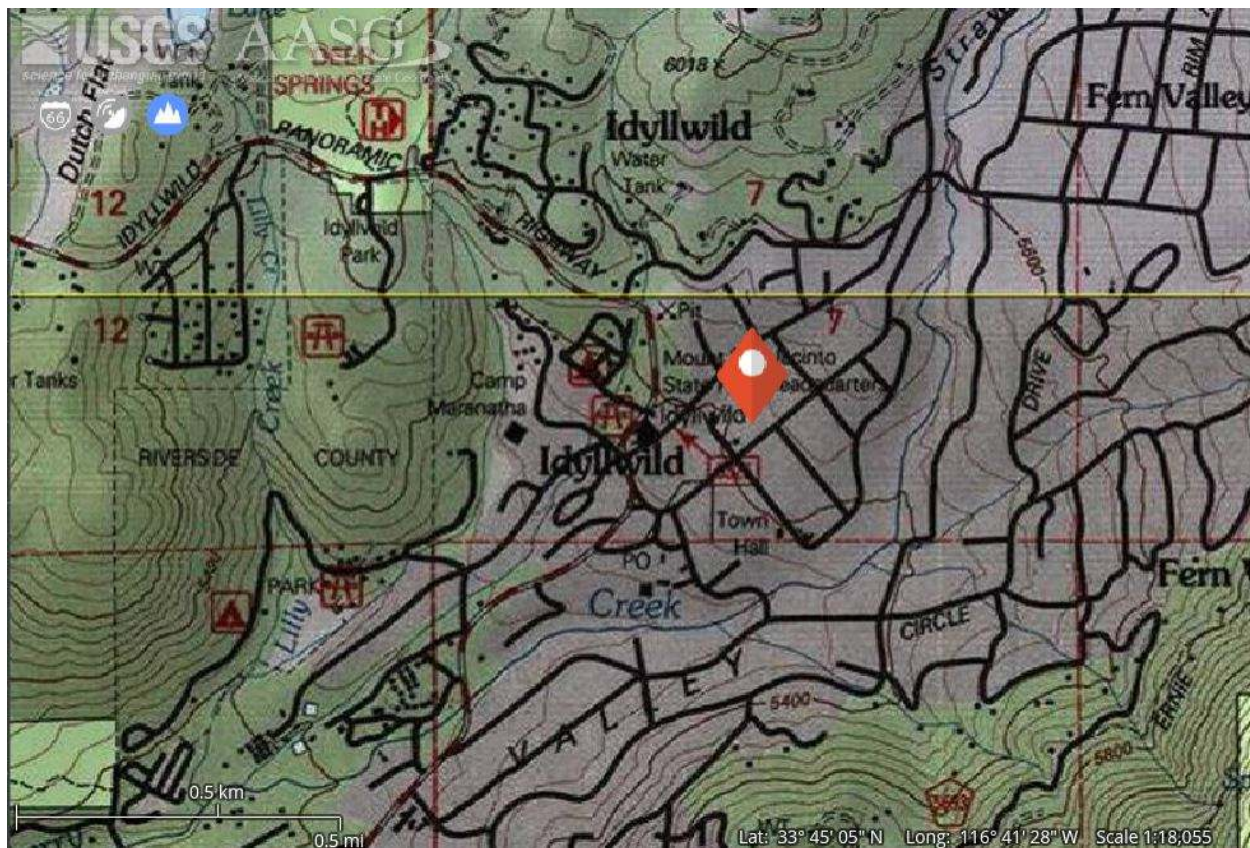
GEODYNE ENGINEERING, INC.	
Approximate Test Pit Locations Plan	
Project No. GD19G508	Figure: B

Design Review For:
**Idyllwild Stewart
Extended Stay Lodges**

19-324
9-12-2019
**25840 Idyllwild Rd. Idyllwild,
CA.92549**

DR100

**Site Plan
Concept**



NORTH



Limited Geotechnical Investigation
Proposed Extended Stay Lodges at:
25840 Idyllwild Road, Idyllwild, Riverside County, CA

GEODYNE ENGINEERING, INC.

USGS Topographic Map

Project No. GD19G508

Figure: C

APPENDIX B

Unified Soil Classification System

Major Divisions			Letter	Symbol	Description
Coarse-grained Soils More than ½ retained on the No. 200 Sieve	Gravels More than ½ coarse fraction retained on the No. 4 sieve	Clean Gravels	GW		Well-graded gravels and gravel-sand mixtures, little or no fines.
			GP		Poorly-graded gravels and gravel-sand mixtures, little or no fines.
		Gravels With Fines	GM		Silty gravels, gravel-sand-silt mixtures.
			GC		Clayey gravels, gravel-sand-clay mixtures.
	Sands More than ½ passing through the No. 4 sieve	Clean Sands	SW		Well-graded sands and gravelly sands, little or no fines.
			SP		Poorly-graded sands and gravelly sands, little or no fines.
		Sands With Fines	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sandy-clay mixtures.
Fine-grained Soils More than ½ passing through the No. 200 Sieve	Silts and Clays Liquid Limit less than 50%		ML		Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL		Organic clays of medium to high plasticity.
	Silts and Clays Liquid Limit greater than 50%		MH		Inorganic silts, micaceous or diatomaceous fines sands or silts, elastic silts.
			CH		Inorganic clays of high plasticity, fat clays.
			OH		Organic clays of medium to high plasticity.
Highly Organic Soils		PT		Peat, muck, and other highly organic soils.	
Consistency Classification					
Granular Soils			Cohesive Soils		
Description - Blows Per Foot (Corrected)			Description - Blows Per Foot (Corrected)		
	<u>MCS</u>	<u>SPT</u>		<u>MCS</u>	<u>SPT</u>
Very loose	<5	<4	Very soft	<3	<2
Loose	5 - 15	4 - 10	Soft	3 - 5	2 - 4
Medium dense	16 - 40	11 - 30	Firm	6 - 10	5 - 8
Dense	41 - 65	31 - 50	Stiff	11 - 20	9 - 15
Very dense	>65	>50	Very Stiff	21 - 40	16 - 30
			Hard	>40	>30
MCS = Modified California Sampler			SPT = Standard Penetration Test Sampler		

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lodges-Idyllwild**
 Date: 11/5/2019

TP No: 1
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavator
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5	SW			Bareground, <u>0.0'-3.0': Artificial FILL (Qaf)</u> Light grayish brown fine-coarse SAND, slightly silty, with pebbles, medium dense, dry-damp, 3/4 inch tree roots,						No ring sample recovery 
				<u>3.0'-6.5': Decompsed granite (DG)</u> Tan-light brown, coarse texture, highly weathered, friable to touch, at 5.5' becomes mottled, white, black dots,						
				End of Test Pit @ 6.5-feet No groundwater or caving Excavation backfilled with cutting spoil,						
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lodges-Idyllwild**
 Date: 11/5/2019

TP No: 2
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavator
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5	SW		Bareground, <u>0.0'-2.0': Artificial FILL (Qaf)</u> Light grayish brown SAND, slightly silty fine-coarse, medium dense, dry-damp,						No ring sample recovery 	
			<u>2.0'-7.0': Decomposed granite (DG)</u> Tan-light brown, fine-coarse texture, friable, highly weathered,							
		End of Test Pit @ 7.0-feet No groundwater or caving Excavation backfilled with cutting spoil, 								

Bareground,
0.0'-2.0': Artificial FILL (Qaf)
 Light grayish brown SAND, slightly silty
 fine-coarse, medium dense, dry-damp,
2.0'-7.0': Decomposed granite (DG)
 Tan-light brown, fine-coarse texture, friable,
 highly weathered,

End of Test Pit @ 7.0-feet
No groundwater or caving
Excavation backfilled with cutting spoil,



LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lodges-Idyllwild**
 Date: 11/5/2019

TP No: 3
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavator
 CA. Mod. Manual Sampling

Depth in Feet	Soil Type	Sample Type		☐ Bulk Sample ☐ Standard Split Spoon Sample ☒ Thin Wall Tube ☒ 2.5" Ring Sample ☐ Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
SOIL DESCRIPTION										
5	SW			Bareground, <u>0.0'-3.0': Artificial FILL (Qaf)</u> Brownish gray SAND, slightly silty fine-coarse, medium dense, dry-damp, pebbles, tree roots size 1"-3" in diameter						No ring sample recovery 
				<u>3.0'-7.0': Decomposed granite (DG)</u> Grayish brown, fine-coarse texture, friable, highly weathered,						
				End of Test Pit @ 7.5-feet No groundwater or caving Excavation backfilled with cutting spoil,						
10										
15										

LOG OF EXPLORATORY TEST PIT

Project No. GD19G508
 Project: **Extended Stay Lounges-Idyllwild**
 Date: 11/5/2019

TP No: 3
 Test Pit Location: See Plot Plan
 Groundwater: None
 Type: Backhoe excavator

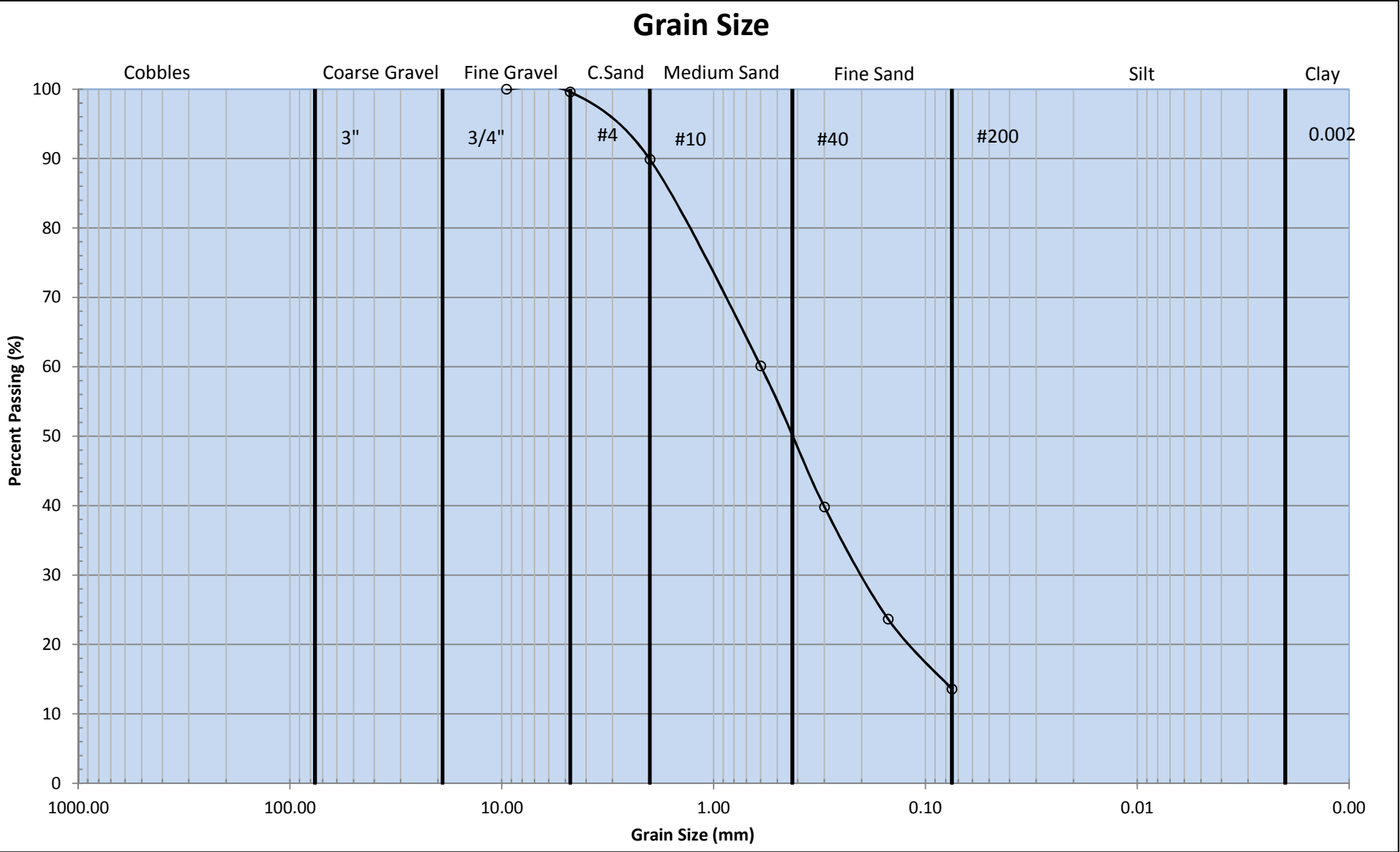
Depth in Feet	Soil Type	Sample Type		Thin Wall Tube 2.5" Ring Sample Bulk Sample Standard Split Spoon Sample Static Water Table	Blows/foot	Moisture Content, %	Dry Density, pcf	% Heave	% Collaps	Comments
		Undisturbed	Bulk							
		SOIL DESCRIPTION								
5										Bareground, <u>.0.5'-2.5': Decompsed granite (DG)</u> Tan-light brown, brittle, hard,
										End of Test Pit @ 2.5-feet No groundwater or caving Excavation backfilled with cutting spoil,
10										
										
15										



APPENDIX C

		PARTICLE SIZE ANALYSIS				
		ASTM C -136				
Project Name:	Extended Stay - Idyllwild					
Sample Type	Sand (Component Granite, Micaceous)					
		Boring No: TP-1		Depth: 2-7 '		
Date Sampled	11/8/2019		Test Performed By: HM			
Dry Weight After Wash (g):		759.3				
Sieve Size	Wet Retained	Accumulated Wet Retained	% Retained	Accumulated	Passing	
	(g)	(g)		%	%	
2 1/2"	0	0	0.00	0.00	100.00	
2 "	0	0	0.00	0.00	100.00	
1 1/2 "	0	0	0.00	0.00	100.00	
1"	0	0	0.00	0.00	100.00	
3/4"	0	0	0.00	0.00	100.00	
1/2"	0	0	0.00	0.00	100.00	
3/8"	0	0	0.00	0.00	100.00	
4	3	3	0.40	0.40	99.60	
10	73.8	76.8	9.72	10.11	89.89	
16	0	76.8	0.00	10.11	89.89	
30	225.9	302.7	29.75	39.87	60.13	
40		302.7	0.00	39.87	60.13	
50	154.1	456.8	20.30	60.16	39.84	
60	0	456.8	0.00	60.16	39.84	
100	122.7	579.5	16.16	76.32	23.68	
200	76.5	656	10.08	86.40	13.60	
PAN	103.3	759.3	13.60	100.00	0.00	
CEM LAB						

Figure D



Project Name: **Exrended Stay - Idyllwild**

Location: TP-1

Sample No: 1

Date: 11/8/2019

Job No. ST19G123

Depth: 2'-7'

By: HM

Gravel 0.0

Sand 86.4

PP-200 13.6

Total 100.0

LL= _____

PL= _____

PI= _____

Figure E

DIRECT SHEAR TEST DATA

Address: _____

File No.: GD19G508

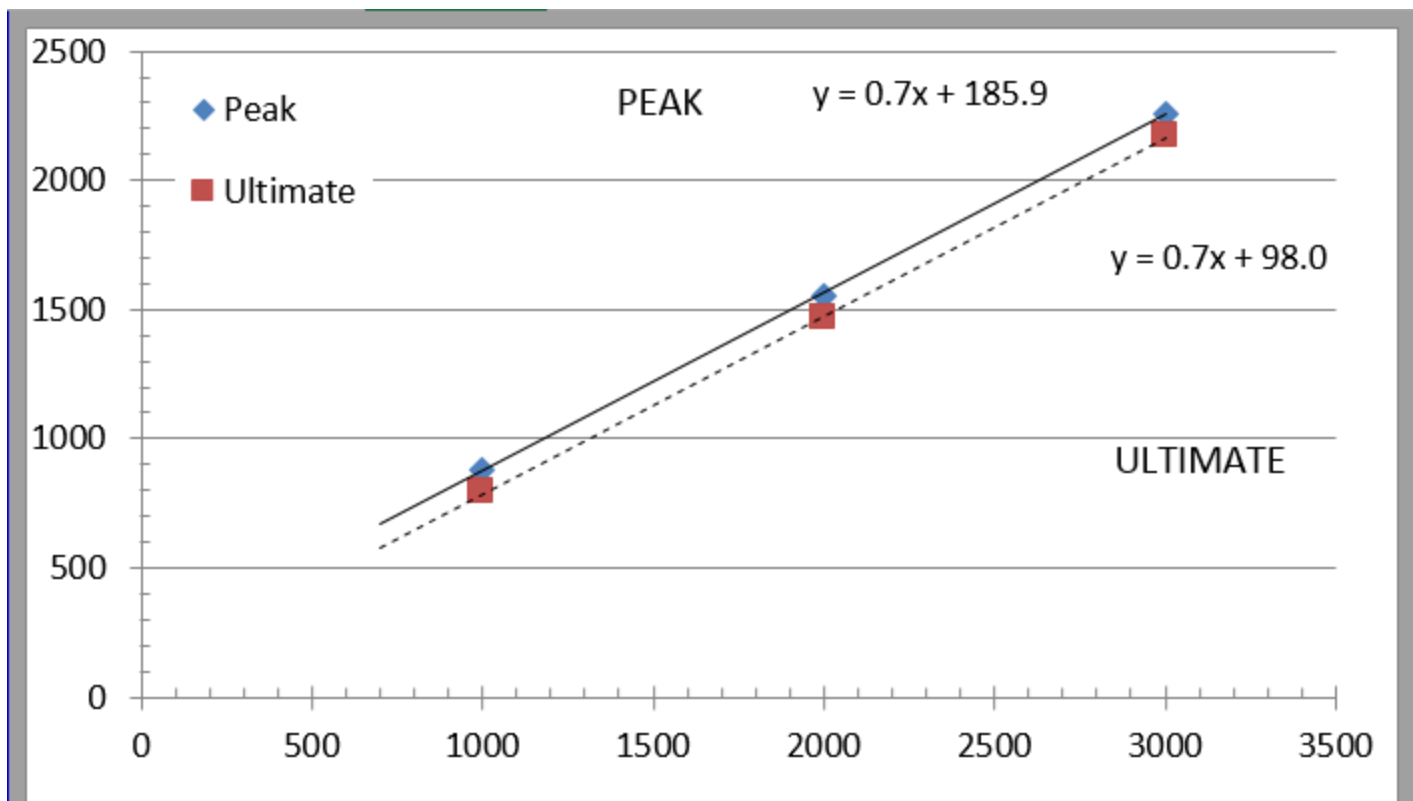
Method: Drained

Saturated: Yes

ASTM D3080

[illegible]

CEM LAB



Actual Values Being Plotted

Normal	Peak	Ultimate
1000	881	793
2000	1556	1468
3000	2261	2173

Figure G

Samples Tested	1	2	3	4
Boring ID	TP-2	TP-2	TP-2	0
Depth (in/ft.)	3.0-7.0	3.0-7.0	3.0-7.0	0
Initial Dry Density (pcf)	112.5	112.5	112.5	0
Initial Moisture Content (%)	8	8	8	0
Normal Stress (psf)	1000	2000	3000	0
Maximum Shear Stress (psf)	881	1556	2261	0
Ultimate Shear Stress (psf)	793	1468	2173	0

ASTM D3080

	Peak	Ultim.
Friction, phi (Deg)	35	35
Cohesion (psf)	186	98
Date	11/9/2019	
Method:	Drained	
Consolidation:	Yes	
Saturation:	Yes	
Strain Rate (in/min):	0.005	

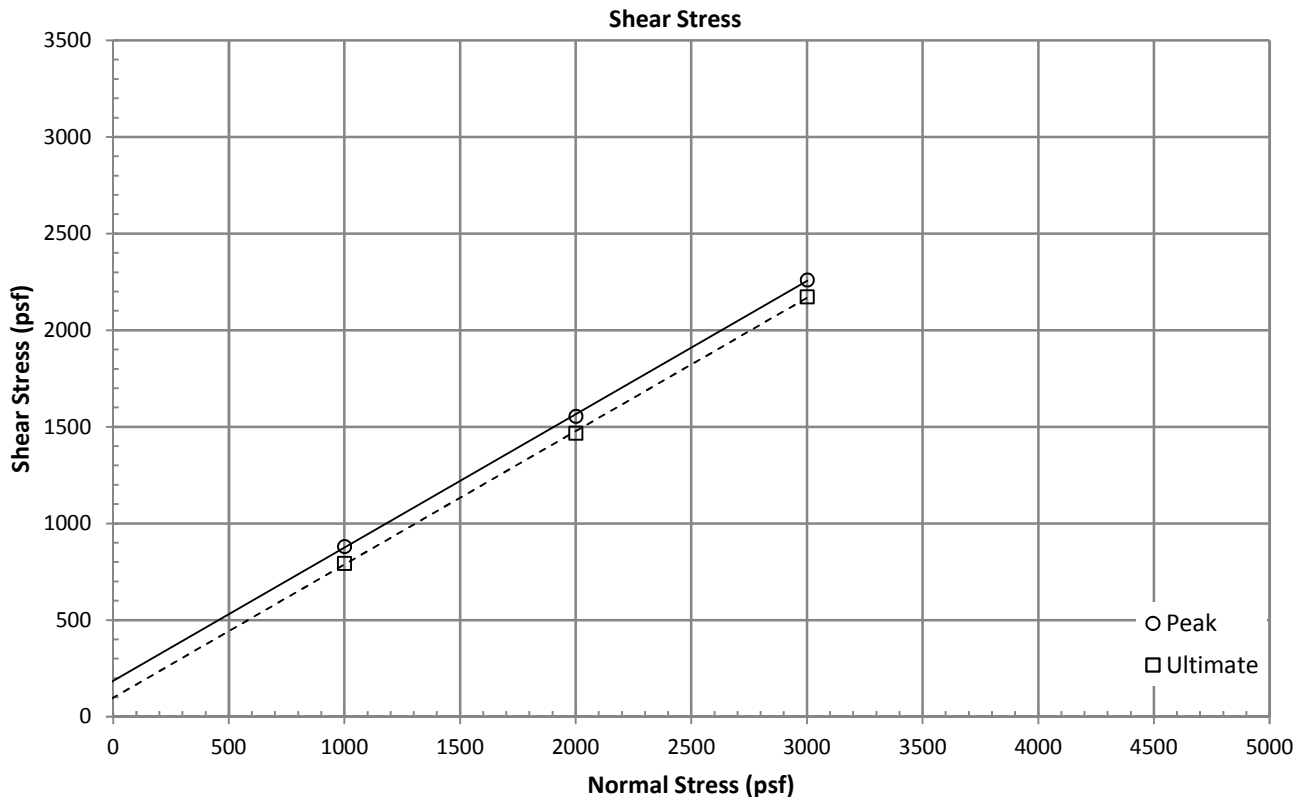
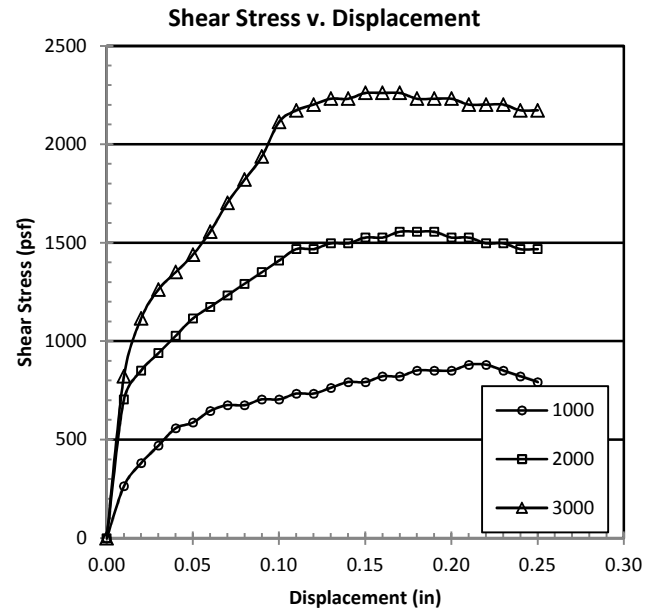
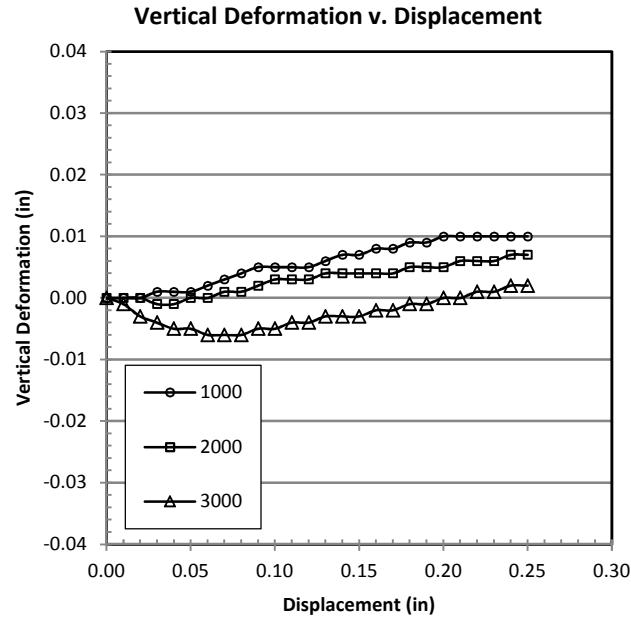


Figure H

DIRECT SHEAR TEST RESULTS

Extended Stay - Idyllwild

F.N.: GD19G508

Date: Nov 2019

Remolding Soil Sample

FORMULA		
Maximum Dry Density		125
Moisture %		8
Dry Density	90 % Of Max D.D.	112.5
Ring Volume		1.2787
$(1.2787)(D.D)(100+Moist)$		155.36

Wt. Wet Soil + Tere	g	132.4
Wt. Wet Dry + Tere	g	126.8
Wt of Tere	g	53.6
Moisture Lost	g	5.6
Dry Soil	g	73.2
Moisture	%	7.65

FORMULA		
Maximum Dry Density		125
Moisture %		7.65
Dry Density	90 % Of Max D.D.	112.5
Ring Volume		1.2787
$(1.2787)(D.D)(100+Moist)$		154.86

Total Soil For 4 Rings	g	619.4
Bowl	g	263.1
Total Mix Before Water Added	g	882.5
Total Mix For 4 Rings+ Bowl	g	884.5
Total Water added	g	2.0
Total Mix	g	884.5

P.N.

Projet : Extended Stay - Idyllwild

Excavation TP-2

Depth: 3.0'-7.0 '

Figure I

CONSOLIDATION FOR SHEAR

ASTM D-2435

Project Name: **Extended Stay - Idyllwild**
 Location: _____
 File No: **GD19G508**

Excavation: **TP-2**
 Depth: **3.0'-7.0'**

Tested By: **H-M**

Sample Type: **Remolded to 90 % of M.D.D and O.M.C.**

Ring I D 2.5 "

Boring No.	Depth (ft.)	Date	Time	Load (psf)	Water Added	Dial Read (in.)	Deform. (in.)	Consol %	Swell or Collapse
TP-2	0-2.5'	11/8/2019	9.30	1000		0.4409			
		11/8/2019	9.40			0.4400			
		11/9/2019	9.30			0.4409	0.0009	0.09	Swell
TP-2	0-2.5'	11/8/2019	9.30	2000		0.5029			
		11/8/2019	9.40			0.5021			
		11/9/2019	9.30			0.5012	-0.0009	-0.09	Collapse
TP-2	0-2.5'	11/8/2019	9.30	3000		0.5271			
		11/8/2019	9.40			0.5265			
		11/9/2019	9.30			0.5249	-0.0016	-0.16	Collapse

SAMPLE INFORMATION FOR CONSOLIDATION AND SHEAR

Description	Micaceous Silty Sand Slightly Clay		Micaceous Silty Sand Slightly Clay		Micaceous Silty Sand Slightly Clay			
Boring No.								
Depth (ft.)								
Load (psf)	1000		4000		5000			
	Before	After	Before	After	Before	After		
Ring + Wet Soil	294.79		299.01		298.66			
Ring Wt.	139.43		143.65		143.3			
Wt. of Soil	155.36		155.36		155.36			
Factor	0.002817	0.002841	0.002817	0.002836	0.002817	0.002834		
Wet Density	121.48	120.44	121.48	120.66	121.48	120.74		
Cup No.		90		26		86		
Cup + Wet Soil		177.18		174.67		175.6		
Cup + Dry Soil		148.36		147.63		149.19		
Moist. Loss		28.82		27.04		26.41		
Cup		8.67		8.76		8.47		
Dry Soil		139.69		138.87		140.72		
Moist. Cont. (%)	8.00	20.63	8.00	19.47	8.00	18.77		
Dry Density	112.48	99.84	112.48	100.99	112.48	101.66		

CEM LAB

Figure J

MAXIMUM DENSITY

ASTM D-1557

Project Name: **Extended Stay - Idyllwild**

Excavation: **TP-2**

Location: _____

Depth: **2.0'-7.0'**

Description: **Medium Brown Sand**

File No: **GD19G508**

Date: **11/6/2019**

Sieve Size **4**

Mold Size **4"**

No. of Layers **5**

% Retained **None**

Method **A**

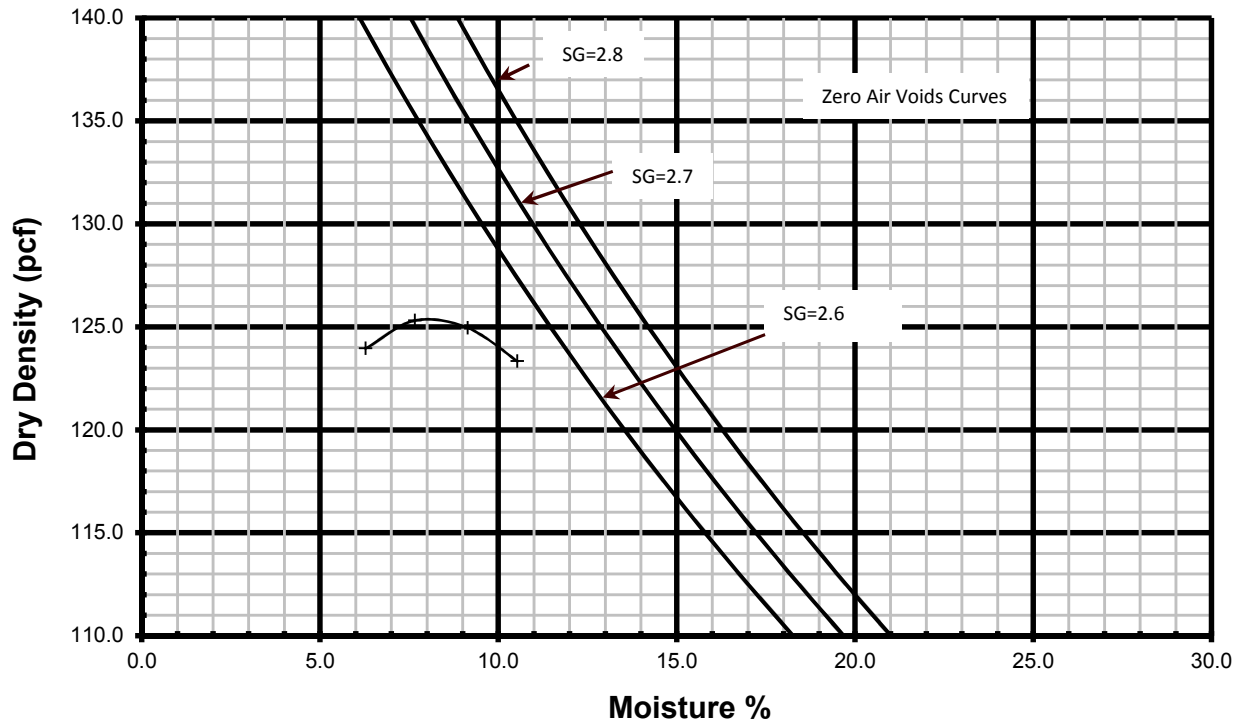
By: **H-M**

Test point number		1	2	3	4		
Wt. wet soil + mold	g	3839.4	3887.3	3909.6	3908.7		
Wt. wet soil + mold	lbs	8.46	8.56	8.61	8.61		
Wt. of mold	lbs	4.07	4.07	4.07	4.07		
Wt. wet of soil	lbs	4.39	4.49	4.54	4.54		
Wet density	pcf	131.74	134.90	136.38	136.32		
Dry density	pcf	123.95	125.31	124.96	123.33		

Moisture Determination (Oven)

Container number		27	22	18	16		
Wt. wet of soil+tare	g	284.5	348.8	288.6	276.9		
Dry wt. soil+tare	g	268.2	324.6	265.1	251.3		
Tare wt.	g	8.62	8.62	8.04	8.27		
Wt. of moisture	g	16.30	24.20	23.50	25.60		
Dry wt. of soil	g	259.58	315.98	257.06	243.03		
Moisture Content	g	6.28	7.66	9.14	10.53		

Max Density



Maximum Density **125.0** pcf

Optimum Moisture **8.0** %

ANAHEIM TEST LAB, INC

196 Technology Drive, Unit D
Irvine, CA 92618
Phone (949)336-6544

DATE: 11/06/2019

P.O. NO: Transmittal

LAB NO: C-3338

SPECIFICATION: CTM-417/422/643

MATERIAL: Soil

TP-3 @ 3'-7'

Project: Idyllwild Ext. Stay
Date Sampled: 11/05/2019

ANALYTICAL REPORT

CORROSION SERIES SUMMARY OF DATA

pH	SOLUBLE SULFATES per CT. 417 ppm	SOLUBLE CHLORIDES per CT. 422 ppm	MIN. RESISTIVITY per CT. 643 ohm-cm
7.1	131	54	31,000

RESPECTFULLY SUBMITTED


WES BRIDGER LAB MANAGER

APPENDIX D



Search Information

Address:

Coordinates: 33.747119, -116.711249

Elevation: ft

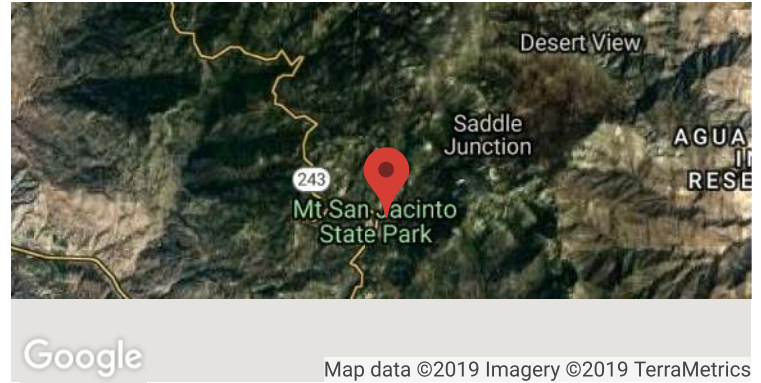
Timestamp: 2019-11-12T06:02:43.127Z

Hazard Type: Seismic

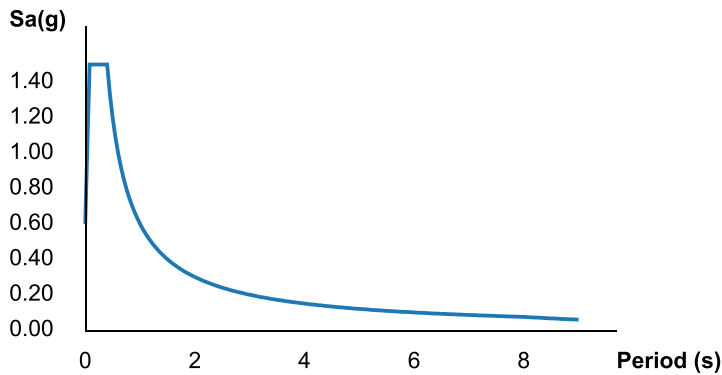
Reference Document: ASCE7-10

Risk Category: II

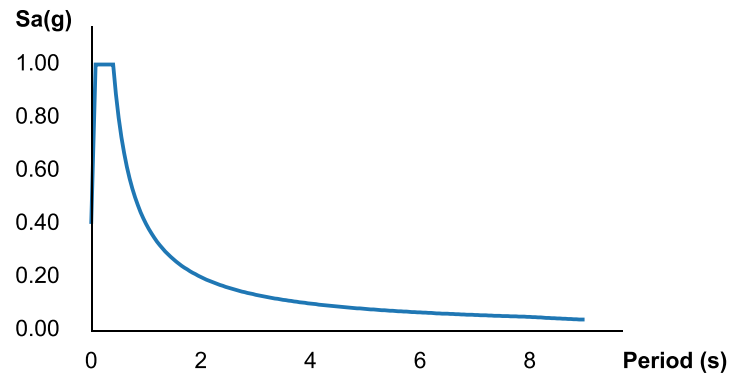
Site Class: B



MCE_R Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	1.5	MCE _R ground motion (period=0.2s)
S_1	0.6	MCE _R ground motion (period=1.0s)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	0.6	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2s SA
S_{D1}	0.4	Numeric seismic design value at 1.0s SA

▼Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	1	Site amplification factor at 1.0s

CR_S	1.056	Coefficient of risk (0.2s)
CR_1	1.016	Coefficient of risk (1.0s)
PGA	0.504	MCE_G peak ground acceleration
F_{PGA}	1	Site amplification factor at PGA
PGA_M	0.504	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	2.035	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.926	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.788	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.775	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.504	Factored deterministic acceleration value (PGA)

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

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