Appendix 6.0

Geotechnical Investigation and Percolation Test Results

GEOTECHNICAL INVESTIGATION AND PERCOLATION TEST RESULTS

WON MEDITATION CENTER
19993 GRAND AVENUE
LAKE ELSINORE, CALIFORNIA

GEOCON
W E S T, I N C.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

ANDMORE PARTNERS, INC.
LOS ANGELES, CALIFORNIA

OCTOBER 14, 2019
PROJECT NO. T2877-22-02

Project No. T2877-22-02
October 14, 2019

Andmore Partners Inc.
3530 Wilshire Boulevard, Suite 1830
Los Angeles, California 90010

Attention: Mr. Sean Mo

Subject: GEOTECHNICAL INVESTIGATION
AND PERCOLATION TEST RESULTS
WON MEDITATION CENTER
19993 GRAND AVENUE
LAKE ELSINORE, CALIFORNIA

Dear Mr. Mo:

In accordance with your authorization of Proposal No. IE-2456, Geocon West Inc. (Geocon) herein submits the results of our geotechnical investigation and percolation test results for the subject site. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The site is considered suitable for development provided the recommendations of this report are followed.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON WEST, INC.

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GEOTECHNICAL INVESTIGATION AND PERCOLATION TEST RESULTS

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation and percolation testing results for the meditation center proposed at 19993 Grand Avenue, in Lake Elsinore, California (see *Vicinity Map*, Figure 1). The purpose of the geotechnical investigation and percolation testing was to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property including faulting, liquefaction and seismic shaking based on the 2016 California Building Code (CBC) seismic design criteria. In addition, we are providing recommendations for remedial grading, shallow foundations, concrete slab-on-grade, concrete flatwork, preliminary pavement sections, lateral loading, and retaining walls. This investigation also includes a review of readily available published and unpublished geologic literature (see *List of References*).

The scope of this investigation included performing a site reconnaissance, field exploration, laboratory testing, engineering analyses, and preparation of this report. We performed our field investigation on September 16 and 17, 2019 by excavating nine backhoe test pits and two percolation holes 2 to 15 feet below the existing ground surface. The *Geologic Map*, Figure 2, presents the approximate locations of the test pits. *Appendix A* provides a detailed discussion of the field investigation including logs of the test pits and percolation test results. Details of the laboratory tests and a summary of the test results are presented in *Appendix B* and on the test pit logs in *Appendix A*.

Recommendations presented herein are based on analyses of data obtained from our site investigation and our understanding of proposed site development. If project details vary significantly from those described herein, Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 19993 Grand Avenue in the City of Lake Elsinore, California, and consists of a 16.4-acre irregular shaped parcel (APN # 382-140-002). A single-family residence is located in the southeast portion of the site. Access to the site is through a gated driveway along the eastern boundary, southwest of the intersection of Grand Avenue and Corydon Road.

The site is bounded by unincorporated Riverside County on the west and south, the City of Wildomar on the east and south, and rural residences within the City of Lake Elsinore on the north. Located in the foothills of the Santa Ana Mountains, the property has moderately high relief with granitic slopes descending to the east. Maximum heights in the area are approximately 1602 feet above mean sea level (MSL) at inclinations of approximately 2.3 to 1 (horizontal to vertical). In the area of proposed improvements, the site drains to the east. Vegetation consists of shrubs, grasses, and sparse trees throughout the majority of the property at the time of our field work. Elevations in the vicinity of the proposed structures range from approximately 1,376 feet above MSL in the northwest to approximately 1,355 feet above MSL in the southeast. The existing elevations at the proposed parking lot in the southeast corner of the site range from 1,334 feet above MSL to 1,325 feet above MSL.

The proposed development is currently planned to include a meditation center with a two-story main building, two multi-room guest houses, and associated improvements. The proposed construction will be limited to approximately three-acres, including a parking lot and access roads on the southwest flank of a northwest trending ridge. Plans for the proposed development were provided by Andmore Partners. The proposed structures and pertinent site details are depicted on the *Geologic Map* (see Figure 2).

We expect that the construction will include wood or light gauge steel framed buildings supported on spread footing foundations and with concrete slab-on-grade floors. We expect column loads will be up to 125 kips and wall loads will be up to 5 kips per linear foot. Preliminary geotechnical recommendations for design of the structure are based on these assumptions and provided herein. If structural improvements vary from our description, Geocon should be contacted to provide updated geotechnical recommendations.

The site descriptions and proposed development are based on a reconnaissance, review of published geologic literature, our field investigation, a review of the plans, and discussions with you. If development plans differ from those described herein, Geocon should be contacted for review of the plans and possible revisions to this report.

3. GEOLOGIC SETTING

The property is located in the northern part of the Peninsular Ranges Geomorphic Province, consisting of northwest-trending, predominately Cretaceous-age granitic mountain ranges bisected by alluvial, fault-controlled valleys. Quaternary- to Tertiary-age sediments flank the ranges, and lie at depth beneath the Holocene-age alluvium-filled valleys. The Province is further characterized by relatively stable structural blocks bound by active faulting.

Two distinct, relatively stable structural blocks within the Province, the Santa Ana Block to the west and Perris Block to the east, are bisected by the Elsinore fault zone (Woodford et al., 1971). The Santa Ana block is dominated by the Mesozoic-age undifferentiated low-grade metamorphic rocks and Cretaceous-age crystalline rocks that make up the Santa Ana Mountains in the vicinity of the site. The bedrock is unconformably overlain by Miocene-age basalt flows. Flanking the relatively steep, east facing slopes that define the western edge of the Elsinore fault zone, are Pleistocene-age fanglomerate and sandstone. The eastern edge of the zone is less pronounced, with scarps in the low-lying sandstone hills and buried by young alluvial deposits. The Perris block, bound by the Elsinore fault zone on the West and San Jacinto fault zone on the east, is dominated by Mesozoic-age metasedimentary rocks, Cretaceous-age crystalline ranges, and Pleistocene-age sedimentary rocks (Woodford et al. 1971).

Locally, several Holocene-age alluvium-filled valleys separate the older units. The subject site is on the western flanks of the Elsinore Valley. The Elsinore fault zone in the area of the property is complex (Geocon West, 2019). Based on a review of published geologic maps of the area, the site is underlain by Cretaceous-age granitic rocks (Kennedy, 1977; Mann, 1955) and Holocene-age alluvial deposits (Kennedy, 1977; CDMG, 1977). The granular deposits were derived primarily from the uplifted Elsinore and Santa Ana Mountains just west of the property (CDMG, 1977).

Faulting in the region is dominated by the San Andreas fault system, from east to west consists of the San Andreas, San Jacinto, Elsinore, Newport-Inglewood, and several offshore faults. The faulting predominately of northwest-striking, right lateral faults with local steeply dipping normal components. The Elsinore fault zone includes the Wildomar branch approximately 2,000 feet northeast of site and the Willard branch approximately 680 feet northeast of the site. The property is not located within a State of California Alquist-Priolo Earthquake Fault Zone [APEFZ]. However, it is located within a Riverside County Fault Study Zone (RCFSZ) for the Willard fault zone, a strand of the Elsinore fault zone. Geocon (2019), performed a fault rupture hazard study under separate cover and concluded that active faulting was not present on the site.

4. SOIL AND GEOLOGIC CONDITIONS

The geologic materials encountered consist of a veneer of topsoil, undocumented fill, Holocene-age alluvial fan deposits and Cretaceous-age granitic bedrock consisting of quartz monzonite. The undocumented artificial fill was encountered in the borings to a maximum depth of 4½ feet. Thicker deposits may be encountered between borings in the rest of the property. Descriptions of the soil and geologic conditions are shown on the boring logs located in *Appendix A* and are described herein in order of increasing age.

4.1 Topsoil (No Map Symbol)

A thin veneer of topsoil was encountered overlying the granitic bedrock within test pit T-6 and consisted of grayish brown, dry, silty fine to medium sand, with some coarse sand.

4.2 Undocumented fill (afu)

Undocumented fill was encountered in test pit T-4 and consisted of loose, dry, whitish gray silty fine to coarse sand with some cobble. The undocumented fill is likely derived from an existing road cut into the granitic bedrock.

4.3 Alluvial Fan Deposits (Qal)

Holocene-age alluvial fan deposits were encountered southern and eastern portion of the site overlying the granitic bedrock. As observed during our field exploration, alluvium consisted predominately of silty to gravelly sand, that was gray to light brown, and dry. Varying amounts of granitic cobbles and boulders were observed within the alluvium.

4.4 Quartz Monzonite (Kqm)

Cretaceous-age Quartz Monzonite was observed in western and northern portion of the site and underlies the alluvium at depth. The roadcut exposed bedrock that is highly to moderately weathered. The rock is medium grained, gray, black, and white, and slightly jointed. Where weathered, the granitic bedrock unit was hard and slightly friable. Joints were generally slightly open with some oxidation and more advanced weathering along the joint surface.

5. GROUNDWATER

We did not encounter groundwater or seepage during the site investigation. According to the California Department of Water Resources, measurements within several wells in the area indicated the depth to groundwater is between 50 to 60 feet below the existing ground surface. It is not uncommon for seepage conditions to develop where none previously existed. Groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

6. GEOLOGIC HAZARDS

6.1 Faulting

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a currently established State of California Alquist-Priolo Earthquake Fault Zone for surface fault rupture hazards. However, it is within a Riverside County Fault Hazard Zone. Geocon (2019) prepared a fault rupture hazard study for the site and concluded that active fault was not present at the site and the no structural setbacks are required. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site.

According to the *Fault Activity Map of California* (2010), the closest active faults to the site are the Willard strand of the Elsinore fault, located 680 feet to the northeast, and the Wildomar strand of Elsinore fault, located approximately 2,000 feet to the northeast. Faults within a 50-mile radius of the site are listed in Table 6.1.1. Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 6.1.2

TABLE 6.1.1
KNOWN ACTIVE FAULTS WITHIN 50 MILES OF THE SITE

Fault Name	CGS Number	Maximum Earthquake Magnitude (Mw)	Distance from Site (miles)	Direction from Site
Elsinore (Wildomar)	460	6.8	<1	E
Elsinore (Glen Ivy North)	461	6.8	2	E
Wolf Valley	469	6.8	14	SSE
Elsinore (Main Street)	446	6.8	14	S
San Jacinto (Casa Loma)	457	6.9	21	E
San Jacinto (Clark)	459	6.9	23	E
Chino	431	6.7	23	SW
Elsinore (Julian)	483	6.8	23	SE
Elsinore (Whittier)	444	6.8	28	SW
San Gorgonio Pass (Western Extension)	448	7.1	26	E
San Gorgonio Pass	455	7.1	31	E
San Andreas (South Branch-Banning)	452	7.5	31	SE
San Andreas (Cajon Canyon to Burro Flats)	427A	7.5	37	E
San Jacinto (San Jacinto)	401	7.2	39	ENE
Red Hill Etiwanda Avenue	398	6.5	40	E
San Jacinto (Glen Helen)	402	6.7	41	NE
Lytle Creek	400	6.7	41	NE
Cucamonga	399	6.9	41	NE
Newport Inglewood (North Branch)	440	7.1	43	W
Palos Verdes	437	6.5	45	W
Coyote Creek Fault	479	6.9	45	SE
Pinto Mountain	425	7.2	46	E
San Andreas (Palmdale to Cajon Canyon)	358	7.5	50	NE

Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 6.1.2.

TABLE 6.1.2
HISTORIC EARTHQUAKE EVENTS WITH REPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto	December 25, 1899	6.7	28	NE
San Jacinto	April 21, 1918	6.8	28	NE
Loma Linda Area	July 22, 1923	6.3	27	N
Long Beach	March 10, 1933	6.4	38	W
Buck Ridge	March 25, 1937	6.0	55	SE
Imperial Valley	May 18, 1940	6.9	119	SSE
Desert Hot Springs	December 4, 1948	6.0	63	E
Tehachapi	July 21, 1952	7.5	136	NW
Arroyo Salada	March 19, 1954	6.4	111	S
Borrego Mountain	April 8, 1968	6.5	61	SE
San Fernando	February 9, 1971	6.6	83	NW
Whittier Narrows	October 1, 1987	5.9	54	NW
Joshua Tree	April 22, 1992	6.1	79	ENE
Landers	June 28, 1992	7.3	64	NE
Big Bear	June 28, 1992	6.4	49	NE
Northridge	January 17, 1994	6.7	82	WNW
Hector Mine	October 16, 1999	7.1	90	NE
Ridgecrest/China Lake	July 5, 2019	7.1	149	N

6.2 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects the earth surface. The potential for ground rupture is considered to be very low due to the absence of active or potentially active faults at the subject site.

6.3 Liquefaction

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not.

Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

As discussed in the Groundwater Section of this report, groundwater is anticipated in greater than 50 feet below the ground surface. Based on the absence of groundwater, the medium dense nature and relatively shallow depth of the alluvium, the potential for liquefaction and seismically induced settlement at the site is negligible and not a design consideration.

6.4 Expansive Soil

The alluvium generally consists of silty and poorly graded sands. Laboratory testing results indicate a sample of the near surface soil exhibits a "very low" expansion potential (expansion index [EI] of 20 or less) with test results showing an expansion index of 0.

6.5 Hydrocompression

Hydrocompression is the tendency of unsaturated soil structure to collapse upon wetting resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and recompacted during remedial grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocompression of the soil exists.

Due to the relatively shallow alluvium underlain by granitic bedrock, and the recommended remedial grading in the conclusion section of this report, the potential for hydrocompression is not a design consideration.

6.6 Seiches and Tsunamis

Seiches are caused by the movement of an inland body of water due from a seismic event. Lake Elsinore is approximately 2.3 miles north of the site, with a water surface elevation of approximately 1,238 feet MSL, and a depth of approximately 42 feet. Recent improvements at the lake include channelizing potential influx of water along the southwest portion of the lake into dedicated drainage channels that flow into Murrieta Creek. Therefore, flooding due a seiche is not a design consideration.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The site is located approximately 22 miles from the Pacific Ocean at an elevation greater than 1,300 feet MSL, with the Elsinore and Santa Ana Mountains between the coast and the site. Therefore, the risk of tsunamis affecting the site is negligible and not a design consideration.

6.7 Inundation

Lake Elsinore and Canyon Lake are in the vicinity of the site. According to the State of California, Department of Water Resources, Division of Safety of Dams, the site is not within an inundation zone due to dam failure of either lake. Lake Elsinore is in a natural depression, and has no dam to fail. Failure of the Canyon Lake dam would channel water in Lake Elsinore and raise the lake elevation causing flooding to south of the lake. The limits of flooding are approximately Palomar Road and Corydon Road, approximately 3,500 feet east of the site. Therefore, inundation due to dam failure is not a design consideration.

6.8 Landslides

Landslides are not mapped on or near the site. Due to the granitic nature of the slopes at the site, we opine that landslides are not present at the property or at a location that could impact the subject site.

6.9 Rock Fall Hazards

Due to the granitic nature of the ascending slopes and observed boulders near the site, rock falls may impact the site. The slopes are vegetated and observation was obscured. Further evaluation should be considered for potential rock fall evaluation.

6.10 Slope Stability

Graded slopes are not proposed on the site at this time, and the intact nature of the natural granitic slopes near the site lead us to opine slope stability is not a design consideration.

7. SITE INFILTRATION

Percolation testing was performed in accordance with the procedures outlined in *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* (RC BMP) for infiltration basins. The percolation test locations are depicted on the *Geologic Map* (see Figure 2).

Percolation test holes were excavated to four feet using backhoe equipped with a 24-inch diameter bucket. The final foot was hand excavated and a 10-inch-diameter perforated 5-gallon bucket was placed faced down in the resulting void space. Two inches gravel were place at the bottom of the hole. A 3-inch diameter hole was cut into the bottom of the bucket (facing up). A 3-inch PVC pipe was placed into the hole and extended to the gravel layer. The test pit was backfilled with the PVC pipe just above the surface to convey water into the portion of the hole for testing. The test locations were pre-saturated prior to testing. Percolation testing began within 24 hours after the holes were presaturated. Percolation data sheets are presented in *Appendix A* of this report. Calculations to convert the percolation test rate to infiltration test rates are presented in Table 7.0 below. During the tests, the amount of time it took to pour 5 gallons of water into the test hole and measure the initial reading, the majority of water had already percolated into the ground. At every 10 minute reading interval, all of the water had percolated into the ground. According to RCBMP Appendix A Table I, Infiltration Basin, Option 1, a minimum factor of safety of 3 must be applied to the measured values below.

TABLE 7.0
INFILTRATION TEST RATES FOR PERCOLATION AREAS

Parameter	P-1	P-2
Depth (inches)	55.1	53.4
Test Type	Sandy	Sandy
Change in head over time: ΔH (inches)	8.9	3.0
Average head: Havg (inches)	4.4	1.5
Time Interval (minutes): Δt (minutes)	10	10
Radius of test hole: r (inches)	5	5
Tested Infiltration Rate: It (inches/hour)	19.2	11.2

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 From a geotechnical engineering standpoint, the site is suitable for construction of the proposed development provided the recommendations presented herein are implemented in design and construction of the project.

8.1.2 Potential geologic hazards at the site include seismic shaking.

8.1.3 The site is located less than 1 mile from the nearest active fault. Based on our background research, referenced surface fault rupture hazard investigation, and this investigation, it is our opinion active, potentially active, or inactive faults do not extend across the site. Risks associated with seismic activity consist of the potential for moderate to strong seismic shaking.

8.1.4 Our field investigation indicates geologic units at the site include undocumented fill, alluvium and granitic bedrock at the surface. The undocumented fill and the alluvium are not considered suitable for the support of compacted fill and settlement-sensitive structures. Remedial grading of these deposits will be required as discussed herein. The existing site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

8.1.5 A significant amount of on-site soils are granular in nature, having little to no cohesion and may be subject to caving in unshored excavations. It is the responsibility of the contractor to ensure that excavations and trenches are properly shored and maintained in accordance with OSHA rules and regulations to maintain the stability of adjacent existing improvements.

8.1.6 The laboratory tests indicate that the site soils are non-expansive and have a “very low” expansion potential. If medium to highly expansive soils are encountered at the site, they should be exported from the site or selectively graded and placed in the deeper fill areas to allow for the placement of low expansion material at the finish pad grade.

8.1.7 Grading plans were not available to review at the time of this report. However, based on a review of the site plan, existing grades and anticipated grades, cuts and fills of up to 15 feet are expected, not including remedial grading.

8.1.8 Remedial grading will address collapse potential of the alluvial soils. Proper site drainage should be maintained. Landscape planters that saturate the subsurface or stormwater infiltration structures should not be used within 20 feet of the proposed buildings or other on grade improvements.

8.1.9 Excavations into the granitic bedrock and alluvial fan deposits are expected to encounter oversize materials (greater than 12 inches). Oversize materials are not suitable for reuse in the upper 10 feet of engineered fill. Processing of cobbly site soils (screening or crushing) should be anticipated before reuse as fill material.

8.1.10 Due the anticipated granitic bedrock, consideration should be given to overexcavating utility trenches and any other below grade improvements (i.e. perimeter wall footings) during grading.

8.1.11 We did not encounter groundwater during our investigation and do not expect groundwater would impact site improvements. However, wet conditions and seepage could affect proposed construction if grading and improvement operations occur during or shortly after a rain event.

8.1.12 Proper drainage should be maintained in order to preserve the design properties of the fill in the sheet-graded pad and slope areas.

8.1.13 Changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Once final grading plans become available, they should be reviewed by this office to evaluate the necessity for review and possible revision of this report.

8.1.14 Recommended grading specifications are provided in *Appendix C*.

8.2 Excavation and Soil Characteristics

8.2.1 Excavation of the undocumented fill and alluvium should be possible with moderate effort using conventional heavy-duty equipment. Some difficulty in excavation may be encountered where cobbles are encountered. Excavations within the upper portions of the bedrock should be rippable. Areas of non-rippable bedrock should be anticipated to be encountered.

8.2.2 The soil encountered in the field investigation is considered to be "non-expansive" (expansion index [EI] of less than 20) as defined by 2016 California Building Code (CBC) Section 1803.5.3. Table 8.2.2 presents soil classifications based on the expansion index. Based on the laboratory test results, we expect a majority of the soil encountered will possess a "very low" expansion potential (EI between 0 and 20). Although unlikely, any medium to highly expansive soils encountered at the site should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements. Additional testing for expansion potential should be performed during grading and once final grades are achieved.

TABLE 8.2.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM D 4829 Expansion Classification	2016 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

8.2.3 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. *Appendix B* presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the location tested possess a sulfate content of 0.000 percent (less than 10 parts per million [ppm]) equating to an exposure class of "S0" as defined by 2016 CBC Section 1904.3 and ACI 318. Table 8.2.3 presents a summary of concrete requirements set forth by 2016 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

TABLE 8.2.3
REQUIREMENTS FOR CONCRETE EXPOSED TO
SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0	**SO_4<0.10**	**No Type Restriction**	**n/a**	**2,500**
S1	0.10≤SO_4<0.20	II	0.50	4,000
S2	0.20≤SO_4≤2.00	V	0.45	4,500
S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

[1] Maximum water to cement ratio limits do not apply to lightweight concrete

8.2.4 Laboratory testing indicates the site soils have a minimum electrical resistivity of 10,300 ohm-cm, possess 36 ppm chloride, less than 10 ppm sulfate, and a pH of 7.9. As shown in Table 8.2.4 below, the site would not be classified as "corrosive" to buried metallic improvements, in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2018).

TABLE 8.2.4
CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Corrosive	<1,100	500 or greater	1,500 or greater	5.5 or less

8.2.5 Geocon does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

8.3 Rippability

8.3.1 Based on variability within the granitic bedrock, difficulty in excavating should be expected. We encountered refusal at various depths within the bedrock.

8.3.2 Bedrock will generally be rippable with large construction equipment in good working order such as a D9 dozer with a single shank ripper. Areas of non-rippable bedrock or large core stones may be encountered that will require blasting or expansion breaking to excavate the bedrock should be expected.

8.4 Seismic Design Criteria

8.4.1 We used the computer program *U.S. Seismic Design Maps*, provided by the California Office of Statewide Health Planning and Development (OSHPD) to evaluate the seismic design criteria. Table 8.4.1 summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements as currently proposed should be designed using a Site Class C in accordance with ASCE 7-10 Section 20.3.1. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10 using blow count data presented on the boring logs in *Appendix A*. The values presented in Table 8.4.1 are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.4.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.25g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.904g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.300	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.25g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.175g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.5g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.783g	Section 1613.3.4 (Eqn 16-40)

8.4.2 Table 8.4.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 8.4.2
2016 CBC SITE ACCELERATION PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Site Class	C	Section 1613.3.2
Mapped MCE_G Peak Ground Acceleration, PGA	0.894g	Figures 2 through 42-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.894g	Section 11.8.3 (Eqn 11.8-1)

8.4.3 Conformance to the criteria in Tables 8.4.1 and 8.4.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.5 Temporary Excavations

8.5.1 The recommendations included herein are provided for temporary excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project. Temporary unsurcharged embankments should be designed by the contractor's competent person in accordance with OSHA regulations.

8.5.2 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. Recommendations for temporary shoring can be provided in an addendum if needed.

8.5.3 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's personnel should inspect the soil exposed in the cut slopes during excavation in accordance with OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur. Excavations should be stabilized within 30 days of initial excavation.

8.6 Grading

8.6.1 Grading should be performed in accordance with the recommendations provided in this report, the *Recommended Grading Specifications* contained in *Appendix C* and the City of Lake Elsinore standards.

8.6.2 Prior to commencing grading, a pre-construction conference should be held at the site with the owner/developer, City inspector, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

8.6.3 Site preparation should begin with the removal of deleterious material, debris, buried trash, and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

8.6.4 Undocumented fill and alluvium in the building areas should be removed to expose bedrock. Based on our test pits, the depth of removal should be on the order of 7 to 14 feet., however, test pit TP-8 extended to 15 feet and did not encounter bedrock. The excavations should be extended laterally a minimum distance of 6 feet beyond the building footprint or for a distance equal to the depth of removal, whichever is greater. Where the lateral over-excavation is not possible, structural setbacks or deepened footings may be required.

8.6.5 The actual depth of removal should be evaluated by the engineering geologist during grading operations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned as necessary, and properly compacted.

8.6.6 Cut lots and cut/fill transition lots should be overexcavated to a depth of at least 2 feet below the bottom of footings, or H/3 (where H is the maximum depth of fill within a lot and within a 1:1 projection of the lot).

8.6.7 The site should then be brought to final subgrade elevations with fill compacted in layers. In general, soil native to the site is suitable for use as fill if free from oversize material (rock fragments larger than 6 inches), vegetation, debris and other deleterious material. Layers of fill should be about 6 to 8 inches in loose thickness and no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content, as determined in accordance with ASTM D 1557. Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill. The upper 12 inches of subgrade soil underlying pavement should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content shortly before paving operations.

8.6.8 Oversize material should be expected during the grading of the site. Larger rocks (>12") should be kept ten feet below design grades and out of proposed utility trenches. Rock windrows or the placement of induvial rocks for burial may be accomplished under the observation of Geocon in accordance with recommended grading specifications in *Appendix C.*

8.6.9 Import fill soil (if necessary) should consist of granular materials with a "low" expansion potential (EI of less than 50), free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

8.6.10 Foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing fill, steel, gravel or concrete.

8.7 Utility Trench Backfill

8.7.1 Utility trenches should be properly backfilled in accordance with the requirements of City of Lake Elsinore and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well graded crushed rock or clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe. The bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). <u>The use of well graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil.</u> The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. Backfill of utility trenches should not contain rocks greater than 3 inches in diameter. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized and additional stabilization should be considered at these transitions.

8.7.2 Trench excavation bottoms must be observed and approved in writing by the Geotechnical Engineer, prior to placing bedding materials, fill, gravel, or concrete.

8.8 Earthwork Grading Factors

8.8.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Based on our experience, the shrinkage of undocumented fill and alluvium is expected to be on the order of 5 to 10 percent when compacted to at least 90 percent of the laboratory maximum dry density. The granitic bedrock is expected to bulk on the order of 15 to 20 percent. This estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations

8.9 Foundation and Concrete Slab-On-Grade Recommendations

8.9.1 The foundation recommendations presented herein are for the proposed building subsequent to the recommended grading assuming that the buildings are founded in soils with a low expansion potential. If soils with a medium or high expansion potential are placed within 4 feet of finish grade, Geocon should be contacted for additional recommendations. The proposed structures can be supported on a shallow foundation system bearing in newly placed compacted fill.

8.9.2 Foundations for the structures should consist of either continuous strip footings and/or isolated spread footings. Continuous footings should be at least 18 inches wide and extend at least 18 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 24 inches and should also extend at least 18 inches below lowest adjacent pad grade. A wall/column footing dimension detail depicting footing embedment is provided on Figure 3.

8.9.3 From a geotechnical engineering standpoint, concrete slabs-on-grade for the structure should be at least 4 inches thick and be reinforced with at least No. 3 steel reinforcing bars placed 24 inches on center in both directions. The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slab for supporting equipment and storage loads. A thicker concrete slab may be required for heavier loading conditions. To reduce the effects of differential settlement on the foundation system, thickened slabs and/or an increase in steel reinforcement can provide a benefit to reduce concrete cracking.

8.9.4 Steel reinforcement for continuous footings should consist of at least two No. 4 steel reinforcing bars placed horizontally in the footings, one near the top and one near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

8.9.5 The recommendations presented herein are based on soil characteristics only (EI of 50 or less) and are not intended to replace steel reinforcement required for structural considerations.

8.9.6 Foundations may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf) (dead plus live load). The value presented herein is for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

8.9.7 The maximum expected static settlement for the planned structures supported on conventional foundation systems with the above allowable bearing pressure and deriving support in engineered fill is estimated to be 1½ inch and to occur below the heaviest loaded structural element. Differential settlement is estimated to be on the order of ¾ inch over a horizontal distance of 40 feet. Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements within this report should be reviewed and revised, if necessary

8.9.8 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.9.9 The bedding sand thickness should be evaluated by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 4 inches. Placement of 3 inches and 4 inches of sand is common practice in southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl.

8.9.10 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

8.9.11 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil, or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.9.12 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

8.10 Concrete Flatwork

8.10.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein. Slab panels should be a minimum of 4 inches thick and, when in excess of 8 feet square, should be reinforced with No. 3 reinforcing bars spaced 24 inches on center in each direction to reduce the potential for wide cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

8.10.2 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some movement due to swelling or settlement; therefore, the steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.10.3 Where exterior flatwork abuts structures at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.10.4 The recommendations presented herein are intended to reduce the potential for cracking as a result of differential movement. However, even with the incorporation of the recommendations presented herein, concrete will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.11 Conventional Retaining Walls

8.11.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

8.11.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 60 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

8.11.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls with a level backfill surface should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 55 pcf.

8.11.4 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (Section 1803.5.12 of the 2016 CBC).

8.11.5 A seismic load of 10 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2016 CBC. The seismic load is applied as an equivalent fluid pressure along the height of the wall and the calculated loads result in a maximum load exerted at the base of the wall and zero at the top of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of PGA_M calculated from ASCE 7-10 Section 11.8.3.

8.11.6 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.11.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. A typical drain detail for each option is shown on Figure 4. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

8.12 Lateral Loading

8.12.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid density of 300 pounds per cubic foot (pcf) should be used for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

8.12.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.40 should be used for design. The friction coefficient may be reduced depending on the vapor barrier or waterproofing material used for construction in accordance with the manufacturer's recommendations.

8.12.3 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

8.13 Preliminary Pavement Recommendations

8.13.1 We calculated the flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) and Lake Elsinore *Standard Drawings* using a of Traffic Index of 5. The project civil engineer and owner should evaluate the final Traffic Index for the pavements and review the pavement designations to determine appropriate locations for pavement thickness. Laboratory testing indicates an R-value of 68. We have used a preliminary R-value of 50 (the maximum allowable by Caltrans Design Manual) for the subgrade soils for the purposes of this analysis. The final pavement sections should be based on the R-value of the subgrade soil encountered at final subgrade elevation. Table 8.13.1 presents the preliminary flexible pavement sections for local street class in accordance with the City of Lake Elsinore *Standard Drawing No. 100A*. Geocon should be contacted for additional recommendations if other TI's are applicable.

TABLE 8.13.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

City Roadway Classification / Anticipated Traffic	Assumed Traffic Index (TI)	Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Local Street / Automobiles and Light-Duty Vehicles	5	50	3.5	4.0

8.13.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

8.13.3 Base materials should conform to Section 26-1.028 of the *Standard Specifications for The State of California Department of Transportation (Caltrans)*. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook)*.

8.13.4 A rigid Portland cement concrete (PCC) pavement section should be placed in heavy truck areas, driveway aprons, and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R *Guide for Design and Construction of Concrete Parking Lots* and City of Lake Elsinore *Standard Drawing No. 209*. using the parameters presented in Table 8.13.4.

TABLE 8.13.4
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	200 pci
Modulus of Rupture for Concrete, M_R	500 psi
Traffic Category, TC	A, B, and C
Average Daily Truck Traffic, ADTT	10, 25, and 100

8.13.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.13.5.

TABLE 8.13.5
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Stalls (TC=A)	5.0
Moderate Truck Traffic (TC=B)	6.0
Heavy Truck and Fire Lane Areas (TC=C)	6.5

8.13.6 The PCC pavement should be placed over a subgrade that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

8.13.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., 6-inch and 7.5-inch-thick slabs would have an 8- and 9.5-inch-thick edge, respectively). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.13.8 In order to control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab in accordance with the referenced ACI report.

8.13.9 The performance of pavements is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement surfaces will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

8.14 Site Drainage and Moisture Protection

8.14.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.14.2 In the case of basement walls or building walls retaining landscaping areas, a water-proofing system should be used on the wall and joints, and a Miradrain drainage panel (or similar) should be placed over the waterproofing. The project architect or civil engineer should provide detailed specifications on the plans for all waterproofing and drainage.

8.14.3 Landscape planters that saturate the subsurface should not be used within 20 feet of the proposed structure or other settlement sensitive on grade improvements. Localized surface settlement should be anticipated in areas where water is allowed to infiltrate into the subsurface.

8.14.4 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

8.14.5 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

8.14.6 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.15 Grading and Foundation Plan Review

8.15.1 Geocon should review the project grading and foundation plans prior to final design submittal to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations, if necessary.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon.

3. This report is issued with the understanding that it is the responsibility of the owner or their representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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8. California Department of Conservation, 1996, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California*, Open File Report 96-08.

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SOURCE: Google Earth Pro, 2019

VICINITY MAP

GEOCON
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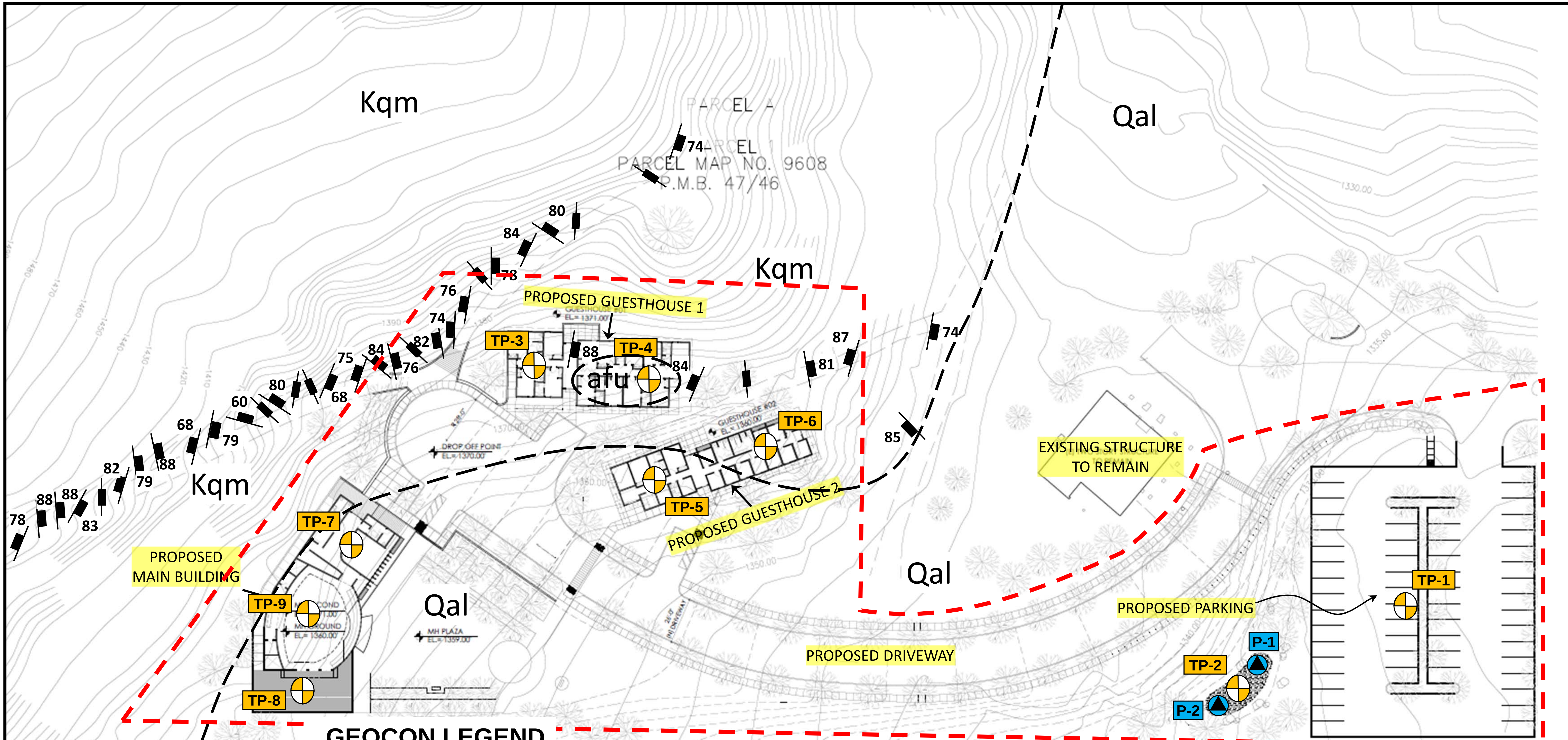
PHONE 951-304-2300 FAX 951-304-2392

WON MEDITATION CENTER
19993 GRAND AVENUE
LAKE ELSINORE, CALIFORINA

PDT		

OCTOBER 2019	PROJECT NO. T2877-22-02	FIG. 1



GEOCON LEGEND

Locations are approximate

TP-9 .....TEST PIT LOCATION

P-2 ..... PERCOLATION TEST LOCATION

afu ..... UNDOCUMENTED FILL

80 ..... STRIKE AND DIP OF JOINT

..... STRIKE OF VERTICAL JOINT

Qal ..... ALLUVIAL FAN DEPOSITS

..... GEOLOGIC CONTACT

Kqm ..... QUARTZ MONZONITE

..... LIMITS OF THIS REPORT

N

0 60 120

Source: *Enlarged Site Plan*, Andmore Partners, dated July 22, 2019.

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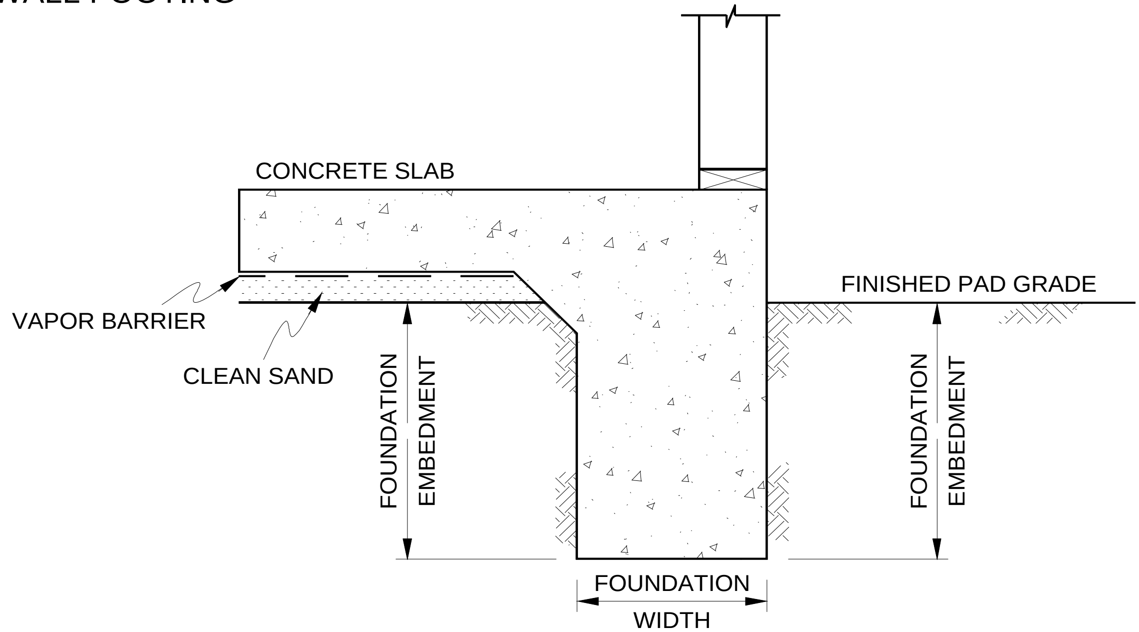
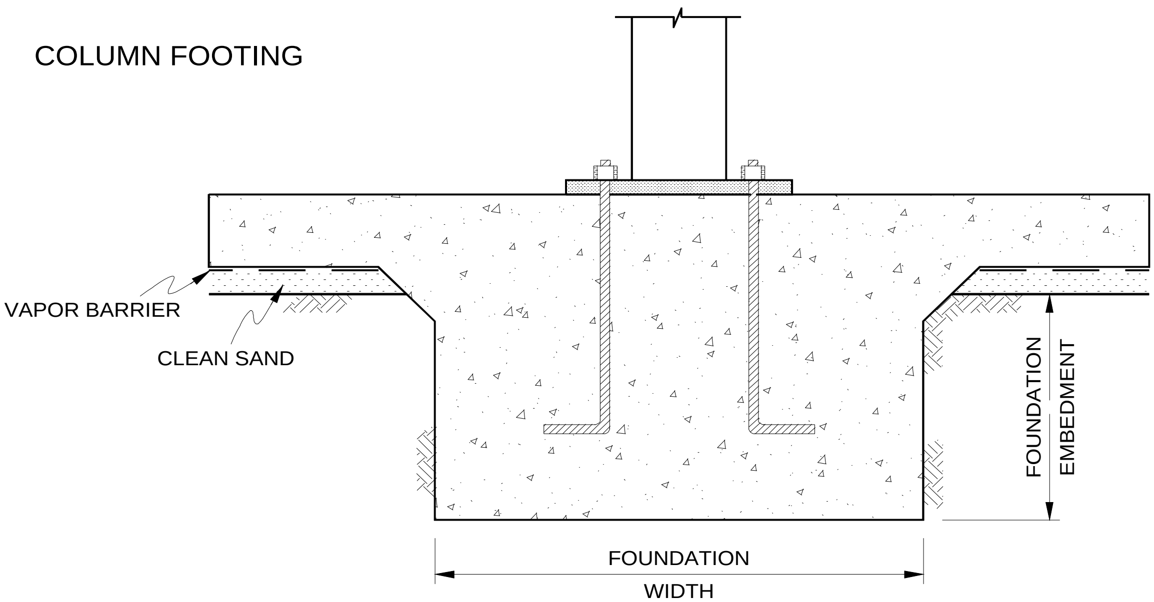
GEOLOGIC MAP		
WON MEDITATION CENTER 19993 GRAND AVENUE LAKE ELSINORE, CALIFORINA		
SEPTEMBER 2019	PROJECT NO. T2877-22-02	FIG. 2

PDT

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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WALL / COLUMN FOOTING DETAIL

WON MEDITATION CENTER
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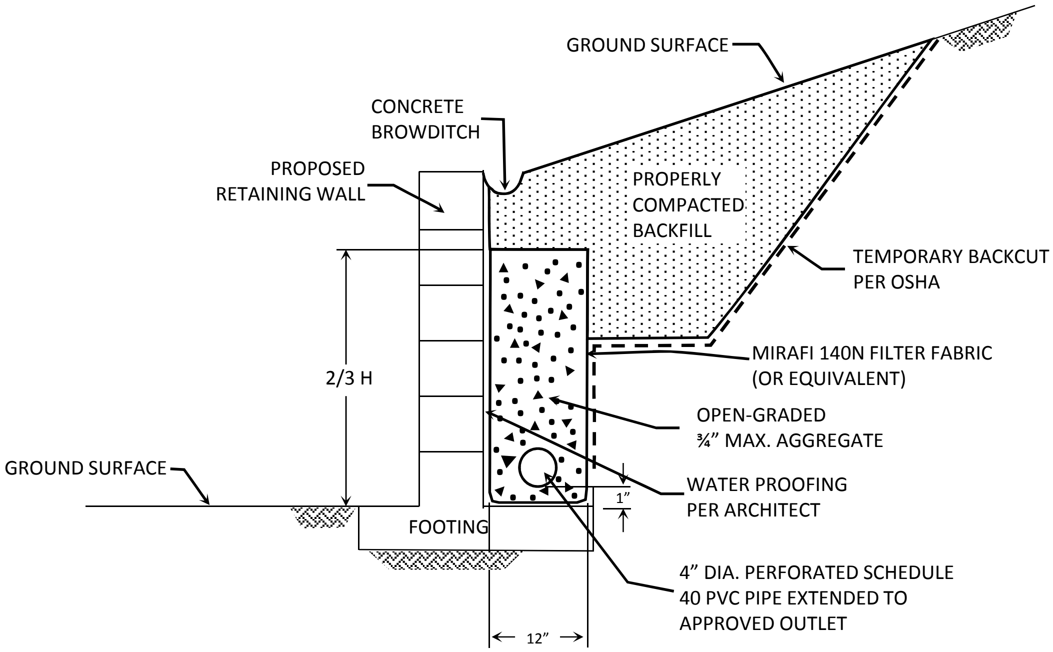
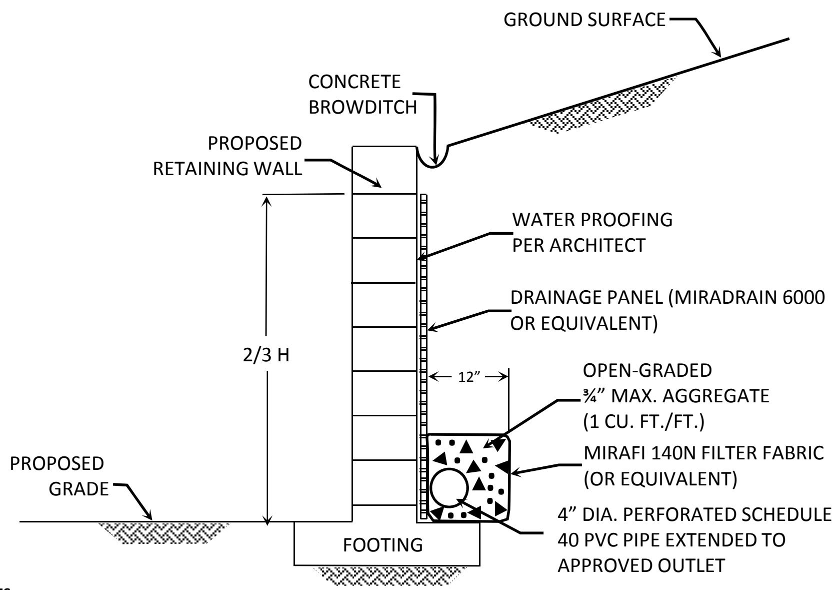
LAKE ELSINORE, CALIFORNIA

OCTOBER 2019	PROJECT NO. T2877-22-02	FIG. 3





NOTES:

DRAIN SHOULD BE UNFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPIMG

CONCRETE BROW DITCH RECOMMENDED FOR SLOPE HEIGHTS
GREATER THAN 6 FEET

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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WON MEDITATION CENTER
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LAKE ELSINORE, CALIFORNIA

PDT			OCTOBER 2019	PROJECT NO. T2877-22-02	FIG. 4

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on September 16 and 17, 2019, and consisted of a site reconnaissance and excavation eleven exploratory test pits utilizing a rubber-tire backhoe equipped with a 24-inch bucket. Field work for our investigation included a subsurface exploration, soil sampling, and percolation testing. The test pits were excavated to depths of 2 to 15 feet below the existing ground surface. We collected bulk samples from the test pits. The samples of disturbed soils were transported to our laboratory for testing.

We visually examined the soil conditions encountered within the test pits, classified, and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the test pits are presented on Figures A-1 through A-11. The logs depict the general soil and geologic conditions encountered and the depth at which we obtained the samples. The *Geologic Map*, Figure 2 presents the locations of the exploratory test pits.

Percolation testing was performed on September 16, 2019 in accordance with *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* for infiltration basins. The percolation tests were run in accordance with *Section 2.3.*, *Shallow Percolation Test*. The percolation test data is presented on Figures A-12 and A-13.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP-1** ELEV. (MSL.) **1328** DATE COMPLETED **09/16/2019** EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; weeds; roots			
2					-Rusted pipe encountered			
					-Some granitic derived cobbles			
4	TP-1@4-5'				-Increase in cobbles			
					Total Depth 5' Groundwater not encountered Backfilled 9/16/2019			

Figure A-1,
Log of Test Pit TP-1, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

TEST PIT TP-2

ELEV. (MSL.) **1332** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP	**ALLUVIAL FAN DEPOSITS (Qal)** Poorly Graded SAND, medium dense, dry, grayish brown; fine to coarse sand; some gravel; trace silt; weeds; roots			
2							96.6	3.4
4					-Becomes damp		91.7	5.5
6								
8					**QUARTZ MONZONITE (Kqm)** Highly weathered, slightly jointed, yellowish brown, moderately strong, GRANITIC BEDROCK; medium-grained; excavates as a gravelly sand with cobble			
10								
					-Becomes moderately weathered			

Total Depth 11.5' (Refusal)
Groundwater not encountered
Backfilled 9/16/2019

Figure A-2,
Log of Test Pit TP-2, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2877-22-02

TEST PIT TP-3

ELEV. (MSL.) **1372** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	TP-3@0-1'				**QUARTZ MONZONITE (Kqm)** Moderately weathered, slightly jointed, yellowish brown, strong, GRANITIC BEDROCK; medium-grained; excavates as a cobbly sand with gravel			
2					Total Depth 2' (Refusal) Groundwater not encountered Backfilled 9/16/2019			

Figure A-3,
Log of Test Pit TP-3, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

TEST PIT TP-4

ELEV. (MSL.) **1369** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 2.5	TP4@1-2'			SP	**UNDOCUMENTED FILL (afu)** Poorly Graded SAND with gravel, loose, dry, whithish light gray; fine to coarse sand; some cobble			
2.5 – 6.5				SM	Silty SAND, medium dense, dry, brown; fine to coarse sand -Becomes damp		106.7	5.5
6.5 – 13					**QUARTZ MONZONITE (Kqm)** Highly weathered, slightly jointed, yellowish brown, moderately strong, GRANITIC BEDROCK; medium-grained; excavates as a gravelly sand with silt and cobble -Becomes predominately fine-grained; some medium-grained; excavates as a silty sand with some friable gravel -Becomes moderately weathered; strong			

Total Depth 13' (Refusal)
Groundwater not encountered
Backfilled 9/16/2019

Figure A-4,
Log of Test Pit TP-4, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2877-22-02

TEST PIT TP-5

ELEV. (MSL.) 1359 DATE COMPLETED 09/16/2019

EQUIPMENT BACKHOE BUCKET 24" BY: PDT

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty SAND, medium dense, dry, light brown; fine to coarse sand; weeds; roots			
2					-Becomes damp, dark grayish brown; trace porosity; root hairs		103.2	6.8
4								6.1
6								
8								
10								
12					**QUARTZ MONZONITE (Kqm)** Highly weathered, slightly jointed, yellowish brown, moderately strong, GRANITIC BEDROCK; medium-grained; excavates as a silty sand with gravel -Becomes moderately weathered; strong			
14					Total Depth 14' (Refusal) Groundwater not encountered Backfilled 9/16/2019			

Figure A-5,
Log of Test Pit TP-5, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

TEST PIT TP-6

ELEV. (MSL.) **1362** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**TOPSOIL** Silty SAND, loose, dry, grayish brown; fim to medium sand; roots; weeds			
2					**QUARTZ MONZONITE (Kqm)** Moderately weathered, slightly jointed, yellowish brown, strong, GRANITIC BEDROCK; medium-grained; excavates as a cobbly sand with gravel -Becomes moderately weathered; strong			

Total Depth 3' (Refusal)
Groundwater not encountered
Backfilled 9/16/2019

Figure A-6,
Log of Test Pit TP-6, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2877-22-02

TEST PIT TP-7

ELEV. (MSL.) **1366** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SP	**ALLUVIAL FAN DEPOSITS (Qal)** Poorly Graded SAND with silt, medium dense, dry, brown; fine to coarse sand; trace gravel; weeds; roots			
2							112.1	4.4
					-Some gravel			
4	TP-4@4-5'						108.2	4.2
					-Becomes damp; some cobble			
6					-Becomes moist			
8								
10								
12								
14					**QUARTZ MONZONITE (Kqm)** Highly weathered, slightly jointed, yellowish brown, moderately strong, GRANITIC BEDROCK; medium-grained; excavates as a gravelly sand with silt and cobble			

Total Depth 15'
Groundwater not encountered
Backfilled 9/16/2019

Figure A-7,
Log of Test Pit TP-7, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

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PROJECT NO. T2877-22-02

TEST PIT TP-8

ELEV. (MSL.) **1363** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty SAND, medium dense, dry, brown; fine to coarse sand; trace porosity; weeds; roots			
2							100.1	4.4
4							109.6	5.4
					-Becomes damp			
6								
					-Becomes moist			
8								
10								
12								
14								
					Total Depth 15' Groundwater not encountered Backfilled 9/16/2019			

Figure A-8,
Log of Test Pit TP-8, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

TEST PIT TP-9

ELEV. (MSL.) **1371** DATE COMPLETED **09/16/2019**

EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ALLUVIAL FAN DEPOSITS (Qal)** Silty SAND, medium dense, dry, brown; fine to coarse sand; weeds; roots			
2								
4					-Becomes damp; trace porosity			
6					-Few cobbles			
8					-Becomes moist			
10								
12								
14					**QUARTZ MONZONITE (Kqm)** Moderately weathered, slightly jointed, yellowish brown, strong, GRANITIC BEDROCK; medium-grained; excavates as a cobbly sand with gravel			

Total Depth 14' (Refusal)
Groundwater not encountered
Backfilled 9/16/2019

Figure A-9,
Log of Test Pit TP-9, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)

... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TEST PIT P-1 ELEV. (MSL.) 1331 DATE COMPLETED 09/16/2019 EQUIPMENT BACKHOE BUCKET 24" BY: PDT	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SP	**ALLUVIAL FAN DEPOSITS (Qal)** Poorly Graded SAND with silt, loose, dry, grayish brown; fine to coarse sand; some gravel; roots; weeds			
2					-Decrease in silt; increase in sand and gravel			
4	P-1@4-5'							
					Total Depth 5' Final foot excavated by hand Groundwater not encountered Backfilled 9/17/2019			

Figure A-10,
Log of Test Pit P-1, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2877-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT P-2** ELEV. (MSL.) **1333** DATE COMPLETED **09/16/2019** EQUIPMENT **BACKHOE BUCKET 24"** BY: **PDT**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4	P-2@4-5'			SP	**ALLUVIAL FAN DEPOSITS (Qal)** Poorly Graded SAND, medium dense, dry, grayish brown; fine to coarse sand; some gravel; trace silt; roots; weeds			
					Total Depth 5' Final foot excavated by hand Groundwater not encountered Backfilled 9/17/2019			

Figure A-11,
Log of Test Pit P-2, Page 1 of 1

T287-22-02 TEST PITS.GPJ

SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PERCOLATION TEST REPORT

Project Name:	Andmore Won Meditation Center	**Project No.:**	T2877-22-02
Test Hole No.:	P-1	**Date Excavated:**	9/16/2019
Length of Test Pipe:	61.7 inches	**Soil Classification:**	SP
Height of Pipe above Ground:	6.6 inches	**Presoak Date:**	9/16/2019
Depth of Test Hole:	55.1 inches	**Perc Test Date:**	9/17/2019
Check for Sandy Soil Criteria Tested by:	Weidman	**Percolation Tested by:**	Weidman

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	7:50 AM 8:15 AM	25	25	19.7	0.0	19.7	1.3
2	8:15 AM 8:40 AM	25	50	8.9	0.0	8.9	2.8

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:40 AM 8:50 AM	10	10	8.9	0.0	8.9	1.1
2	8:50 AM 9:00 AM	10	20	8.9	0.0	8.9	1.1
3	9:00 AM 9:10 AM	10	30	8.9	0.0	8.9	1.1
4	9:10 AM 9:20 AM	10	40	7.3	0.0	7.3	1.4
5	9:20 AM 9:30 AM	10	50	8.9	0.0	8.9	1.1
6	9:30 AM 9:40 AM	10	60	8.9	0.0	8.9	1.1

Infiltration Rate (in/hr):	19.2
Radius of test hole (in):	5
Average Head (in):	4.4

Figure A-12

PERCOLATION TEST REPORT

Project Name:	Andmore Won Meditation Center	**Project No.:**	T2877-22-02
Test Hole No.:	P-2	**Date Excavated:**	9/16/2019
Length of Test Pipe:	60.0 inches	**Soil Classification:**	SP
Height of Pipe above Ground:	6.6 inches	**Presoak Date:**	9/16/2019
Depth of Test Hole:	53.4 inches	**Perc Test Date:**	9/17/2019
Check for Sandy Soil Criteria Tested by:	Weidman	**Percolation Tested by:**	Weidman

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	7:50 AM 8:15 AM	25	25	18.0	0.0	18.0	1.4
2	8:15 AM 8:40 AM	25	50	2.4	0.0	2.4	10.4

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	8:40 AM 8:50 AM	10	10	3.6	0.0	3.6	2.8
2	8:50 AM 9:00 AM	10	20	3.2	0.0	3.2	3.1
3	9:00 AM 9:10 AM	10	30	3.0	0.0	3.0	3.3
4	9:10 AM 9:20 AM	10	40	3.0	0.0	3.0	3.3
5	9:20 AM 9:30 AM	10	50	3.0	0.0	3.0	3.3
6	9:30 AM 9:40 AM	10	60	3.0	0.0	3.0	3.3

Infiltration Rate (in/hr):	11.2
Radius of test hole (in):	5
Average Head (in):	1.5

Figure A-13

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for maximum dry density and optimum moisture content, expansion index, corrosivity, grain size distribution, R-values, and direct shear strength. The results of the laboratory tests are presented on Figures B-1 through B-3.

SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS ASTM D1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% of dry wt.)
TP-5 @ 4-5’	Silty SAND (SM), dark grayish brown	134.0	9.0
TP-7 @ 4-5’	Poorly graded SAND with silt, trace gravel, brown	135.5	7.0

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS ASTM D4829

Sample No.	Moisture Content		After Test Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
TP-5 @ 4-5’	8.0	12.7	120.3	0

SUMMARY OF CORROSIVITY TEST RESULTS

Sample No.	Chloride Content (ppm)	Sulfate Content (%)	pH	Resistivity (ohm-centimeter)
TP-5 @ 4-5’	36	0.000	7.9	10,300

Chloride content determined by California Test 422.
Water-soluble sulfate determined by California Test 417.
Resistivity and pH determined by Caltrans Test 643.

SUMMARY OF LABORATORY R-VALUE TEST RESULTS ASTM D2844

Sample No.	R-Value
TP-1 @ 0-5’	68

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LABORATORY TEST RESULTS

WON MEDITAION CENTER
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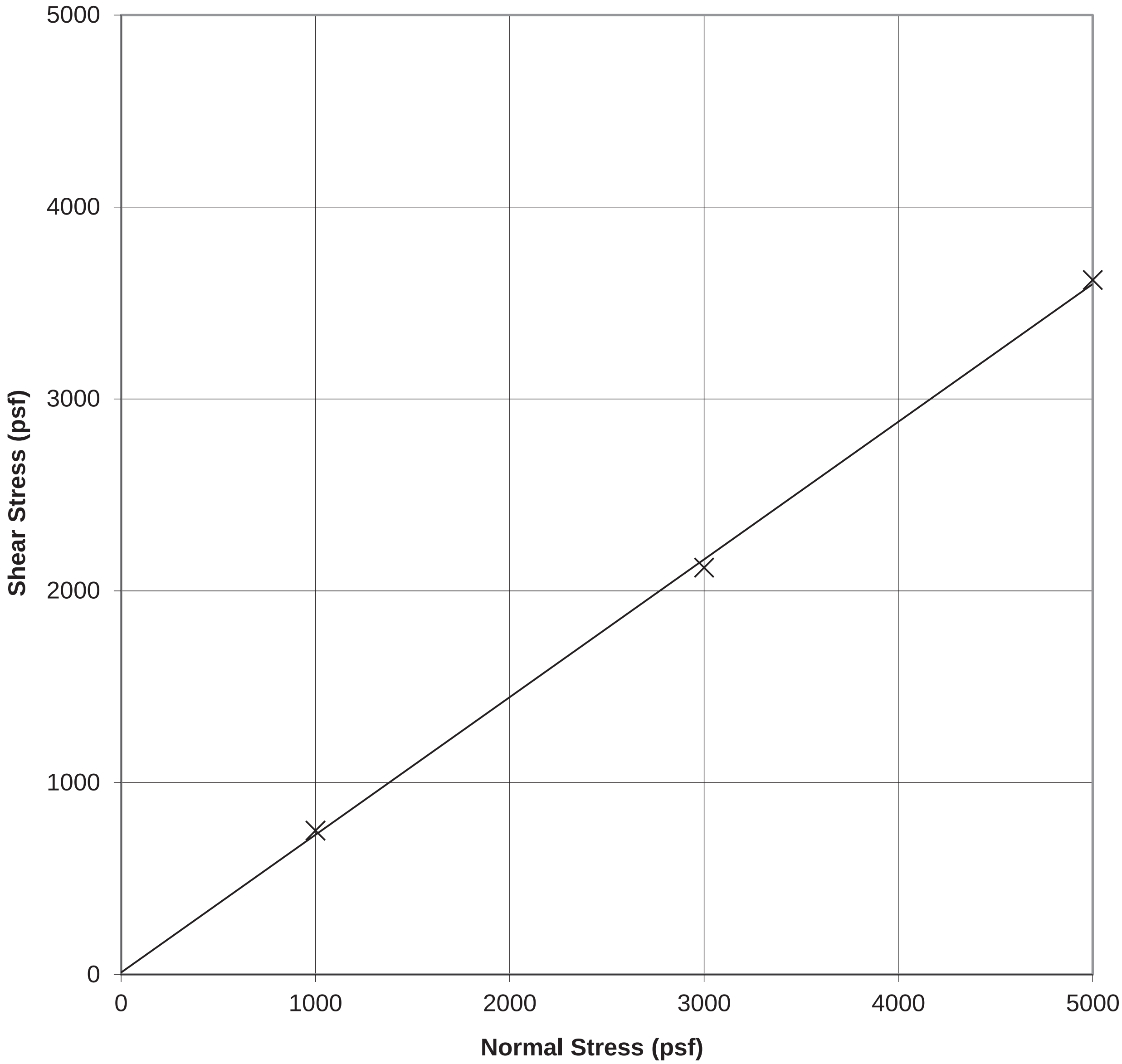
LAKE ELSINORE, CALIFORNIA



SAMPLE ID	SOIL TYPE	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)	C (psf)	ϕ (deg)
*TP-7 @ 4'	SP	122.0	6.9	9.8	230	34

*Sample remolded to approximately 90% of the test maximum dry density at optimum moisture content.

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DIRECT SHEAR TEST RESULTS

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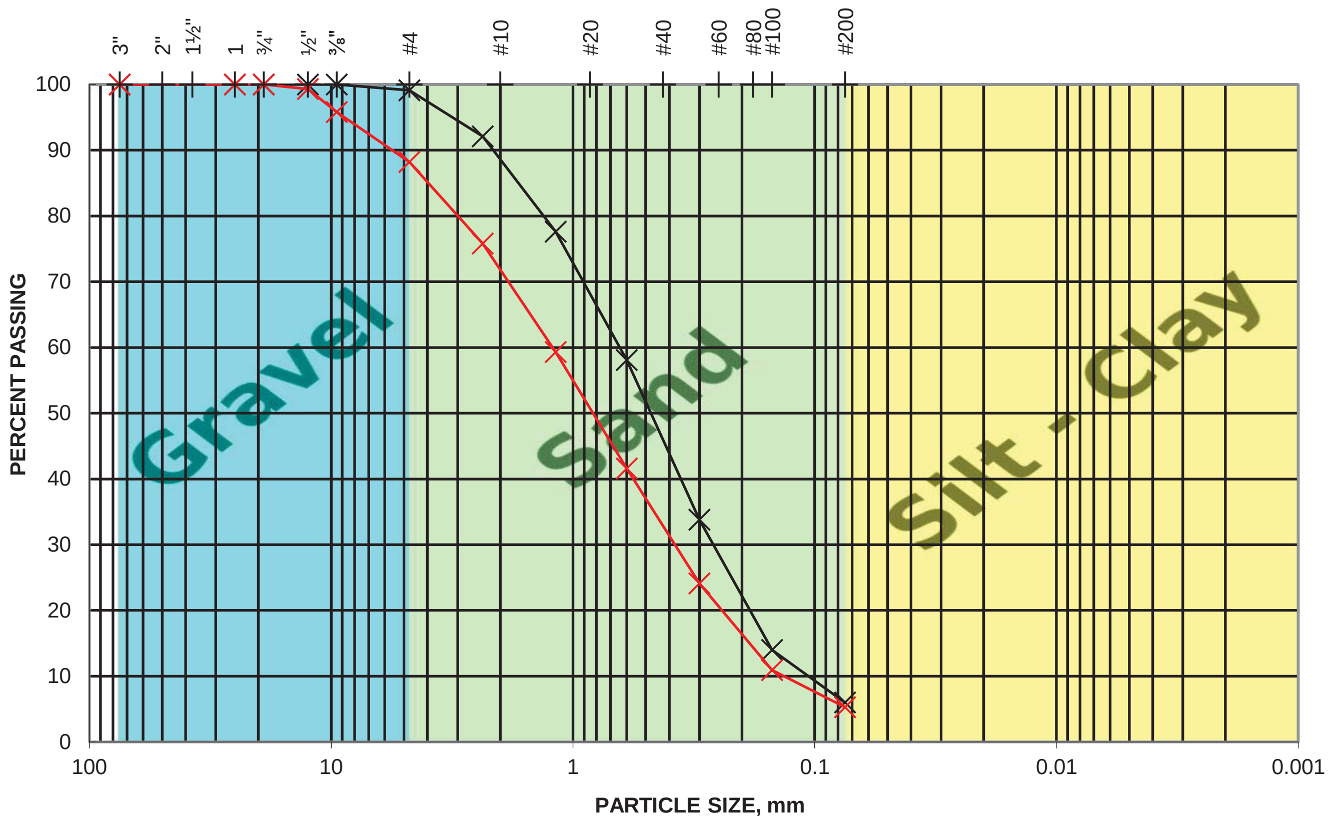
OCTOBER 2019 | PROJECT NO. T2877-22-02 | FIG B-2



SAMPLE ID	SAMPLE DESCRIPTION
P-1 @ 4-5'	SP - Poorly Graded SAND, few silt, trace gravel
P-2 @ 4-5'	SP - Poorly Graded SAND, some gravel, trace silt

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GRAIN SIZE DISTRIBUTION

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OCTOBER 2019	PROJECT NO. T2877-22-02	FIG B-3

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

WON MEDITATION CENTER
19993 GRAND AVENUE
LAKE ELSINORE, CALIFORNIA

PROJECT NO. T2877-22-02

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

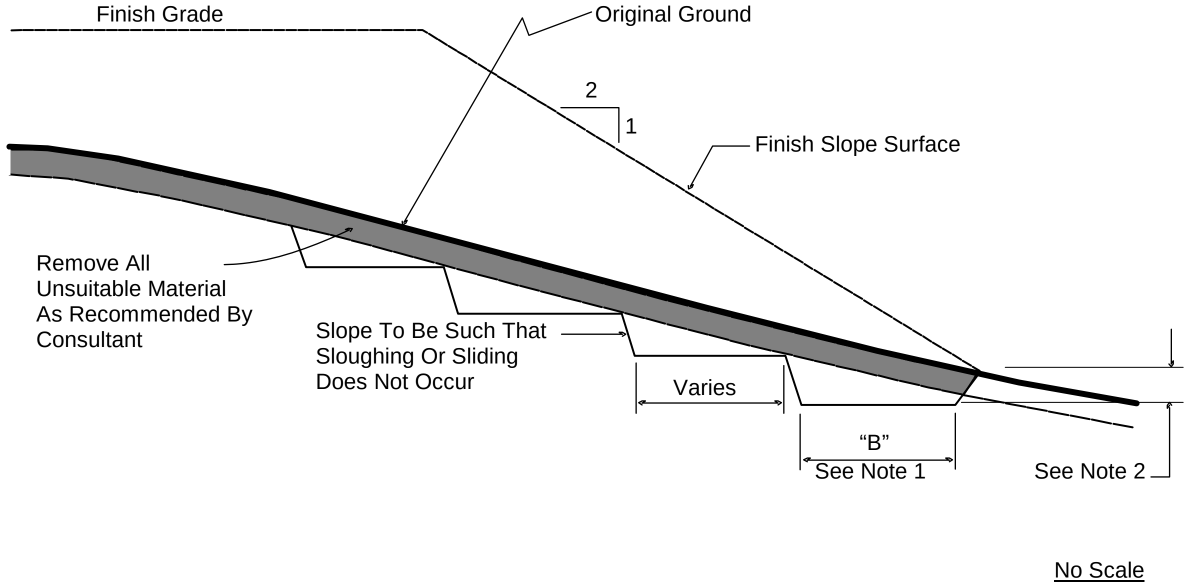
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

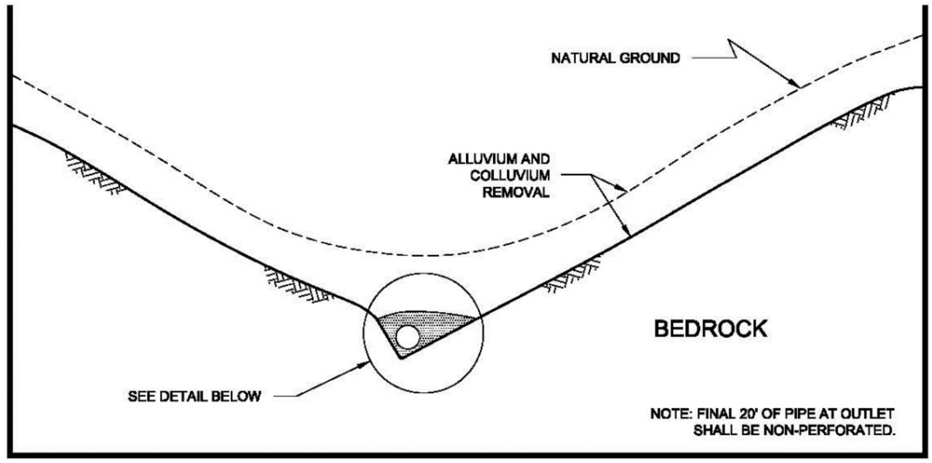
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

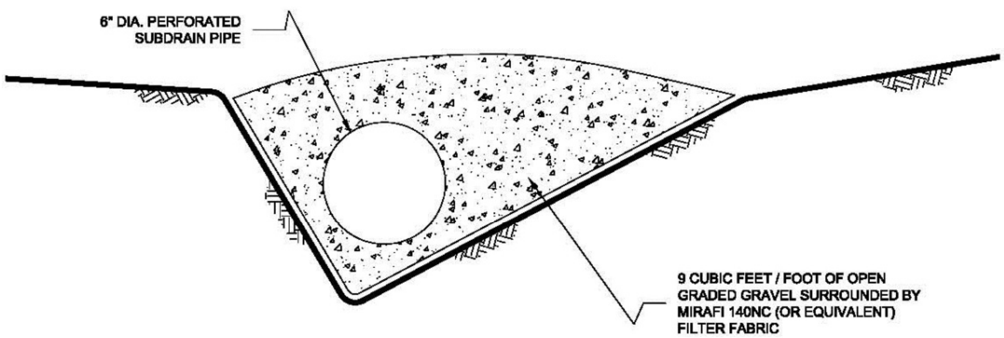
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

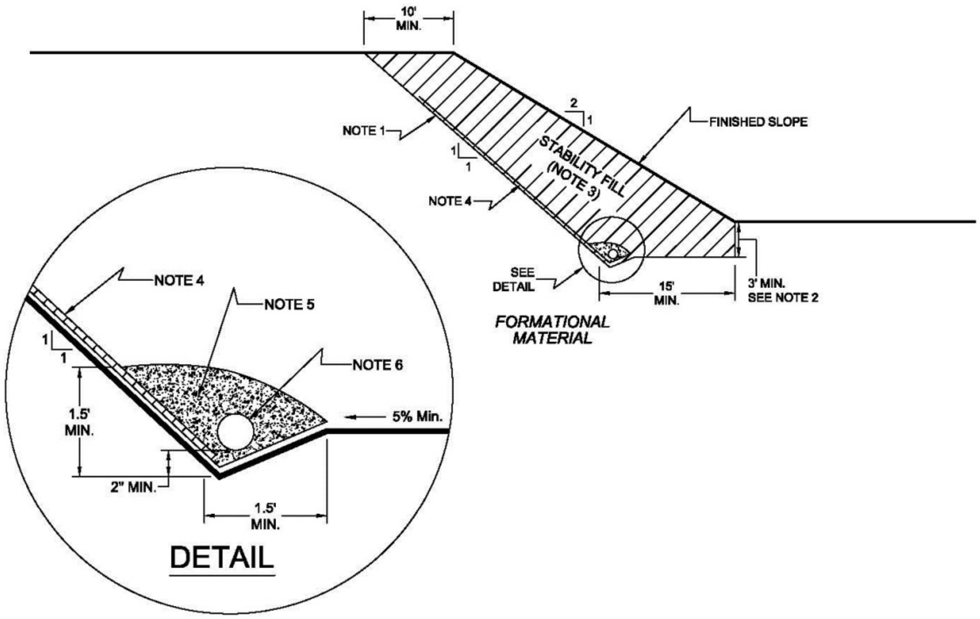
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

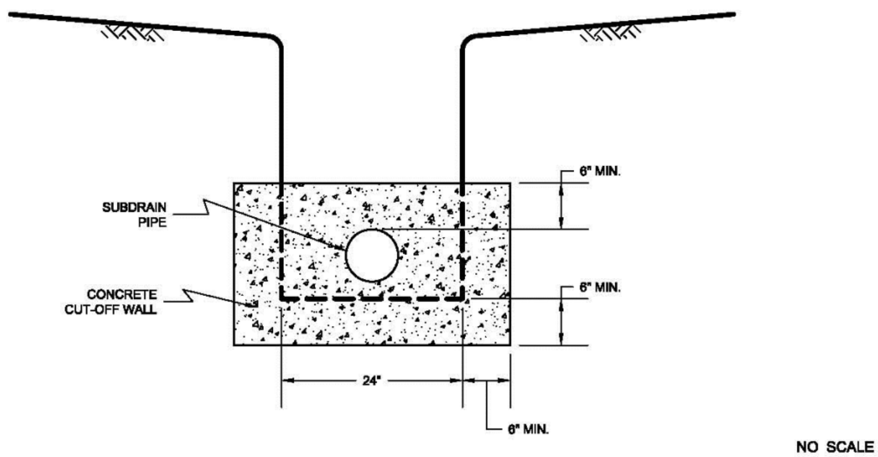
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

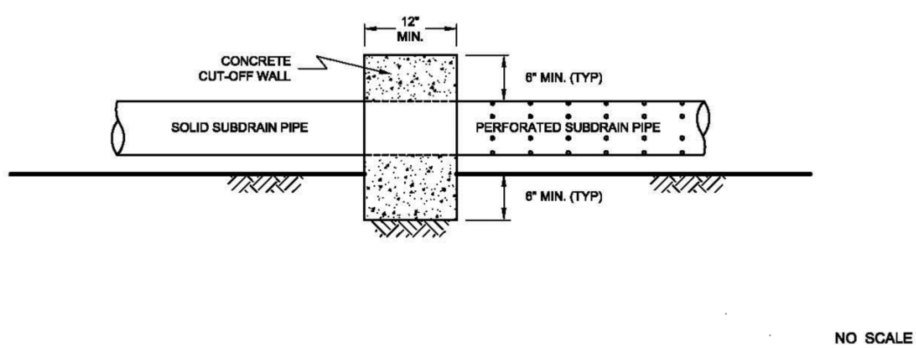
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

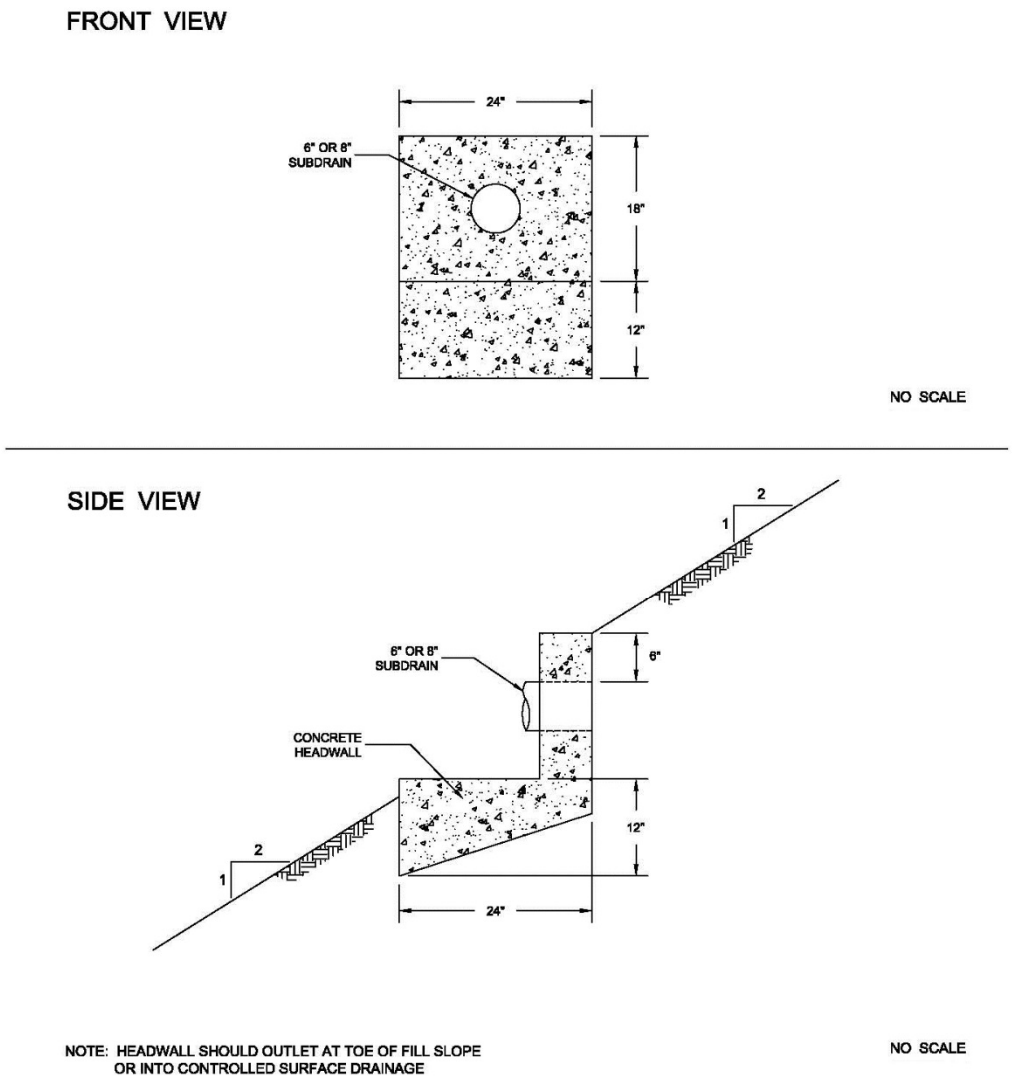


SIDE VIEW



7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*.

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.