soil PACIFIC INC.

Geotechnical and Environmental Services

February 13, 2019
Project No. A-6749-18

Mr. Shahin Motamed
Lake View Center LLC.
18103 Skypark Center # B -2
Irvine Ca 92614

SUBJECT: Soil and Foundation Evaluation Report
Proposed Commercial Buildings
Lots 14-17 APN Numbers; 375 - 092 - 002, 003, 004, 005, & 006
Lake Shore Drive, Lake Elsinore, California

Dear Sir;

Pursuant to your authorization, we are pleased to submit our report for the subject project. Our evaluation was conducted in October 2018. This evaluation consists of field exploration; sub-surface soil sampling; laboratory testing; engineering evaluation and preparation of the following report containing a summary of our conclusions and recommendations.

The opportunity to be of service is appreciated. Should any questions arise pertaining to any portion of this report, please contact this firm in writing for further clarification.

Respectfully submitted,

Soil Pacific, Inc.

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Soil and Foundation Evaluation Report
Proposed Commercial Buildings
Lots 14-17 APN Numbers; 375 - 092 - 002, 003, 004, 005, & 006
Lake Shore Drive, Lake Elsinore, California

Prepared For:

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Lots 14-17 of Lake Shore Drive, Lake Elsinore, California

LIMITATIONS

Between exploratory excavations and/or field testing locations, all subsurface deposits, consequent of their anisotropic and heterogeneous characteristics, can and will vary in many important geotechnical properties. The results presented herein are based on the information in part furnished by others and as generated by this firm, and represent our best interpretation of that data benefiting from a combination of our earthwork related construction experience, as well as our overall geotechnical knowledge. Hence, the conclusions and recommendations expressed herein are our professional opinions about pertinent project geotechnical parameters which influence the understood site use; therefore, no other warranty is offered or implied.

All the findings are subject to field modification as more subsurface exposures become available for evaluations. Before providing bids, contractors shall make thorough explorations and findings. Soil Pacific Inc., is not responsible for any financial gains or losses accrued by persons/firms or third party from this project.

In the event the contents of this report are not clearly understood, due in part to the usage of technical terms or wording, please contact the undersigned in writing for clarification.

SECTION 1.0
PRELIMINARY EVALUATION

1.1 Site Description

The area covered by our investigation consists of a property identified as Lots 14-17 APN Numbers; 375 - 092 - 002, 003, 004, 005, & 006 and located at Lake Shore Drive, Lake Elsinore, California.

The property is comprised of 5 parcels within a mixed used zone of the City of Lake Elsinore. The subject site is located within 500 feet north of Elsinore Lake. The item parcels limited at the north by Ryan Avenue, at the east side by Manning Street, and at the west by Iowa Street (a projected Street). Site access will be provided from Lake Shore Drive.

The parcels are attached and formed a trapezoidal shaped vacant and undeveloped land. The entire property is a part of a south facing descending slope with steeper flank at the north. The slope angle toward the south/southwest becoming more gentle and flattened.

The average elevation of the site is about 1300 feet above the main see level. Maximum height of slope is 90 feet. Property sheet flow is toward south southeast.

1.2 Planned Land Use

It is understood that the proposed construction will consist of commercial buildings. Site plan or conceptual development plan is underway and it is ready to review.

1.3 Field Exploration

A subsurface exploration program was performed under the direction of our staff engineer from SPI in September 2018. The exploration involved the excavation of four (2) exploratory borings (TP-1, TP-2,TP-3 and TP-4). Sub-surface test pit opening was limited to 5-10 feet below grade. The borings were advanced utilizing a backhoe. Earth materials encountered within the exploratory borings were classified and logged by the field engineer in accordance with the visual-manual procedures of the Unified Soil Classification System (USCS), ASTM Test Standard D2488. Following our exploration, borings were loosely backfilled with the soil cuttings. The approximate locations of the exploratory borings are shown on the Exploration Location Map Figure A-1-1. Descriptive boring logs are presented in Appendix A.

1.4 Laboratory Testing

1.4.1. Classification

Soils were classified visually according to the Unified Soil Classification System. Moisture content and dry density determinations were made for the samples taken at various depths in the exploratory excavations. Results of moisture-density and dry-density determinations, together with classifications, are shown on the boring logs, Appendix A.

1.4.2 Expansion

An expansion index test was performed on a representative sample of on-site soils at proposed garde in accordance with the California Building Code. Soil expansion potential at proposed building area have very low or null for potential of expansion (EI=0).

1.4.3 Direct Shear

Shear strength parameters are determined by means of strain-controlled, double plain, direct shear tests performed in general accordance with ASTM D-3080. Generally, three or more specimens are tested, each under a different normal load, to determine the effects upon shear resistance and displacement, and strength properties such as Mohr strength envelopes . The direct shear test is suited to the relatively rapid determination of consolidated drained strength properties because the drainage paths through the test specimen are short, thereby allowing excess pore pressure to be dissipated more rapidly than with other drained stress tests. The rate of deformation is determined from the time required for the specimen to achieve fifty percent consolidation at a given normal stress. The test can be made on all soil materials and undisturbed, remolded or compacted materials. There is however, a limitation on maximum particle size. Sample displacement during testing may range from 10 to 20 percent of the specimen's original diameter or length.

The sample's initial void ratio, water content, dry unit weight, degree of saturation based on the specific gravity, and mass of the total specimen may also be computed. The shear test results are plotted on the attached shear test diagrams and unless otherwise noted on the shear test diagram, all tests are performed on undisturbed, saturated samples.

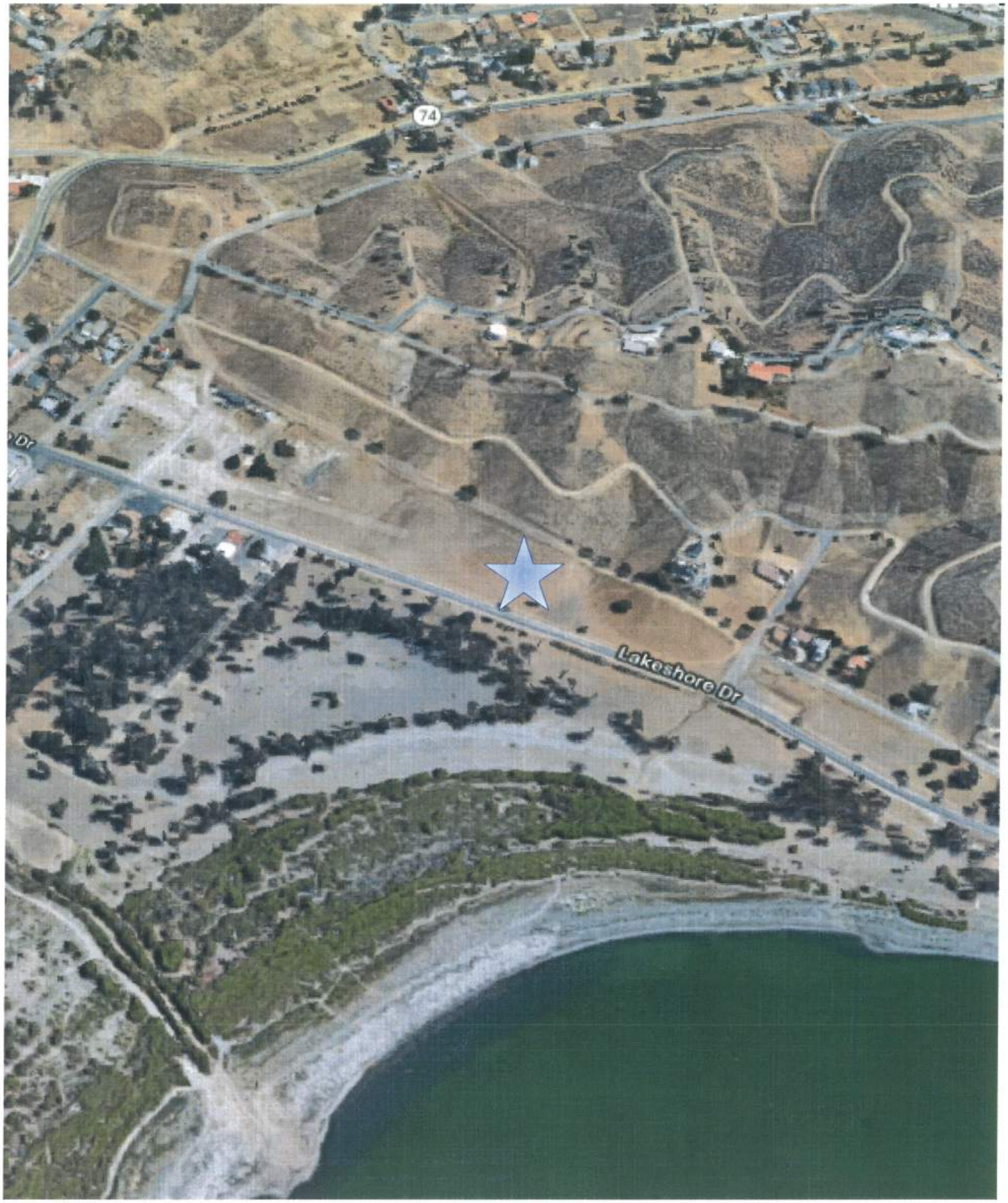


Fig. 1: Site aerial photo.

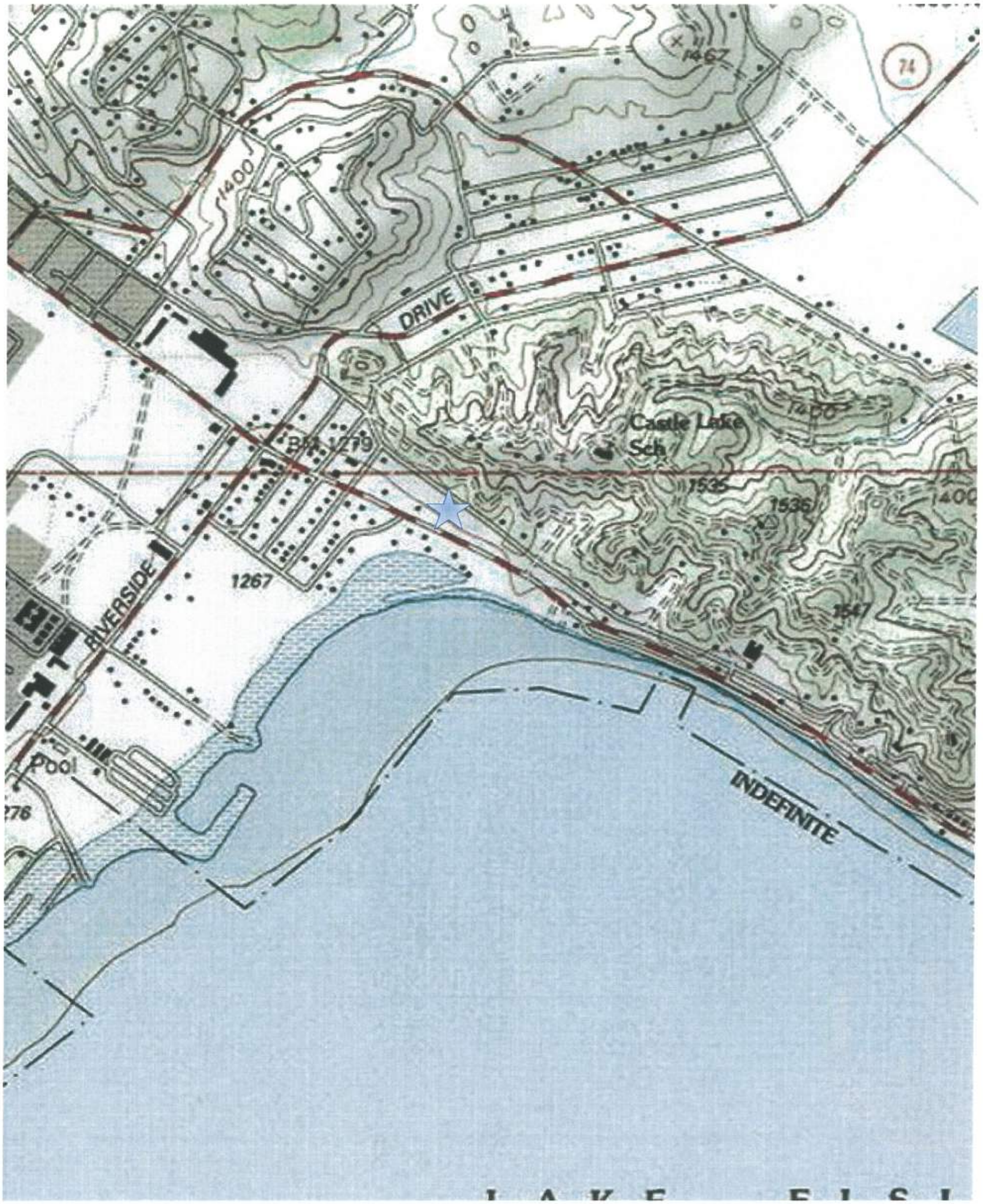


Figure 2: Site Topographic Map (USGS AAGS)

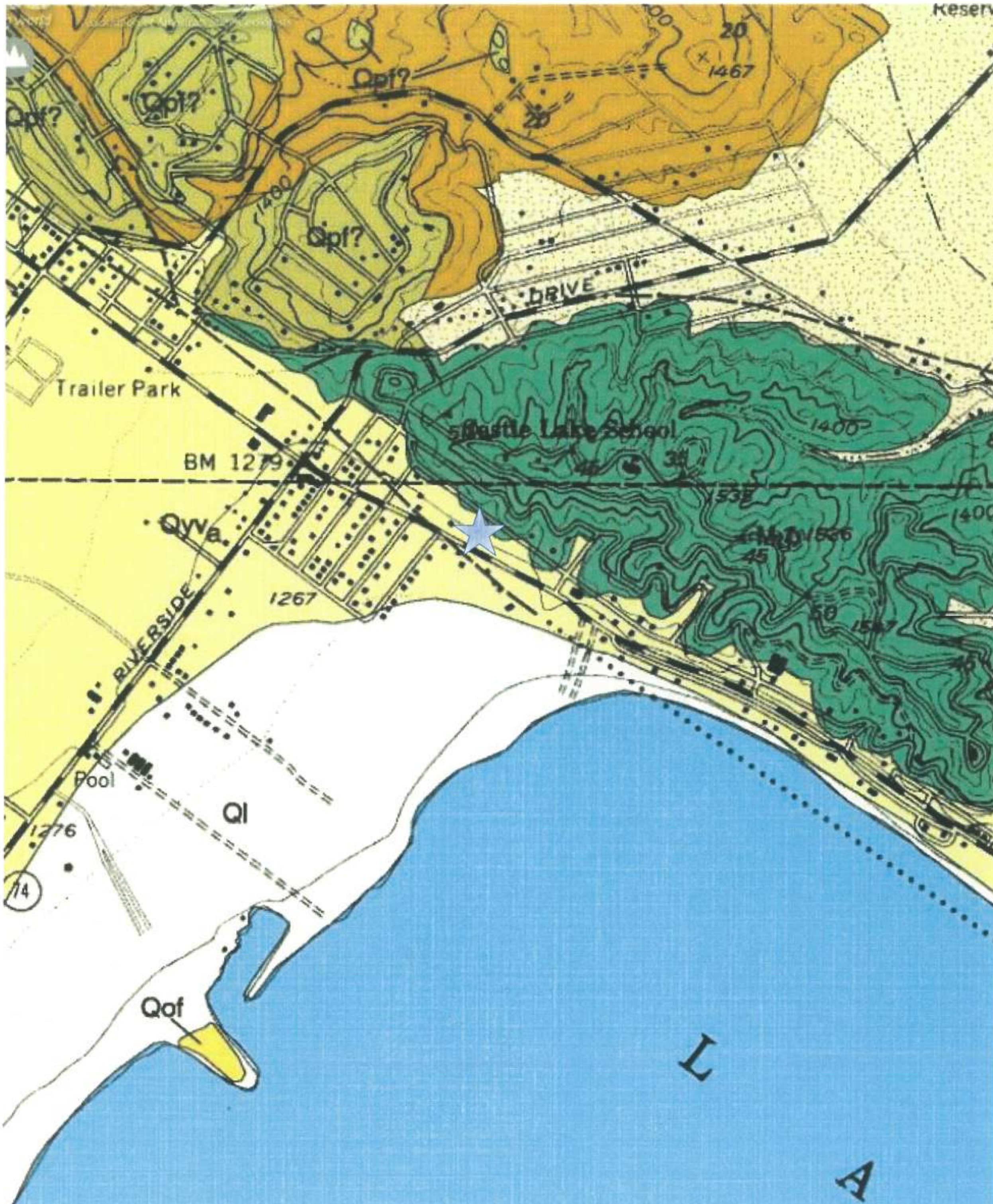


Figure 3: Site geological map.

Section 2.0
Conclusions

The proposed construction is considered feasible from a soils engineering standpoint. All earthwork should be performed in accordance with applicable engineering recommendations presented herein or applicable Agency Codes, whichever are the most stringent.

2.1 Site Condition

Based on the "Topographic Plan" outline, slopes of the ribs are inclined between 3.0:1 (horizontal to vertical) and 3.5:1; with the slope through the southwesterly part of the parcel inclined between 8.0:1 and 14.0:1. A southwest descending drainage course, from Ryan Ave. to Lakeshore Dr., crosses the parcel at about mid -point. An undocumented artificial fill has been constructed to provide access of Ryan Ave. across the drainage, with a culvert being installed through the fill to allow draining waters to pass.

2.2 Regional Geology

The subject parcel is in the northern part of the Peninsular Range Province. Morton, D.M. and Weber, F.H., 1998, "Geologic Map of the Elsinore 7.5' Quadrangle, Riverside County, California, USGS Open-File Report 03-281", indicates the northern part of the subject parcel is underlain by Mesozoic age (Mzp) black and fissile phyllite with foliation planes striking to the northeast and dipping moderately to the southeast. The Mortom map identifies that the southern part of the parcel is underlain by Holocene to Late Pleistocene age (Qyva), sand, silt, and clay unconsolidated, alluvial deposits.

Morton and Weber mapping places the Glen Ivy and Willard Faults, both with Quaternary activity, at 500 feet southwest and 1.5 miles southwest, respectively, of the subject parcel. CDMG, 1975, "Fault Map of California" locates traces of the San Jacinto and San Andreas Faults (both having Holocene displacements) being, respectively, 20.2 and 33.3 miles northeast of the parcel; The CDMG map places the Banning and Newport-Inglewood Faults (both with Quaternary displacements), respectively, at 27.4 miles northeast and 26.8 miles southwest of the site. Additionally, the CDMG map plots thermal springs at Lakeshore Drive 1.1 miles southeast of the parcel.

2.3 Site Geology

Our Engineering Geologist has utilized the following sources of information to establish the geologic conditions at the subject property: the geologic map by Morton and Weber, referenced above; Google Earth air photos; CDMG and CGS publications; the "Geotechnical Plan", referenced above; and reconnaissance mapping of exposures, and logging of five (5) borings, when accompanying Soil Pacific for the site investigation on 29 January 2019.

Please refer to plt plan A–1-1 and four cross section A-A', B-B', C-C' and D-D' developed across the site arbitrarily to show the position of the bedrock throughout the site, proposed development and position of proposed building after the site grading.

Four geologic units are exposed, encountered in the borings, and mappable on the subject parcel: (Qaf); alluvium and colluvium (Qal/Qcol) – which is correlative with Morton's alluvium (Qyva); a phyllite, or slate, bedrock (Mzp): and an igneous bedrock (Khg).

Artificial fill

Artificial fill (Qaf) is located at the drainage course, and in the upper part of borings B-1and B-4, along Ryan Ave. The toe of the fill at the drainage course, and culvert, on the south side of the dirt road, may encroach on the parcel.

Alluvium

The alluvium and colluvium (Qal/Qcol) are found in the western and southwestern parts of the parcel; and were encountered in borings B-3, -4 and, -5. In B -3 and -4, the alluvium is a clay (cl) of five (5) and three (3) feet thickness, respectively. With B-5, a clay (cl) alluvium is found between ground surface and 15 feet below ground surface (bgs); and between 15 and 20 feet bgs the alluvium is a clayey gravel (gc).

Bedrock

The phyllite/slate bedrock (Mzp) is exposed in cuts and encountered in the borings along Ryan Ave.; and in boring B-4. The cuts provided measurements of the foliation planes striking between N. 66- and 75-degrees E. with moderate to steep dips to the southeast – these attitudes being conformable to the Morton mapping. In the borings B-1, B-3, and B-4, the phyllite underlies the artificial fill, alluvium, or colluvium; in B-2, the phyllite is found at ground surface. Additionally, in borings B-1 through B-3, the phyllite can be mapped as weathered, slightly weathered, or relatively fresh (refer to cross section A-A').

At boring B-5, a highly weathered/decomposed, igneous granitic bedrock was encountered underlying the alluvial deposits. This unit is most probable correlative with Morton's Cretaceous age granitics (Khg) that crop out near the subject parcel.

Considering the mappable condition of the phyllite bedrock (Mzp) through the parcel, and without additional site-specific, subsurface information, the contact between the phyllite (Mzp) and igneous (Khg) bedrock is interpreted to be a nonconformity.

2.4 Foundations

All newly designed isolated pad or continuous foundation must be embedded into firm and approved bedrock formation in order to avoid a cut and fill transition.

2.5 Bearing Materials

The existing alluvial soils where discovered by exploring with test pit were mainly porous and or subject to settlement by time. These materials are not an appropriate bearing materials. Therefore, proper site grading, engineering and construction of buttress may be required. As an alternative pile foundation may be designed to support the proposed building.

2.6 Groundwater

The site is located within the buffer zone of Lakeshore Valley Basin (California Department of Water Resources, [CDWR], 2018). Groundwater depth varies within the area and flow direction beneath the subject site is toward the south-southwest. No groundwater wells were listed on the property; however, several groundwater wells are listed in the site vicinity.

During our investigation, groundwater was not encountered within 10 feet of sub-surface exploration below the existing grade. The depth of groundwater may fluctuate depending upon the time and period of the year.

2.7 CBC Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration, which may be determined from the site-specific acceleration response spectrum. To provide the design team with the parameters necessary to construct the site-specific acceleration response spectrum for this project, we used two computer applications that are available on the United States Geological Survey (USGS) website, http://geohazards.usgs.gov/.

Specifically, the Design Maps website http://geohazards.usgs.gov/designmaps/us/application.php was used to calculate the ground motion parameters. 2008 PSHA Interactive Deaggregation website http://geohazards.usgs.gov/deaggint/2008/ can be used to determine the appropriate earthquake magnitude if specific design is required.

The printout attached in Appendix C provides parameters required to construct the site-specific acceleration response spectrum based on the 2016 CBC guidelines.

It should be noted that the subject property is not in an Alquist-Priolo Earthquake Fault Zones; nor are any faults mapped, or inferred, through the property. CGS, 2016, "Earthquake Shaking Potential for California, Map Sheet 48", suggest the degree of ground shaking at the property, due to earthquakes, will 60% to > 70% of gravity; but the degree of shaking at the site will be no greater than shaking at neighboring properties.

2.8 Chemical Contents

Chemical testing for detection of hydrocarbon or other potential contamination is beyond the scope of this report.

2.9 Liquefaction Study/ Secondary Seismic Hazard Zonation

Subject site is underlain by firm and dense bedrock and the potential for Liquefaction susceptibility is null. Liquefaction occurs when seismically-induced dynamic loading of a saturated sand or silt causes pore water pressures to increase to levels where grain-to-grain contact pressure is significantly decreased and the soil material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, settlement and tilting of engineered structures, flotation of buoyant buried structures and fissuring of the ground surface. A common manifestation of liquefaction is the formation of sand boils (short-lived fountains of soil and water emerges from fissures or vents and leave freshly deposited conical mounds of sand or silt on the ground surface). Lateral spreading can also occur when liquefaction occurs adjacent to a free face such as a slope or stream embankment.

The types of seismically induced flooding that may be considered as potential hazards to a particular site normally includes flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site has an average elevation of approximately 200 feet above sea level, and since it does not lie in close proximity to an enclosed body of water, the probability of flooding from a tsunami or seiche is considered to be low. In addition, the site is not located within a designated tsunami inundation area.

Section 3.0
Recommendations

Based on our exploration, and experience with similar projects, the proposed construction is considered feasible from a soils engineering standpoint providing the following recommendations are made part of the plans and are implemented during construction.

3.1 Clearing and Site Preparation

Site grading is anticipated and is proposed. The following typical recommendations can be used by the project civil engineer in charge of grading plan preparation.

1. The areas to receive compacted fill should be stripped of all vegetation, construction debris and trashes, non engineered fill, left in place incompetent material up to approved soils. If soft spots are encountered, a project soil engineer will evaluate the site conditions and will provide necessary recommendations.

2. The excavated areas bottom should be scarified to a minimum of 8 inches, adjusted to optimum moisture content, and reworked to achieve a minimum of 90 percent relative compaction. Overexcavation within equal to depth of excavation or 5 feet. Excavation against adjacent buildings or public way require shoring or slot cut method.

3. Compacted fill should extend at least 5 feet beyond all perimeter footings or to a distance equal to the depth of the certified compacted fill, whichever is the greatest and feasible.

4. Compacted fill, consisting of on-site soil shall be placed in lifts not exceeding 6 inches in uncompacted thickness. The excavated onsite materials are considered satisfactory for reuse in the fill if the moisture content is near optimum. All organic material and construction debris should be removed and shall be segregated. Any imported fill should be observed, tested, and approved by the soils engineer prior to use as fill. Rocks larger than 6 inches in diameter should not be used in the fill.

5. The fill should be compacted to at least 90 percent of the maximum dry density for the material. The maximum density should be determined by ASTM Test Designation D 1557-00.

6. Field observation and compaction testing during the grading should be performed by a representative of Soil Pacific Inc. to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compaction effort should be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent relative compaction is obtained.

3.2 Site Preparation and Excavations

If any unanticipated subsurface improvements (pipe lines, irrigation lines, etc.) are encountered during earthwork construction, this office should be informed and appropriate remedial recommendations would subsequently be provided. During earthwork construction, all remedial removals, and the general grading and construction procedures of the contractor should be observed, and the fill selectively tested by a representative of this office. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and if warranted, additional recommendations will be offered.

3.3 Stability of Temporary Cuts

The stability of temporary cuts required during the removal process depends on many factors, including the slope angle, the shearing strength of the underlying materials, the height of the cut, and the length of time the excavation remains open and exposed to equipment vibrations and rainfall. The geotechnical consultant should be present to observe all temporary excavations at the site. The possibility of temporary excavations failing may be minimized by:

1) keeping the time between cutting and filling operations to a minimum;
2) limiting excavation length exposed at any one time; and,
3) cutting no steeper than a 1: 1 (h:v) inclination for cuts in excess of 4 feet in height.
4) or shoring prior to cut.

Any southwest, or southeast, - facing excavation in the phyllite bedrock (Mzp) will undercut an apparent dip of the unit's foliation planes. Any excavation steeper than 2:1 in artificial fill (Qaf), alluvium/colluvium (Qal/Qcol), or weathered phyllite bedrock (Mzp) will leave these units unsupported, and subject to possible failure. Therefore any unsupported cut, grading must be shored prior to excavation.

3.4 Foundations

The following recommendations may be used in preparation of the design and construction of the foundation system.

3.4.1 Bearing Value

The allowable bearing value for conventional footings, having a minimum width of 18 inches and a minimum embedment of 24 inches embedded into approved competent soils should not exceed 3000 pounds per square foot. This value may be increased by one-third for short duration (wind or seismic) loading.

3.4.2 Isolated Square Pad Footings

The proposed structure can be adequately supported by shallow spread footing or isolated footings. The minimum embedment for individual pad footings should be 24 inches below the approved materials. Allowable bearing value is 3000 psf to a maximum of 6000 psf. The bearing value may be increased by 1/3 when considering short duration seismic or wind loads.

3.4.3 Foundation Settlement

Based upon anticipated structural loads, the total settlement for the proposed foundation is not expected to exceed 1 inch at design load. Differential settlement between adjacent footings and lateral displacement of lateral resisting elements should not exceed .5 inch.

3.4.4 Concrete Type

In lack of soluble sulfate test, Type V concrete should be used.

3.4.5 Slabs-on-grade

1) Residential floor slabs cast on-grade may be used for the interior of the residence and shall be at least four inches (4") in thickness, superseding the presumptive minimum set forth in 2016 CBC, Section 1907.1. All concrete shall be designed in accordance with ACI 318 and placed in accordance with ACI 302. If desired, to minimize the risk of shrinkage cracking and/or vapor transmission, the use of concrete with a water-cement ratio of less than 0.5 by weight, a one-inch top-sized aggregate, and/or a minimum compressive strength of 4000 PSI may be considered.

2) As a minimum standard, all slabs shall be reinforced with number four (#4) bars, sixteen inches (16") on center in both directions. Notwithstanding the minimum reinforcement prescribed herein, the Project Structural Engineer shall design all slabs in full accordance with requirements and procedures contained within appropriate design standard . Care should be taken by the contractor to ensure that the steel reinforcement is cast near the center of the slab and/or in accordance with the recommendations of the Project Structural Engineer.

3) All slabs shall be underlain by at least four inches (4") crushed rock per Calgreen with a minimum 15-mil polyolefin membrane vapor barrier placed below all living areas, superseding the presumptive minimum set forth in 2016 CBC, Section 1907.1. It is recommended that the specific vapor barrier used conform to the specifications of ASTM E1745 (such as Stego Wrap or similar) and be placed in full conformance with the manufacturer's installation guidelines. The engagement of a qualified expert in the mitigation of moisture vapor transmission for the design of the slab foundation system is recommended.

4) In accordance with 2016 CBC Section 1808.7.4, to reduce the risk of interior flooding the top of all slab-foundations shall extend a minimum of twelve inches (12") above the elevation of the adjacent drainage path. Reductions to this prescribed minimum elevation are subject to specific approval of the local Building Authority.

3.4.6 Pile Foundation

Shoring Piles

For shoring purposes, drilled cast-in-place soldier piles or I beams should be placed at 8 feet on center along the edge of the top of the slope. The minimum diameter of the piles is 18 inches. For design purposes, an allowable passive value for the soils below the bottom plane of excavation, may be assumed to be 500 pounds per square foot per foot of depth, up to a maximum of 5,000 pounds per square foot. In order to control the anticipated deflection of the piles the shoring piles may be braced by two row of rakers per structural engineer justification if deemed necessary

The frictional resistance between the soldier piles and retained soil may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.4, based on uniform contact between the concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. Pile or I beams should have a minimum of same depth of embedment into the ground below the lowest excavation grade. The combined static and seismic active will be applied in design criteria. The seismic pressure may be assumed to be in order of 40 pcf for all piles having 30-40 feet in height. Seismic pressure of 40 pcf will be added for any pile having 6 feet or higher. Pile should be designed for static pressure of 42 pcf.

Lagging

Lagging between soldier piles could be omitted within the cohesive soils. In the less cohesive soils, such as the sands and gravels, lagging would be necessary. It is recommended that the exposed soils be observed by the soils engineer to verify the cohesive nature of the soils and the area where lagging may be omitted.

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the soils, the pressure on the lagging will be somewhat less. It is recommended that the lagging be designed for the full design pressure but be limited to a maximum of 400 pounds per square foot. Water should not be allowed to pond on top of the excavation nor to flow towards it.

3.5 Utility Trench Backfill

Utility trenches backfill should be placed in accordance with Appendix D. It is the owners' and contractors' responsibility to inform subcontractors of these requirements and to notify Soil Pacific Inc when backfill placement is to begin.

3.6 Seismic Design and Construction

Construction should be in conformance with seismic design parameters of the latest edition of California Building Code (C.B.C.). Please refer to the Appendix C for closest faults and other related seismic design parameters.

3.7 Surface and Sub-surface Drainage Provisions

Proper surface drainage gradients are helpful in conveying water away from foundations and other improvements. Subsurface drainage provisions are considered essential in order to reduce pore-pressure build-up behind retaining structures. Ponding of water enhances infiltration of water into the local soils, and should not be allowed anywhere on the pad.

3.8 Conventional Retaining Wall

Retaining wall is not proposed or planned. If a conventional retaining wall is planned, the following design criteria may be used:

1) Where a free standing structure is proposed, a minimum equivalent fluid pressure, for lateral soil loads, of 40 pounds per cubic foot, may be used as design for onsite non-expansive granular soils conditions and level backfill (10:1 or less). If the wall is restrained against free movement (= ±1 % of wall height) then the wall should be designed for lateral soil loads approaching the at-rest condition. Thus, for restrained conditions, the above value should be increased to 80 pcf. In addition, all retaining structures should include the appropriate allowances for any anticipated surcharge loads.

2) An allowable soil bearing pressure of 3000 lbs. per square foot may be used in design for footings embedded to a minimum of 24 inches below the lowest adjacent competent grade.

3) A friction coefficient of 0.40 between concrete and natural or compacted soil and a passive bearing value of 500 lbs. per square foot per foot of depth, up to a maximum of 5,000 pounds per square foot at the bottom excavation level may be employed to resist lateral loads.

Back drain system will consist of a free-draining material made up of at least 1 cubic foot of 3/4-inch crushed rock/gravel around pipe drains. If an open space greater than 1 foot exists between the back of the wall and the soil face, gravel backfill should be compacted by vibration. An impervious soil cap should be provided at the top of the wall backfill to prevent infiltration of surface water into the back drain system. The cap may be a combination of concrete and/or compacted fine grained soils. The compacted backfill soil cap should be at least 1 foot thick when used in conjunction with a concrete slab type cap and at least 2 feet thick when used exclusively.

Any surcharges such as traffic and adjacent building loads shall be computed and adhered into the design by the structural engineer justification.

3.9 Concrete Driveway

1. The subgrade soils for all flatwork should be checked to have a minimum moisture content of 2 percentage points above the optimum moisture content to a depth of at least 18 inches.

2. Local irrigation and drainage should be diverted from all flatwork areas. Area drains and swales should be utilized to reduce the amount of subsurface water intrusion beneath the foundation and flatwork areas. Planter boxes adjacent to buildings should be sealed on the bottom and edges to retard intrusion of water beneath the structure.

3. The concrete flatwork should have enough cold joints to prevent cracking. Adequate reinforcement considering the expansion potential is required. A minimum of rebar no. 3 placed at 18 inches on center most be used.

4. Surface and shrinkage cracking of the finished slab may be significantly reduced if a low slump and water-cement ratio is maintained during concrete placement. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking.

5. Construction joints and saw cuts should be designed and implemented by the concrete contractor or design engineer based on the medium expansive soil conditions. Maximum joint spacing should not exceed 8 feet in any direction.

6. Patio or driveway subgrade soil should be compacted to a minimum of 90 percent to a depth of 18 inches. All run-off should be gathered in gutters and conducted off site in a non-erosive manner. Planters located adjacent to footings should be sealed and leach water intercepted.

3.10 Storm Water Management

On-site sheet water can be infiltrate within an area along the Lakeshore Drive where the tick alluvial deposit is identified. The infiltration rate of 5 inches per hour can be used to disburse the sheet water.

3.11 Observation and Testing

It is recommended that **Soil Pacific Inc.** be present to observe and test during the following stages of construction:

❒ Site grading to confirm proper removal of unsuitable materials and to observe and test the placement of fill.

❒ Inspection of all foundation excavations prior to placement of steel or concrete.

❒ During the placement of retaining wall sub-drain and backfill materials.

❒ Inspection of all slab-on-grade areas prior to placement of sand, Visqueen.

❒ After trenches have been properly backfilled and compacted.

❒ When any unusual conditions are encountered.

APPENDIX A

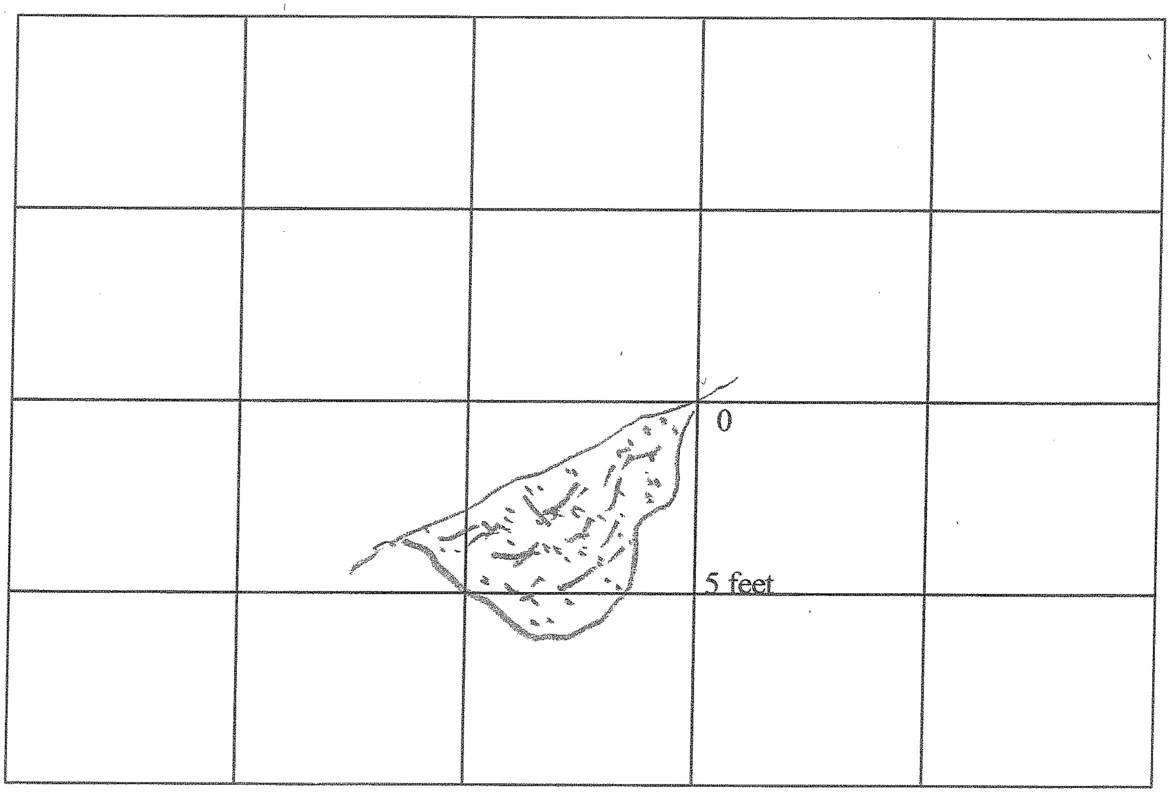
Field Exploration

RECORD OF SUBSURFACE

TEST PIT NO. TP-1

Date: 10/6/18	Logged by: A.Sh.	Equipment:Backhoe

Elev.	USCS	Moist %	Dry Dens. pcf	Depth feet	Description of Earth Materials
	SM	2.2	92.1	1	Brown, dark brown fine to pebbely clastes and bedrock fragment, with clayey matrix, porous and loose when get wet. top soil/slope wash.
	SM	4.0	98.8	3	Redish brown coarse grained to pebelly clasts, slightly porous, native/ Qcol.
	SM			5	
				7	
				9	
				11	End of subsurface exploration 6 feet. Test pit backfilled.
				13	



CROSS SECTION SKETCH

Scale: 1"= 5 feet

Trench Orientation: N-S

SOIL PACIFIC INC.
Geotechnical and Environmental Services

Project Name: Lot 14-17 of Lakeshore Drive, Lake Elsinore

Project Number: A-6649-18

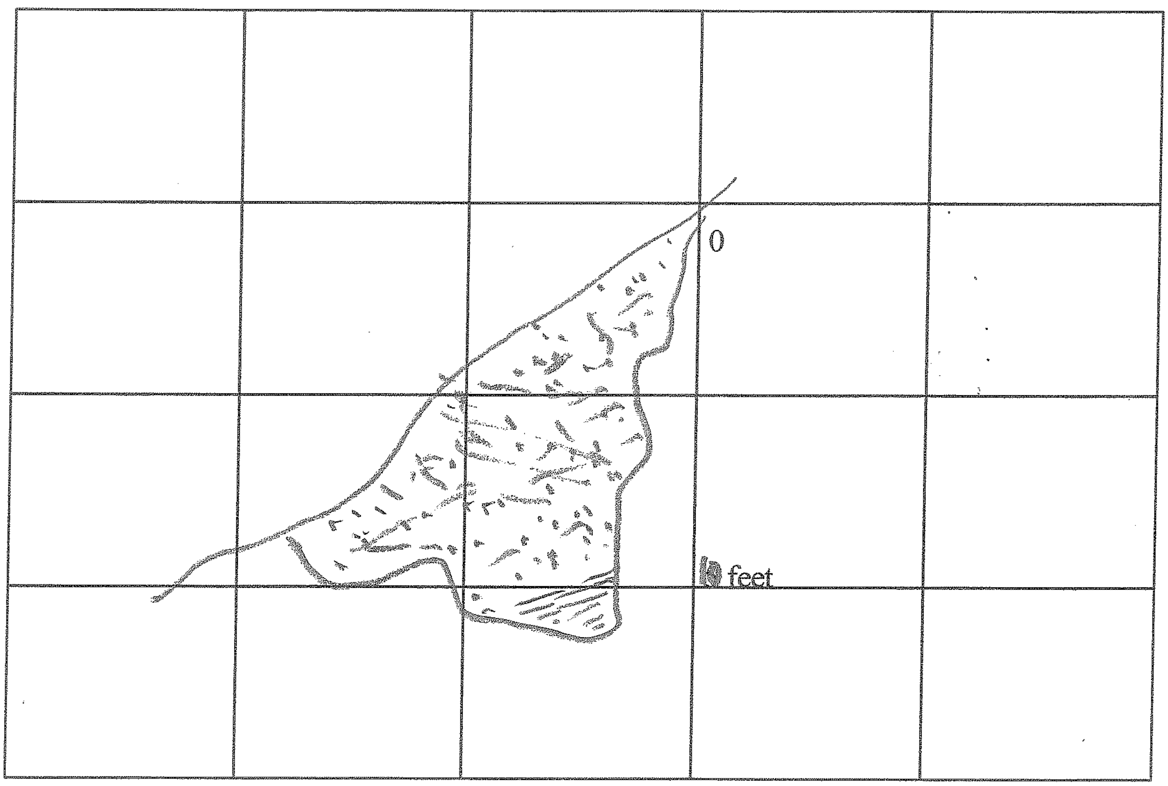
Figure:

RECORD OF SUBSURFACE

TEST PIT NO. TP-2

Date: 10/6/18	Logged by: A.Sh.	Equipment:Backhoe

Elev.	USCS	Moist %	Dry Dens. pcf	Depth feet	Description of Earth Materials
	SM			1 3	Brown, dark brown fine to pebbley clastes and bedrock fragment, with clayey matrix, porous and loose when get wet. top soil/slope wash.
	SM			5	Redish brown coarse grained to pebblley clasts, slightly porous, native/ Qcol.
	SM			7 9	Redish brown, fine to coarse garined silty sand with some clay and scattered clastes and some larger pebbles, moderately dense.
				11 13	At 10 feet fine to coarse grained clayey sand, damp.
					End of subsurface exploration 10 feet. Test pit backfilled.



CROSS SECTION SKETCH

Scale: 1"= 5 feet

Trench Orientation: N-S

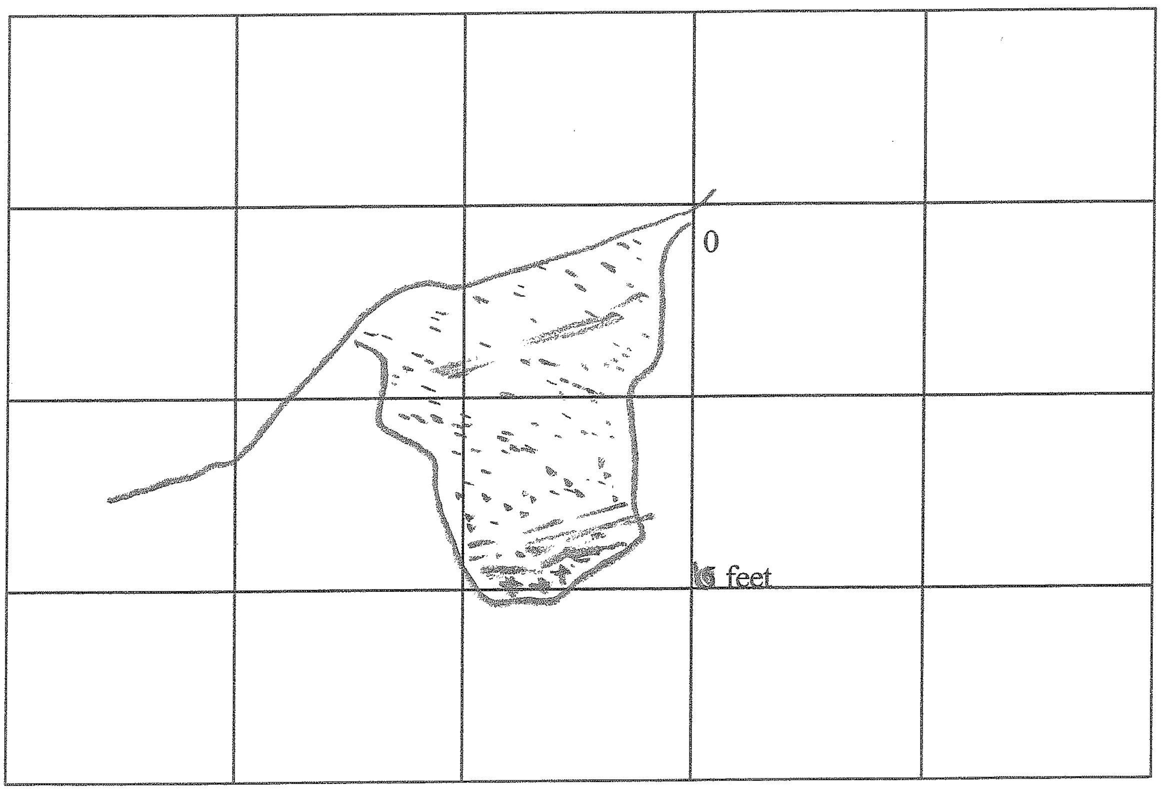
SOIL PACIFIC INC. Geotechnical and Environmental Services	Project Name: Lot 14-17 of Lakeshore Drive, Lake Elsinore	
	Project Number: A-6649-18	Figure:

RECORD OF SUBSURFACE

TEST PIT NO. TP-3

Date: 10/6/18	Logged by: A.Sh.	Equipment:Backhoe

Elev.	USCS	Moist %	Dry Dens. pcf	Depth feet	Description of Earth Materials
	SM			1 3	Brown, dark brown fine to pebbley clastes and bedrock fragment, with clayey matrix, porous and loose when get wet. top soil/slope wash.
	SM			5	Redish brown coarse grained to pebblley clasts, slightly porous, native/ Qcol.
	SM			7 9	Redish brown, fine to coarse garined silty sand with some clay and scattered clastes and some larger pebbles, moderately dense.
	Bedrock			11 13	At 9 feet gray, dark gray hard siliceous bedrock. refusal to explore deeper. End of subsurface exploration 9 feet. Test pit backfilled.



CROSS SECTION SKETCH

Scale: 1"= 5 feet

Trench Orientation: N-S

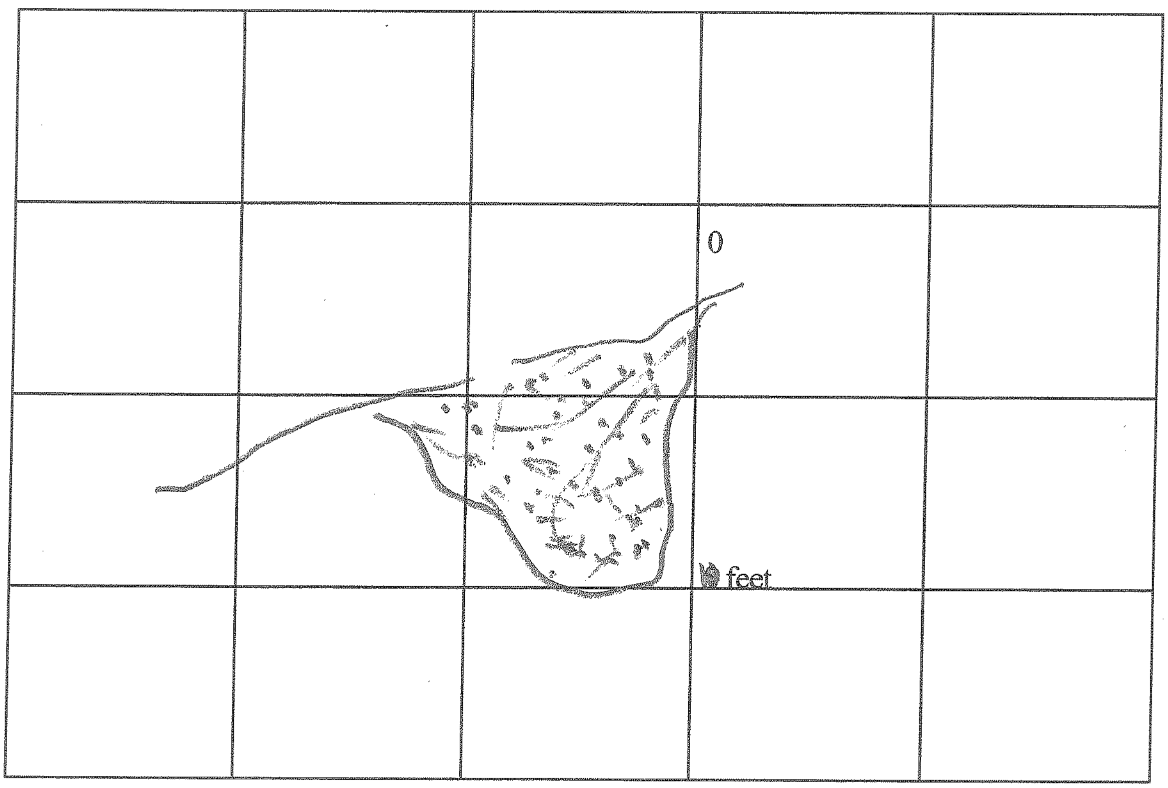
SOIL PACIFIC INC. Geotechnical and Environmental Services	Project Name: Lot 14-17 of Lakeshore Drive, Lake Elsinore	
	Project Number: A-6649-18	Figure:

RECORD OF SUBSURFACE

TEST PIT NO. TP-4

Date: 10/6/18	Logged by: A.Sh.	Equipment: Backhoe

Elev.	USCS	Moist %	Dry Dens. pcf	Depth feet	Description of Earth Materials
	SM			1	Brown, dark brown fine to pebbley clastes and bedrock fragment, with clayey matrix, porous and loose when get wet.
	SM			3	top soil/slope wash.
	SM	5.6	99.6	5	Redish brown coarse grained to pebblley clasts, slightly porous, native/ Qcol.
		8.9	98.3	7	Redish brown, fine to coarse garined silty sand with some clay and scattered clastes and some larger pebbles, moderately dense.
				9	
				11	
				13	End of subsurface exploration 7 feet. Test pit backfilled.



CROSS SECTION SKETCH

Scale: 1"= 5 feet

Trench Orientation: N-S

SOIL PACIFIC INC. Geotechnical and Environmental Services	Project Name: Lot 14-17 of Lakeshore Drive, Lake Elsinore	
	Project Number: A-6649-18	Figure:

APPENDIX B

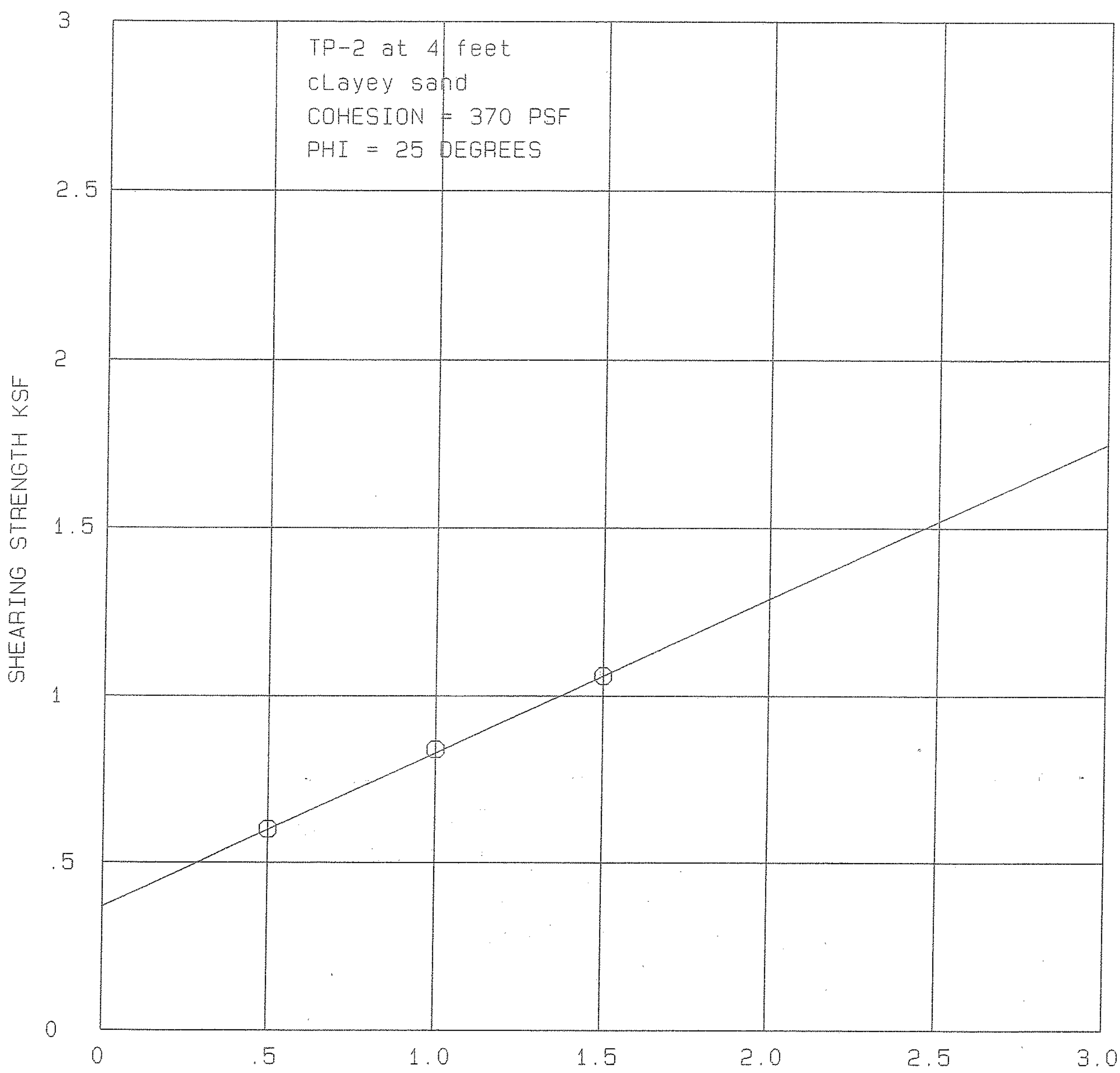
Laboratory

APPENDIX

SHEAR TEST DIAGRAM

J.O. A-6749-18

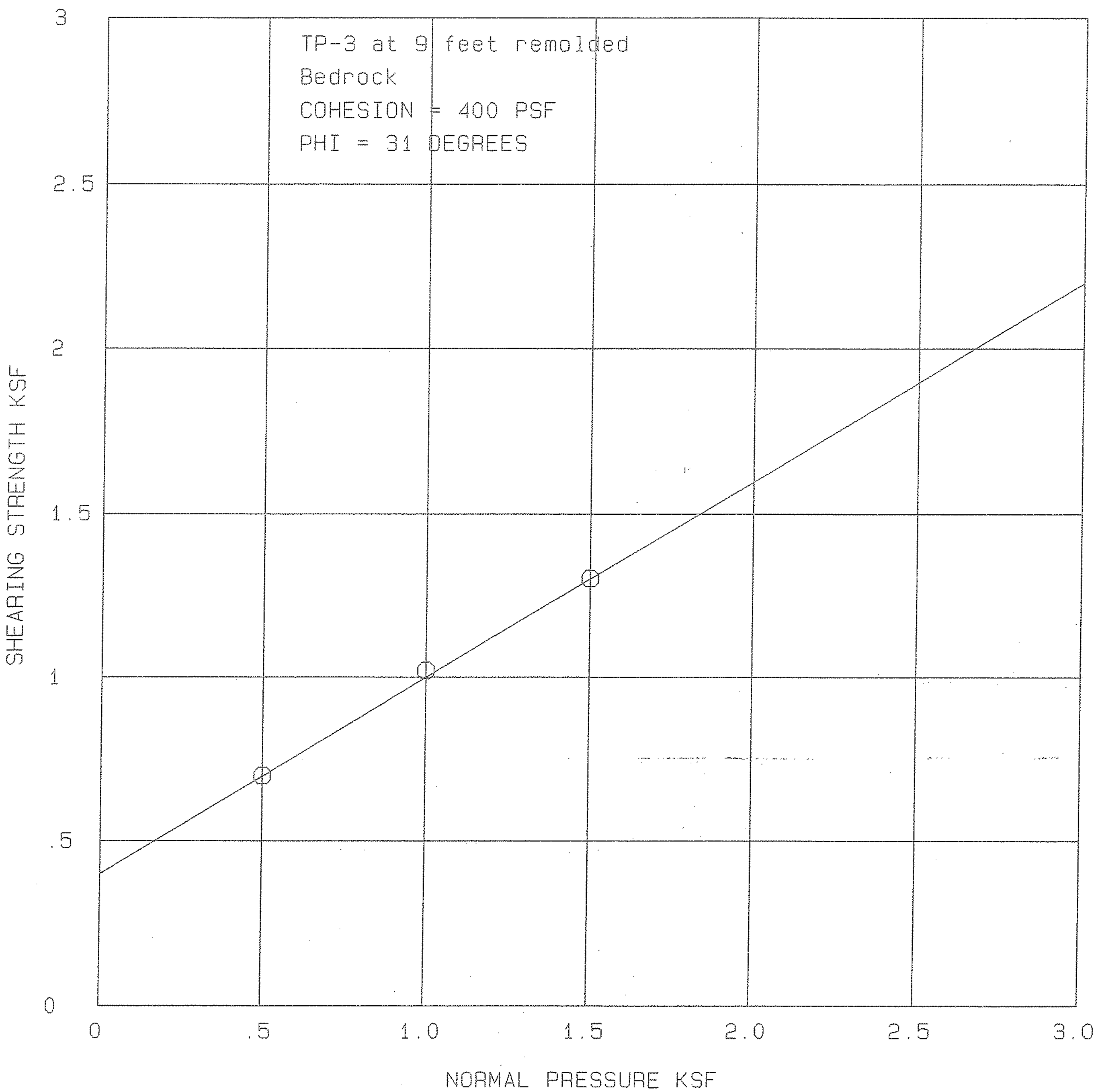
DATE 10/6/18



PLATE

SHEAR TEST DIAGRAM

J.O. A-6749-18 DATE 10/6/18



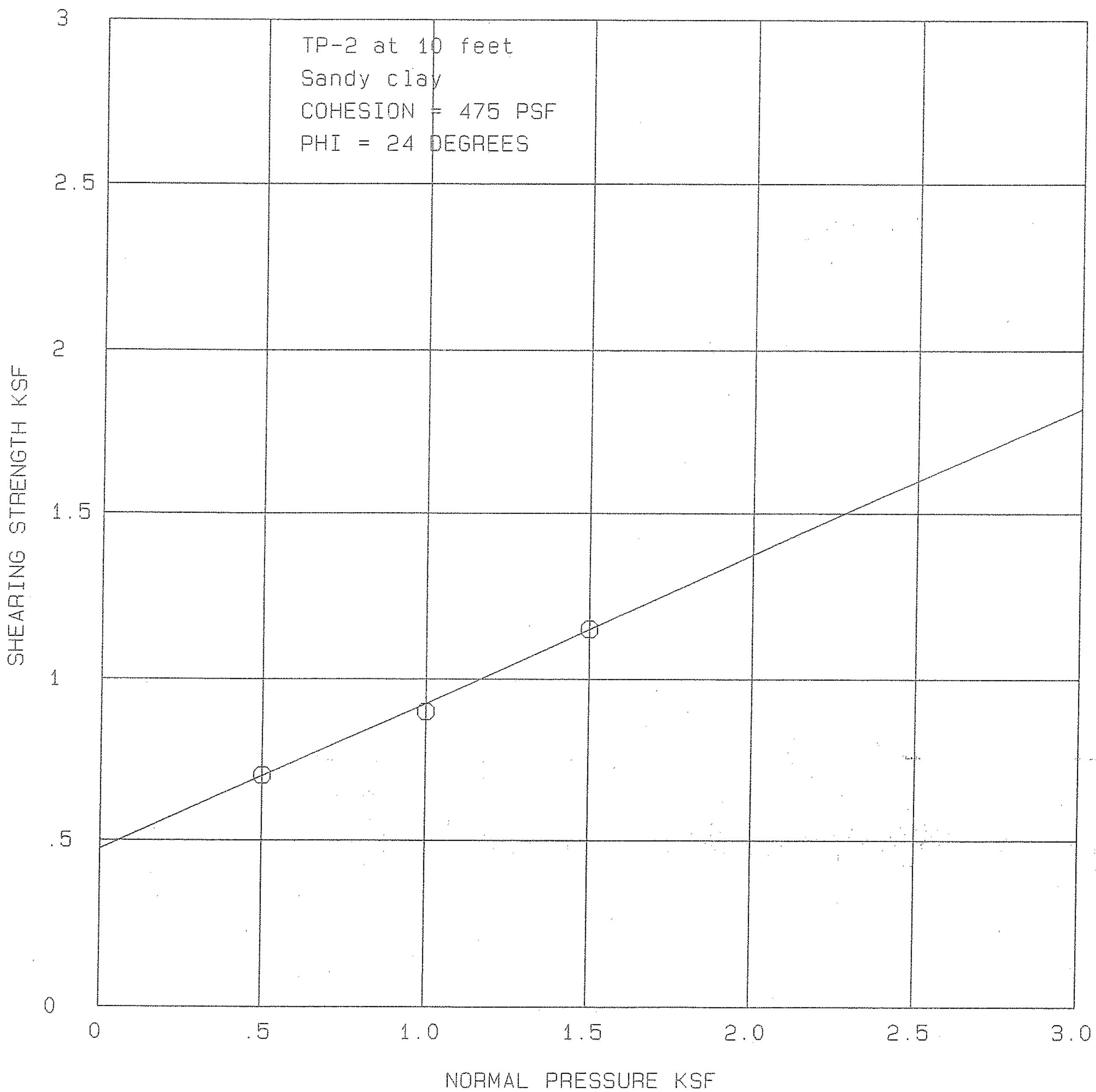
PLATE

APPENDIX

SHEAR TEST DIAGRAM

J.O. A-6749-18

DATE 10/6/18



PLATE

APPENDIX

BEARING VALUE ANALYSIS

J.O. A-6749-18 DATE 10/6/18

COHESION = 400 PSF GAMA = 120 PCF PHI = 31 DEGREES

DEPTH OF FOOTING = 2 FEET

BREADTH OF FOOTING = 2 FEET

FOOTING TYPE = SQUARE

BEARING CAPACITY FACTORS

Nc = 32.7 Nq = 20.6 Ng = 21.6

FOOTING COEFFICIENTS

K1 = 1.2 K2 = .4

REFERENCE: TERZAGHI & PECK; 1967; 'SOIL MECHANICS IN ENGINEERING PRACTICE'; PAGES 217 TO 225.

FORMULA

ULIMATE BEARING = (K1 * Nc * C) + (K2 * GA * Ng * B) + (Nq * GA * D) = 22709.6

$$\text{ALLOWABLE BEARING} = \frac{\text{ULTIMATE BEARING}}{3} = 7569.9$$

THE ALLOWABLE BEARING VALUE SHOULD NOT EXCEED 7569.9 PSF. DESIGN SHOULD CONSIDER EXPANSION INDEX.

PLATE

BEARING VALUE ANALYSIS

J.O. A-6749-18 DATE 10/6/18

COHESION = 400 PSF GAMA = 120 PCF PHI = 31 DEGREES

DEPTH OF FOOTING = 1.5 FEET

BREADTH OF FOOTING = 2 FEET

FOOTING TYPE = CONTINUOUS

BEARING CAPACITY FACTORS

Nc = 32.7 Nq = 20.6 Ng = 21.6

FOOTING COEFFICIENTS

K1 = 1 K2 = .5

REFERENCE: TERZAGHI & PECK; 1967; 'SOIL MECHANICS IN ENGINEERING PRACTICE'; PAGES 217 TO 225.

FORMULA

ULIMATE BEARING = (K1 * Nc * C) + (K2 * GA * Ng * B) + (Nq * GA * D) = 19377.1

$$\text{ALLOWABLE BEARING} = \frac{\text{ULTIMATE BEARING}}{3} = 6459$$

THE ALLOWABLE BEARING VALUE SHOULD NOT EXCEED 6459 PSF. DESIGN SHOULD CONSIDER EXPANSION INDEX.

APPENDIX

TEMPORARY BACKCUT STABILITY

J.O. A-6749-18 DATE 10/6/18

COHESION = 370 PSF GAMA = 120 PCF PHI = 25 DEGREES

CUT HEIGHT = 8 FEET

SLOPE ANGLE OF BACKFILL = 45 DEGREES

ASSUMED FAILURE ANGLE = 52 DEGREES

SOIL TYPE = Clayey sand

PORE PRESSURE NOT CONSIDERED

FORMULA

$$\text{SAFETY FACTOR} = \frac{(C * L) + (GA * AREA * COS(Z) * TAN(PHI))}{GA * AREA * SIN(Z)} = 1.95$$

Z = FAILURE ANGLE

SINCE THE SAFETY FACTOR OF 1.95 IS GREATER THAN THE REQUIRED 1.25, THE TEMPORARY EXCAVATION IS CONSIDERED TO BE STABLE.

PLATE

Earth Pressure Calculations

Soil Strength Parameters:

$\phi := 31$

$\gamma := 120$

Active :

$$Ka := \tan\left[\left(45 - \frac{\phi}{2}\right) \cdot \left(\frac{\pi}{180}\right)\right]^2$$

Active earth Presure

$Ka = 0.32$

$Pa := Ka \cdot \gamma$

slope angle range, degrees

$Pa = 38.412$		LEVEL BACKFILL BEHIND WALL	$Pa = 38.412$
$Pa18 := Pa \cdot 1.08$	5:1	BACKFILL BEHIND WALL	$Pa18 = 41.485$
$Pa18 := Pa \cdot 1.22$	3:1	BACKFILL BEHIND WALL	$Pa18 = 46.862$
$Pa39 := Pa \cdot 1.48$	2:1	BACKFILL BEHIND WALL	$Pa39 = 56.85$

Passive

$$Kp := \tan\left[\left(45 + \frac{\phi}{2}\right) \cdot \left(\frac{\pi}{180}\right)\right]^2$$

$Kp = 3.124$

Pasive Earth Presure

$Pp := Kp \cdot \gamma$

$Pp = 374.884$

Atrest

$$Kat := 1 - \sin\left(\phi \cdot \frac{\pi}{180}\right)$$

$Kat = 0.485$

$Pat := Kat \cdot \gamma$

$Pat = 58.195$

Seismic lateral earth pressure

$\phi := 31 \cdot deg$ angle of internal friction of soil

$\delta := 17 \cdot deg$ angle of friction between soil and wall, (concrete or masonry)

$\gamma := 120$ Soil Unit Weight

$PGAm := .939$

$h := 10$ Height of wall

$$kh := \frac{2}{3} \frac{PGAm}{2}$$

$kh = 0.313$

$$PaE := \frac{3}{8} \cdot \gamma \cdot h^2 \cdot kh$$

$PaE = 1.408 \times 10^3$ Lb

$$EFP := \frac{[2 \cdot (PaE)]}{h^2}$$

$EFP = 28.17$ Seismic Equivelent Fluid Presure (EFP)

Project No.= A – 6749

Seismic lateral earth pressure

$\phi := 38 \cdot deg$ angle of internal friction of soil

$\delta := 17 \cdot deg$ angle of friction between soil and wall, (concrete or masonry)

$\gamma := 130$ Soil Unit Weight

$PGAm := .939$

$h := 40$ Height of wall

$$kh := \frac{2}{3}\frac{PGAm}{2}$$

$$kh = 0.313$$

$$PaE := \frac{3}{8} \cdot \gamma \cdot h^2 \cdot kh$$

$PaE = 2.441 \times 10^4$ Lb

$$EFP := \frac{[2 \cdot (PaE)]}{h^2}$$

$EFP = 30.517$ Seismic Equivelent Fluid Presure (EFP)

Seismic lateral earth pressure

$\phi := 38 \cdot deg$ angle of internal friction of soil

$\delta := 17 \cdot deg$ angle of friction between soil and wall, (concrete or masonry)

$\gamma := 130$ Soil Unit Weight

$PGAm := .939$

$h := 20$ Height of wall

$$kh := \frac{2}{3} \frac{PGAm}{2}$$

$kh = 0.313$

$$PaE := \frac{3}{8} \cdot \gamma \cdot h^2 \cdot kh$$

$PaE = 6.103 \times 10^3$ Lb

$$EFP := \frac{[2 \cdot (PaE)]}{h^2}$$

$EFP = 30.517$ Seismic Equivelent Fluid Presure (EFP)

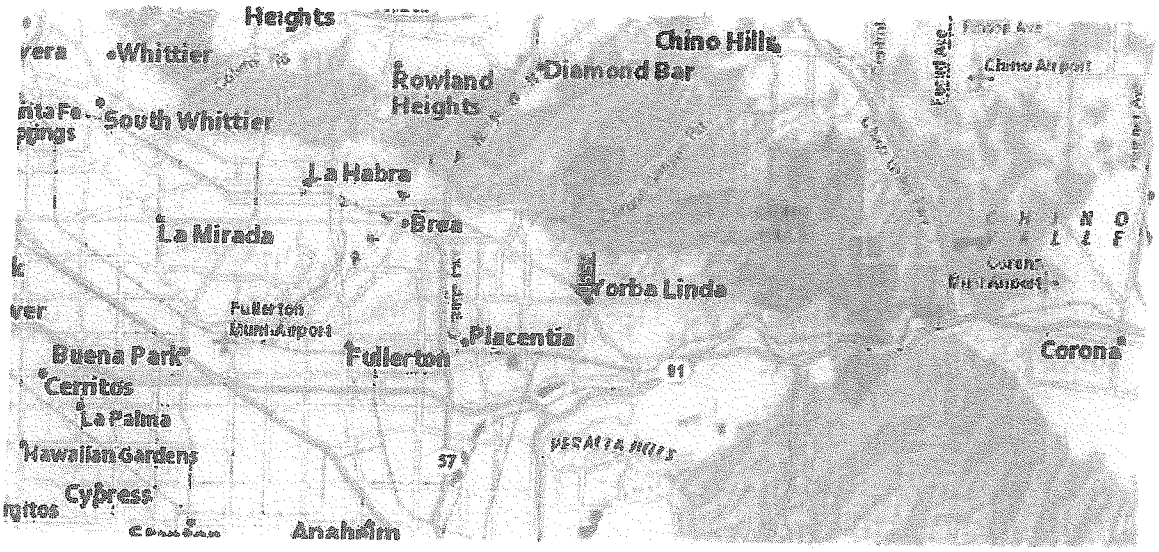
APPENDIX C

References

USGS Design Maps Summary Report

User–Specified Input

Report Title	A-6774-18 Sun November 4, 2018 21:28:31 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.89756°N, 117.8159°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

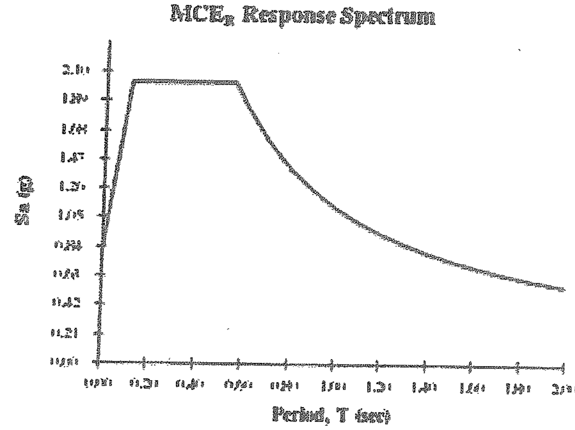


USGS–Provided Output

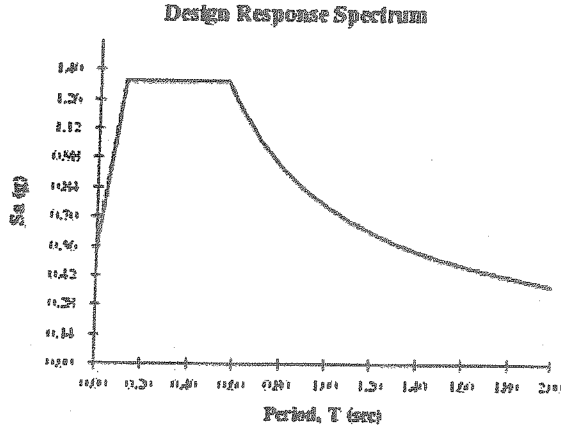
S_S =	2.021 g	S_{MS} =	2.021 g	S_{DS} =	1.347 g
S_1 =	0.749 g	S_{M1} =	1.123 g	S_{D1} =	0.749 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (33.89756°N, 117.8159°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 2.021$ g

From Figure 22-2 [2] $S_1 = 0.749$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.021 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.749 g, F_v = 1.500

Equation (11.4-1):	$S_{MS} = F_aS_S = 1.000 \times 2.021 = 2.021$ g
Equation (11.4-2):	$S_{M1} = F_vS_1 = 1.500 \times 0.749 = 1.123$ g

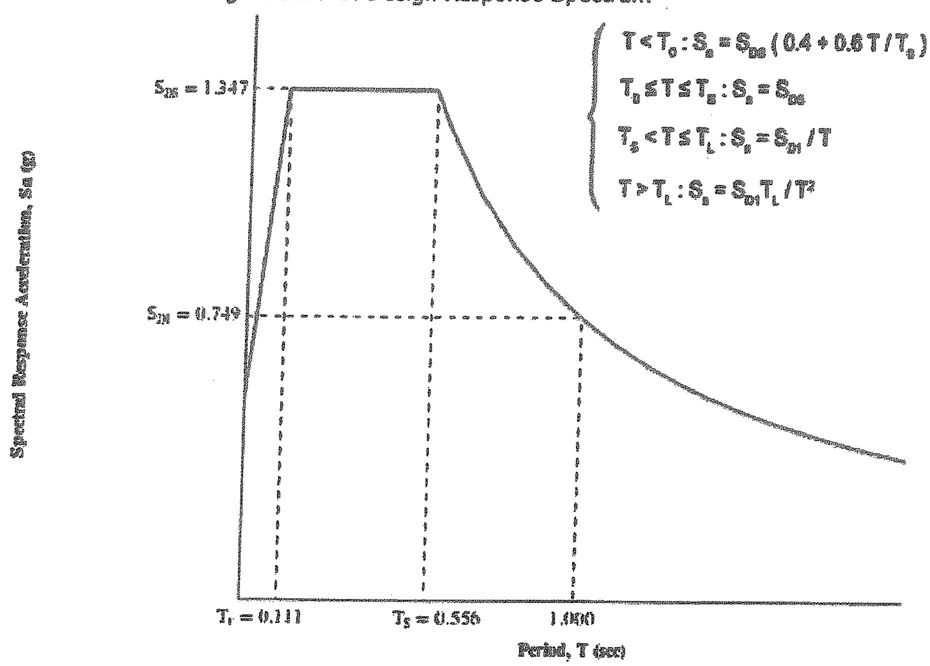
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 2.021 = 1.347$ g
Equation (11.4-4):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 1.123 = 0.749$ g

Section 11.4.5 — Design Response Spectrum

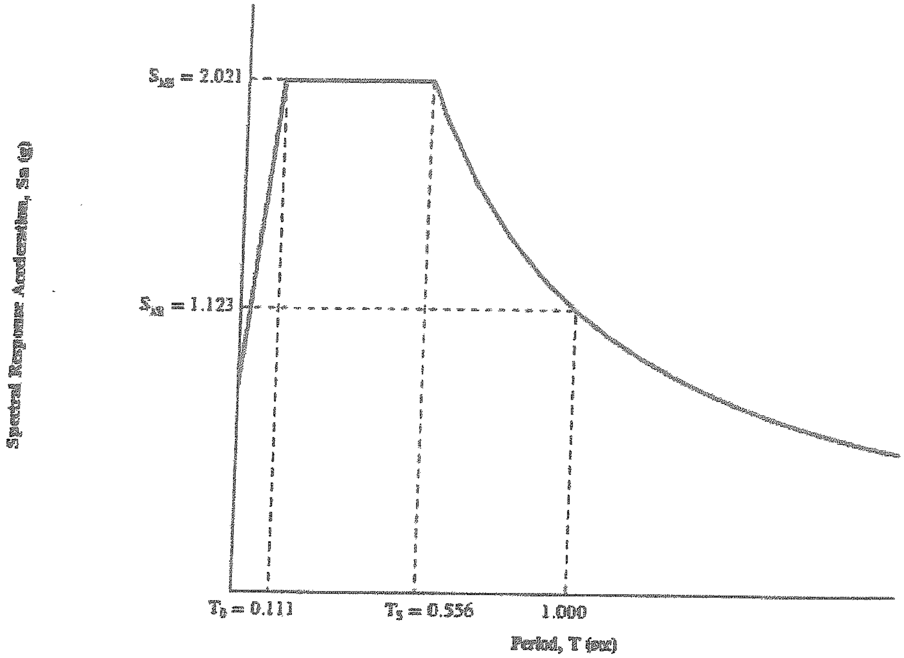
From Figure 22-12 [3]	$T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.770

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.770 = 0.77$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.770 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.960

From Figure 22-18 [6] C_{R1} = 0.966

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.347 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.749 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX D

General Grading Specifications

GENERAL EARTHWORK
AND GRADING SPECIFICATIONS

1. GENERAL INTENT

These specifications present general procedures and requirements for grading and earthwork as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, and excavations. The recommendations contained in the geotechnical report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new recommendations of the geotechnical report.

2.EARTHWORK OBSERVATION AND TESTING

Prior to the commencement of grading, a qualified geotechnical consultant (soils engineer and engineering geologist, and their representatives) shall be employed for the purpose of observing earthwork and testing the fills for conformance with the recommendations of the geotechnical report and these specifications. It will be necessary that the consultant provide adequate testing and observation so that he may determine that the work was accomplished as specified. It shall be the responsibility of the contractor to assist the consultant and keep him apprised of work schedules and changes so that he may schedule his personnel accordingly.

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, poor moisture condition, inadequate compaction, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the consultant will be empowered to reject the work and recommend that construction be topped until the conditions are rectified. Maximum dry density tests used to determine the degree of compaction will be performed in accordance with the American Society of Testing and Materials tests method ASTM D 1557-00.

3.0 PREPARATION OF AREAS TO BE FILLED

3.1 Clearing and Grubbing: All brush, vegetation and debris shall be removed or piled and otherwise disposed of.

3.2 Processing: The existing ground which is determined to be satisfactory for support of fill shall be scarified to a minimum depth of 6 inches. Existing ground which is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until the soils are broken down and free of large clay lumps or clods and until the working surface is reasonably uniform and free of uneven features which would inhibit uniform compaction.

3.3 Overexcavation: Soft, dry, spongy, highly fractured or otherwise unsuitable ground, extending to such a depth that the surface processing cannot adequately improve the condition, shall be overexcavated down to firm ground, approved by the consultant.

3.4 Moisture Conditioning: Overexcavated and processed soils shall be watered, dried-back, blended, and/or mixed, as required to attain a uniform moisture content near optimum.

3.5 Recompaction: Overexcavated and processed soils which have been properly mixed and moisture- conditioned shall be recompacted to a minimum relative compaction of 90 percent.

3.6 Benching: Where fills are to be placed on ground with slopes steeper than 5: 1 (horizontal to vertical units), the ground shall be stepped or benched. The lowest bench shall be a minimum of 15 feet wide, shall be at least 2 feet deep, shall expose firm material, and shall be approved by the consultant. Other benches shall be excavated in firm material for a minimum width of 4 feet. Ground sloping flatter than 5 : 1 shall be benched or otherwise overexcavated when considered necessary by the consultant.

3.7 Approval: All areas to receive fill, including processed areas, removal areas and toe-of-fill benches shall be approved by the consultant prior to fill placement.

4.0 FILL MATERIAL

4.1 General: Material to be placed as fill shall be free of organic matter and other deleterious substances, and shall be approved by the consultant. Soils of poor gradation, expansion, or strength characteristics shall be placed in areas designated by consultant or shall be mixed with other soils to serve as satisfactory fill material.

4.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches, shall not be buried or placed in fills, unless the location, materials, and disposal methods are specifically approved by the consultant. Oversize disposal operations shall be such that nesting of oversize material does not occur, and such that the oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 feet vertically of finish grade or within the range of future utilities or underground construction, unless specifically approved by the consultant.

4.3 Import: If importing of fill material is required for grading, the import material shall meet the requirements of Section 4. 1.

5.0 FILL PLACEMENT AND COMPACTION

5.1 Fill Lifts: Approved fill material shall be placed in areas prepared to receive fill in near-horizontal layers not exceeding 6 inches in compacted thickness. The consultant may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to attain uniformity of material and moisture in each layer.

5.2 Fill Moisture: Fill layers at a moisture content less than optimum shall be watered and mixed, and wet fill layers shall be aerated by scarification or shall be blended with drier material. Moisture-conditioning and mixing of fill layers shall continue until the fill material is at a uniform moisture content or near optimum.

5.3 Compaction of Fill: After each layer has been evenly spread, moisture conditioned, and mixed, it shall be uniformly compacted to not less than 90 percent of maximum dry density. Compaction equipment shall be adequately sized and shall be either specifically designed for soil compaction or of proven reliability, to efficiently achieve the specified degree of compaction.

5.4 Fill Slopes: Compaction of slopes shall be accomplished, in addition to normal compacting procedures, by backfilling of slopes with sheepsfoot rollers at frequent increments of 2 to 3 feet in fill elevation gain, or by other methods producing satisfactory results. At the completion of grading, the relative compaction of the slope out to the slope face shall be at least 90 percent.

5.5 Compaction Testing: Field tests to check the fill moisture and degree of compaction will be performed by the consultant. The location and frequency of tests shall be at the consultant's discretion. In general, the tests will be taken at an interval not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of embankment.

6.0 SUBDRAIN INSTALLATION

Subdrain systems, if required, shall be installed in approved ground to conform to the approximate alignment and details shown on the plans or herein. The subdrain location or materials shall not be changed or modified without the approval of the consultant. The consultant, however, may recommend and upon approval, direct changes in subdrain line, grade or material. All subdrains should be surveyed for line and grade after installation, and sufficient time shall be allowed for the surveys, prior to commencement of filling over the subdrains.

7.0 EXCAVATION

Excavation and cut slopes will be examined during grading. If directed by the consultant, further excavation or overexcavation and refilling of cut areas shall be performed, and/or remedial grading of cut slopes shall be performed. Where fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope shall made and approved by the consultant prior to placement of materials for construction of the fill portion of the slope.

8.0 TRENCH BACKFILLS

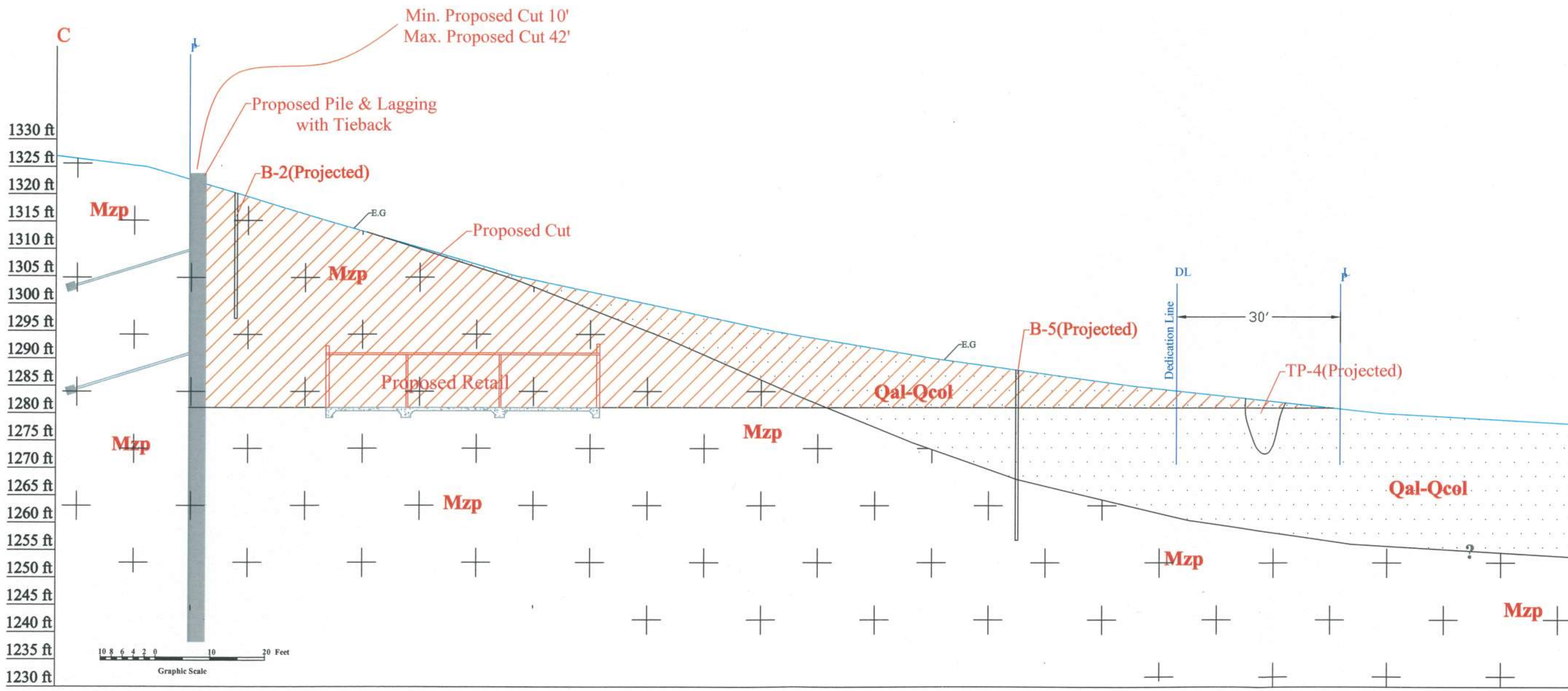
8.1 Supervision: Trench excavations for the utility pipes shall be backfilled under engineering supervision.

8.2 Pipe Zone: After the utility pipe has been laid, the space under and around the pipe shall be backfilled with clean sand or approved granular soil to a depth of at least one foot over the top of the pipe. The sand backfill shall be uniformly jetted into place before the controlled backfill is placed over the sand.

8.3 Fill Placement:'The onsite materials, or other soils approved by the engineer, shall be watered and mixed as necessary prior to placement in lifts over the sand backfill.

8.4 Compaction: The controlled backfill shall be compacted to at least 90 percent of the maximum laboratory density as determined by the ASTM compaction method described above.

8.5 Observation and 'Testing: Field density tests and inspection of the backfill procedures shall be made by the soil engineer during backfilling to see that the proper moisture content and uniform compaction is being maintained. The contractor shall provide test holes and exploratory pits as required by the soil engineer to enable sampling and testing.



LEGEND

Qal/Qcol Alluvial and colluvial deposits= to Morton's Qyva

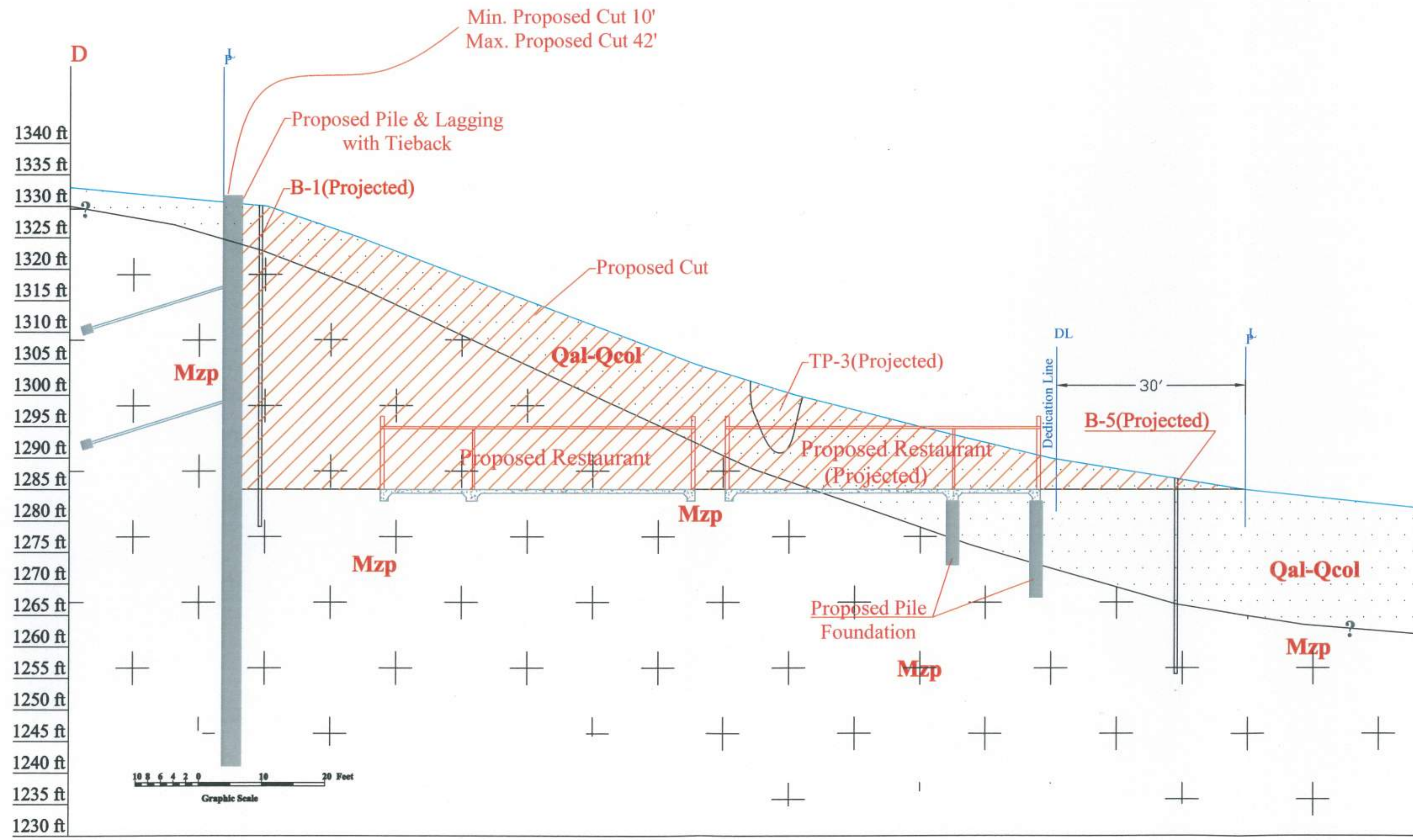
Mzp Mesozoic Phyllite

Testpit Location

soil PACIFIC Inc.
Geotechnical & Environmental Services
675 N. Eckhoff, Suite # A
Orange, CA 92868

Project Location:
Lots 14-17 of Lake Shore Drivr Addition, Lake Elsinore, CA

CROSS SECTION C-C'	
FIGURE-A-1-2	PROJECT NO.: A-6749-19
DATE : 01/31/2019	SCALE: 1"=20'



LEGEND

Qal/Qcol Alluvial and colluvial deposits= to Morton's Qyva

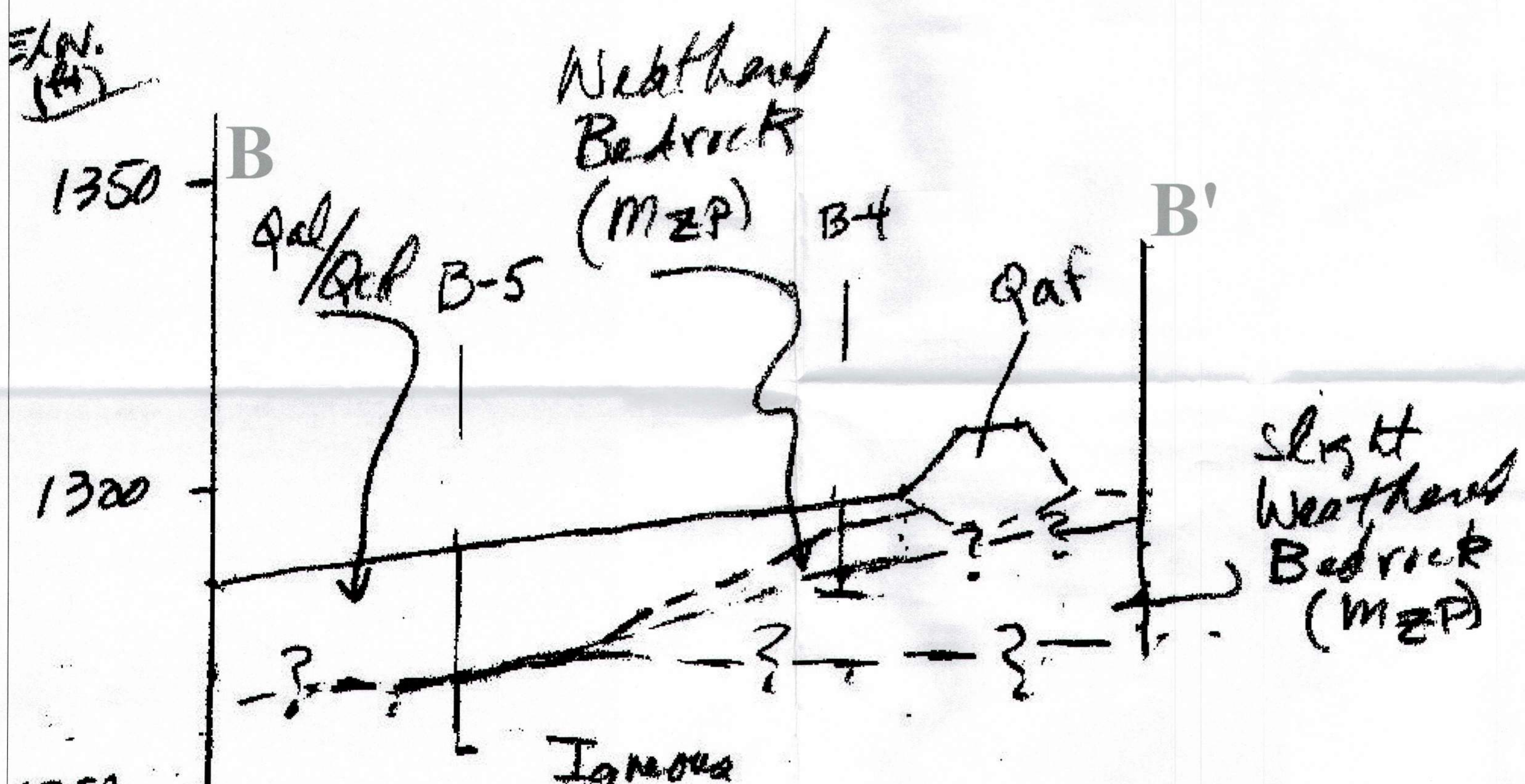
Mzp Mesozoic Phyllite

Testpit Location

soil PACIFIC Inc.
Geotechnical & Environmental Services
675 N. Eckhoff, Suite # A
Orange, CA 92868

Project Location:
Lots 14-17 of Lake Shore Drivr Addition, Lake Elsinore, CA

CROSS SECTION D-D'	
FIGURE-A-1-3	PROJECT NO.: A-6749-19
DATE : 01/31/2019	SCALE: 1"=20'



LEGEND

Qaf	Artificial Fill	Qal/Qcol	Alluvial and colluvial deposits= to Morton's Qyva
Mzp	Mesozoic Phyllite		

Project Location:
Lots 14-17 of Lake Shore Drivr Addition,
Lake Elsinore, CA

CROSS SECTIONS A-A' & B-B'	
FIGURE-A-1-1-1	PROJECT NO.: A-6749-19
DATE : 01/31/2019	SCALE: 1"=30'

LEGEND

Qaf	Artificial Fill	Mzp	Mesozoic Phyllite
Qal/Qcol	Alluvial and colluvial deposits= to Morton's Qyva		Testpit Location
			Percolation Test
	Boring Location		
	Geologic Contact		
	Attitude of foliation		

soil PACIFIC Inc.
Geotechnical & Environmental Services
675 N. Eckhoff, Suite # A
Orange, CA 92868

Project Location:
Lots 14-17 of Lake Shore Drivr Addition,
Lake Elsinore, CA

GEOTECHNICAL PLAN	
FIGURE-A-1-1	PROJECT NO.: A-6749-19
DATE : 01/31/2019	SCALE: 1"=30'