

July 7, 2014

File No. 1426-44

Camp Ramah in California
17525 Ventura Blvd #201
Encino, CA 91316

Att: Randy Michaels, Director of Finance & Administration

Subject: **GEOTECHNICAL INVESTIGATION**
Proposed Construction of
New Residence Buildings and Accessory Structure
385 Fairview Road, Ojai, CA 93024

Dear Mr. Michaels,

As requested, Feffer Geological Consultants performed a geotechnical investigation at the subject site. The purpose of this investigation was to evaluate the geotechnical conditions at the site in the areas of the proposed construction and to provide geotechnical parameters for design and construction of the proposed project.

Based on our investigation, it is our opinion that the proposed construction is feasible from a geotechnical standpoint provided the recommendations contained herein are incorporated into the project plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development. When final plans for the proposed construction become available, they should be forwarded to this office for review and comment.

We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.

Sincerely,

FEFFER GEOLOGICAL CONSULTING, INC.

Joshua R. Feffer
Principal Engineering Geologist
C.E.G. 2138



Dan Daneshfar
Principal Engineer
P.E. 68377

Distribution: Addressee— (3)

County of Ventura
Planning Commission Hearing
Case No. PL18-0052
Exhibit 4 (MND), Attachment 10 - Feffer Geotechnical Report, dated July 7, 2014,
Addendum, dated October 17, 2017 and Responses to Application
Incompleteness determination, dated October 29, 2018

1.1 PURPOSE

1.2 SCOPE OF SERVICES

- Research and review of available pertinent geotechnical literature;
- Subsurface exploration consisting of the excavation of four hand excavated test pits (TP1, TP2, TP3, TP4);
- Sampling and logging of the subsurface soils;
- Laboratory testing of selected soil samples collected from the subsurface exploration to determine the engineering properties of the soil;
- Engineering and geologic analysis of the field and laboratory data; and
- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed construction.

1.3 SITE DESCRIPTION

The project site is located north side of Fairview Road in the City of Ojai, California (Figure 1). The subject site consists of consisting of various buildings and facilities located on a southward sloping property (Figure 2). The subject site is surrounded by undeveloped land and single family residences. Surface drainage is by sheet flow to the south or rear of the property.

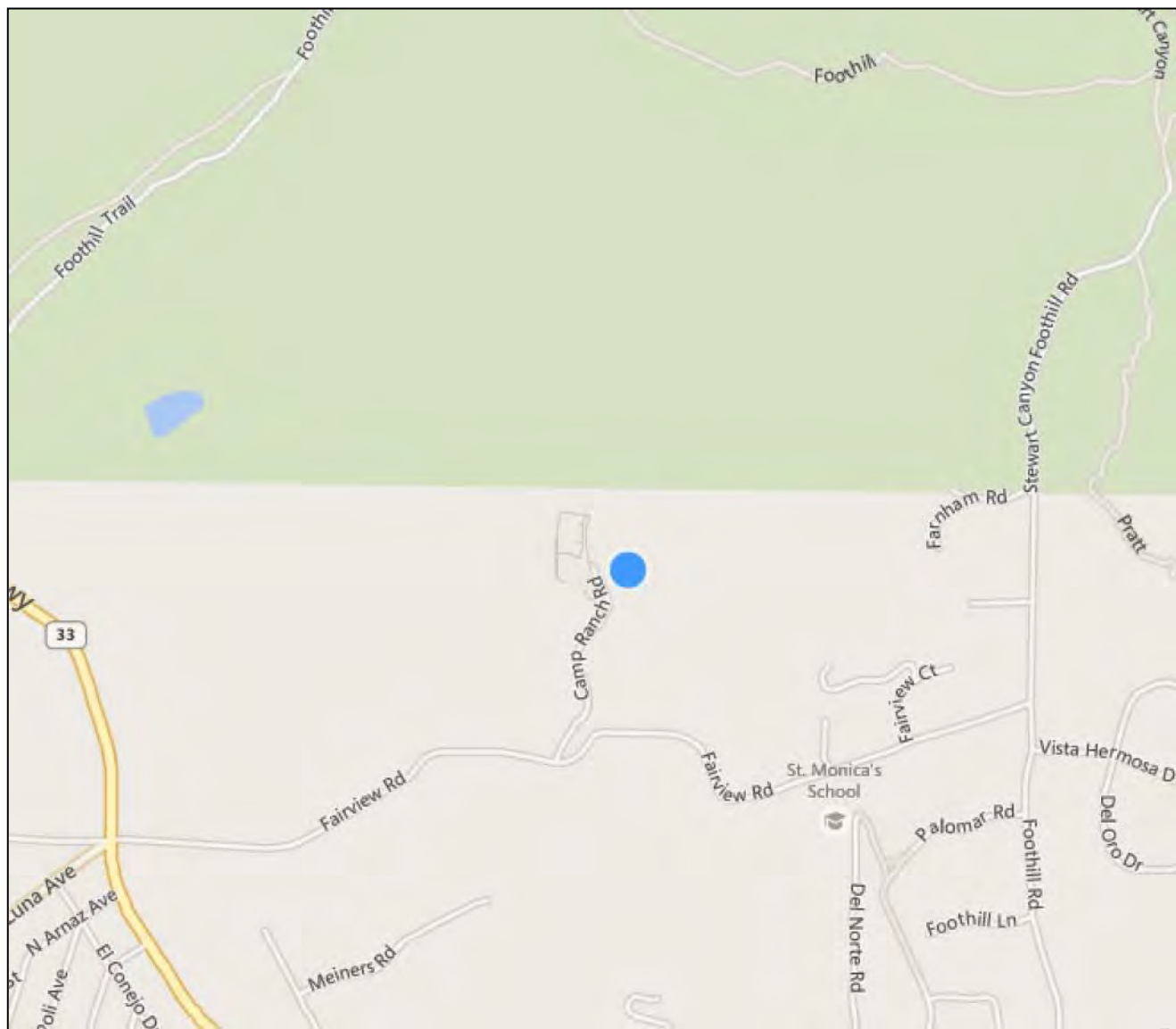


Figure 1. Location map of the site.



Figure 2. Aerial photo of site.

1.5 PROPOSED CONSTRUCTION

Based on the information provided to us, the project will consist of the construction of a housing structure along the area adjacent to the tennis courts within the existing soccer field and construction of an accessory structure to the east of the soccer field. A Site Plan and Cross Sections showing the proposed development are included in Appendix C.

2.0 INVESTIGATION

2.1 GENERAL

Our field investigation was performed on June 19, 2014 and included excavating four test pits and obtaining samples of the earth materials. Our investigation also included laboratory testing of selected soil samples. A brief summary of these various tasks are provided below.

2.2 FIELD EXPLORATION

The subsurface investigation performed at the site consisted of excavating four test pits by use of hand labor. The purpose of the exploratory test pits was to determine the existing subsurface conditions and to collect subsurface soil in the areas of the proposed construction and throughout the site.

The test pits were excavated to a maximum depth of five below the existing ground surface.

The soil materials encountered in Test Pits 1-3 consisted of fill, over alluvium; test pit 4 consisted of colluvium over bedrock.

A review of Regional Geological Maps indicates that the material underlying the subject site is comprised of Quaternary Age Alluvium (Qya1)(Qg) and Sespe Formation (Tsp) bedrock consisting of sandstone.

The test pits were logged by our field geologist using both visual and tactile means. Both bulk and relatively undisturbed soil samples were obtained.

The approximate locations of the test pits are shown on the attached Site Plan included in Appendix C. Detailed test pit logs are presented in Appendix A.

2.3 LABORATORY TESTING

Laboratory testing was performed on representative samples obtained during our field exploration. Samples were tested for the purpose of estimating material properties for use in subsequent engineering evaluations. Testing included in-place moisture and density, hydro-response-swell/collapse, shear strength testing and corrosion. A summary of the laboratory test results is included in Appendix B.

The physical properties of the soils were tested at Soil Labworks, LLC; Chemical testing was performed at HDR Schiff.

The undersigned geologist and engineer have reviewed the data and concur and accept it.

3.0 SITE GEOLOGY, SEISMICITY, POTENTIAL HAZARDS

3.1 SITE GEOLOGY

Regional Geologic Maps¹ (Figure 3) and the subsurface exploration indicated that the property is underlain by Quaternary Age Alluvium (Qya1)(Qg) and Sespe Formation (Tsp) bedrock consisting of sandstone overlain by a thin veneer of fill, and colluvium. Descriptions of the materials encountered in our exploratory test pits are described below.

3.1.1 Fill

The fill consists of fine grained sandy silt which is brown to red brown, slightly moist to moist and dense containing roots. The fill observed was as deep as three feet in the northern portion of the soccer field.

3.1.2 Qya1/Qg

The alluvium consists of admixtures of silt, sands and gravel. The alluvium varies from brown to red-brown. The alluvium is slightly moist to moist, dense containing minor roots.

3.1.3 Colluvium

The colluvium consists of gravelly silty sand and gravelly sand which varies from red-brown to red, orange and pink, slightly moist to moist and dense. The colluvium observed was as deep as one foot in test pit four.

3.1.4 Bedrock

The bedrock encountered consists of Sespe Formation sandstone that is orange, brown and purple, medium to coarse grained with rounded and subrounded cobbles up to 2.5", dry, dense to very hard and massive. There is no out of slope bedding condition at the subject site or on the surrounding slopes.

3.2.3 Groundwater

Groundwater was not encountered during the recent excavations. This area of Ojai is not known to have a high groundwater table. Historically highest groundwater in this area of Ojai is estimated to be more than 40 feet below the ground surface (Plate 1.2, *Historically Highest Groundwater Contours and Borehole Log Data Locations, Matilija 7½ Minute Quadrangle in Seismic Hazard Zone Report for the Matilija Quadrangle*, SHZR-064

¹ *Matilija 7½ Minute Quadrangle in Seismic Hazard Zone Report for the Matilija Quadrangle*, SHZR-064.

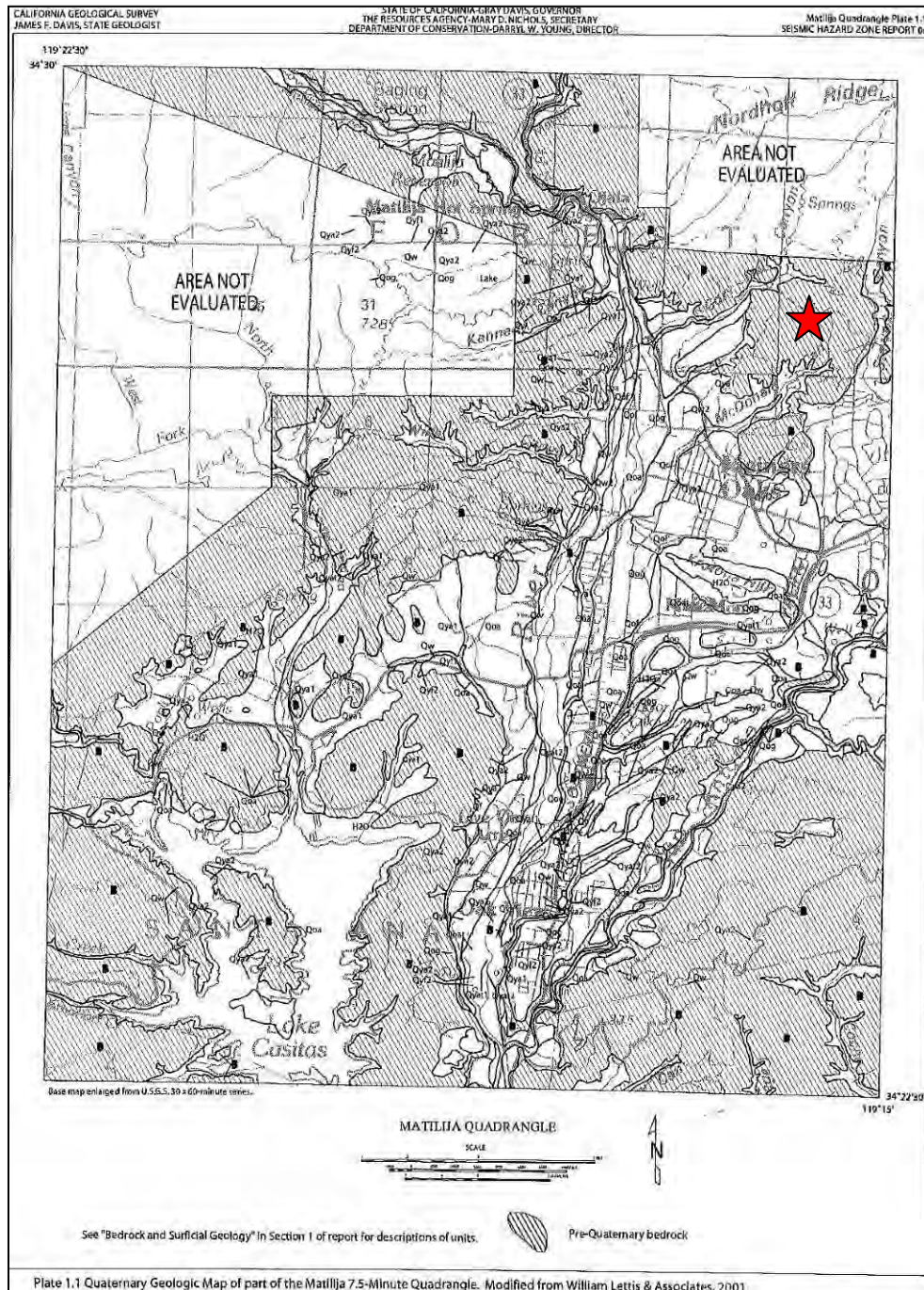
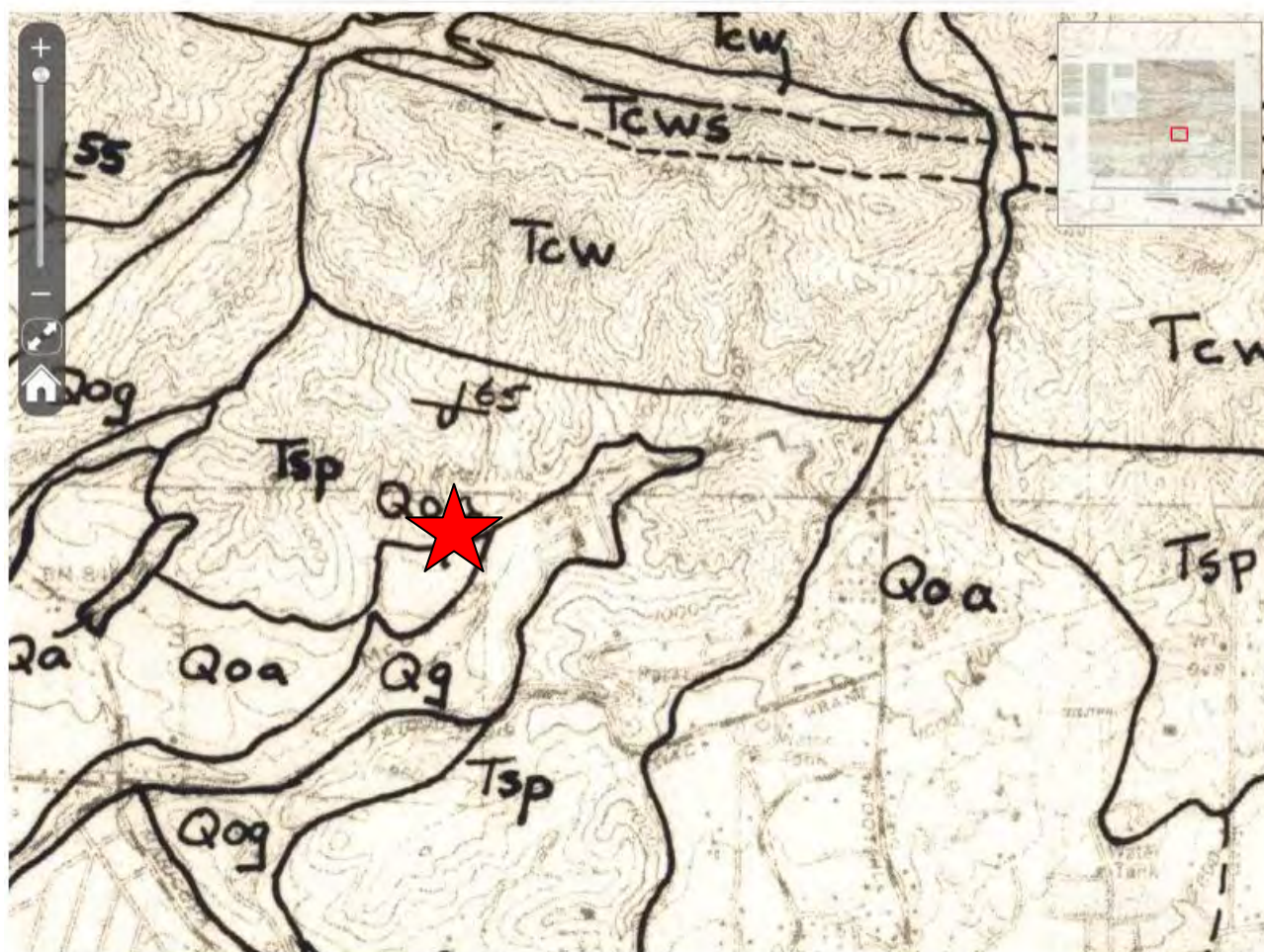


Figure 3. Plate 1.1 Matilija Quadrangle Geologic Map. Site is designated with a red star.

National Geologic Map Database

Image Preview Page



Geologic map

Figure 4. Portion of Matilija 7½ Minute Quadrangle Geologic map. Site is designated with a red star.

3.3 SEISMICITY

A risk common to all areas of Southern California that should not be overlooked is the potential for damage resulting from seismic events (earthquakes). The site is located within a seismically active area, as is all of Southern California. Although we are not aware of any active faults on or within the immediate vicinity of the site, earthquakes generated on large regional faults such as the San Andreas Fault could affect the site.

The closest known potentially active faults to the site are the east-west trending, Santa Ynez Fault, located within two kilometers. Since no active faults cross the property, the surface rupture hazard at the site is very low. Due to the distance from the coastline the site is not susceptible to the effects of tsunamis and seiches.

The subject site is not located in an area designated as being potentially affected by earthquake-induced liquefaction but the northern off site slope is mapped as being subject to earthquake-induced landsliding.

3.4 2013 CALIFORNIA BUILDING CODE CONSIDERATIONS

The proposed development may be designed in accordance with seismic considerations contained in the 2013 California Building Code, Section 1613, the following parameters may be considered for design:

Stiff Soil:

Mapped Spectral Response Acceleration Parameters:

	S_s	:	2.224g
	S_1	:	0.837g
Site Class:	D	:	Stiff Soil
Site Coefficients:	F_a	:	1.0
	F_v	:	1.5

Maximum Considered Earthquake Spectral Response
Acceleration Parameters:

S_{MS}	:	2.224g
S_{M1}	:	1.256g

Design Spectral Response Acceleration Parameters:

S_{DS}	:	1.483g
S_{D1}	:	0.837g

Very Dense Soil and Soft Rock:

Mapped Spectral Response Acceleration Parameters:

	S_S	:	2.224g
	S_1	:	0.837g
Site Class:	C	:	Very Dense Soil and Soft Rock
Site Coefficients:	F_a	:	1.0
	F_v	:	1.3

Maximum Considered Earthquake Spectral Response

Acceleration Parameters:	S_{MS}	:	2.224g
	S_{M1}	:	1.089g

Design Spectral Response Acceleration Parameters:

	S_{DS}	:	1.483g
	S_{D1}	:	0.726g

4.0 GEOTECHNICAL CONSIDERATIONS

4.1 SUBSURFACE SOIL CONDITIONS

Subsurface materials at the site consist of alluvium and bedrock below fill and colluvium respectively. On the subject property there was up to three feet of fill over alluvium and up to one foot of colluvium over bedrock. Laboratory testing indicates that the alluvium and bedrock has a low potential for consolidation and hydrocollapse and is stable. The following paragraph provides general discussions about settlement and expansive soil activity.

4.2 SETTLEMENT

Our investigation indicated that the consolidation and hydrocollapse potential of the alluvium and bedrock at the depth of the proposed construction is low. Recommendations are presented below to mitigate the settlement hazard associated with consolidation of the near surface soils.

4.3 EXPANSIVE SOIL

The on-site, near surface soil was found to possess low to medium expansive characteristics based upon field soil classifications.

4.4 SLOPE STABILITY

The slope above the proposed accessory structure is oriented at a gradient of 3:1 (horizontal to vertical) and is as high as fifty-seven feet.

Cross section B-B' (Appendix C) was developed from the site topographic map. Gross slope stability analysis was performed for the existing slope, as depicted in the attached Geologic Cross Section B-B' by a Taylor's Analysis. The analysis indicates that 2:1 degree slopes in the bedrock have a factor of safety of 1.5 at a height of 98.8 feet; therefore the existing fifty-seven foot high 3:1 slope calculates as stable.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 BASIS

Conclusions and recommendations contained in this report are based upon information provided, information gathered, laboratory testing, engineering and geologic evaluations, experience, and judgment. Recommendations contained herein should be considered minimums consistent with industry practice. More rigorous criteria could be adopted if lower risk of future problems is desired. Where alternatives are presented, regardless of what approach is taken, some risk will remain, as is always the case. Usually the lowest risk is associated with the greatest cost.

5.2 SITE SUITABILITY

The site is within an area including completed housing and building developments. Geotechnical exploration, analyses, experience, and judgment result in the conclusion that the proposed development is suitable from a geotechnical standpoint.

It is our opinion that the site can be improved without hazard of landslide, slippage, or settlement, and improvement can occur without similar adverse impact on adjoining properties. Realizing this expectation will require adherence to good construction practice, agency and code requirements, the recommendations in this report, and possible addendum recommendations made after plan review and at the time of construction.

Based on the results of our subsurface investigation and due to the over-consolidated nature of the alluvial deposits and bedrock, the potential for liquefaction at the site during earthquake shaking is considered to be nil. The location of the proposed construction is not located within an area identified as being within a liquefaction zone (*Ojai 7½ Minute Quadrangle in Seismic Hazard Zone Report for the Ojai Quadrangle*, SHZR-072).

It should be realized that the purpose of the seismic design utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of a large earthquake. This is the case for essentially all structures in Southern California.

5.4

EARTHWORK

5.4.1 General

The existing natural alluvium, bedrock and/or a future compacted fill cap can be used for support of any new footings. Where fill is intended for structural support, the compacted fill cap should extend at least three feet below the bottom of footings and five feet out of the building footprint. If the proposed construction will require grading of the site; it should be done in accordance with good construction practice, minimum code requirements and recommendations to follow. Grading criteria are included within Appendix D.

5.4.2 Site Preparation and Grading

The material at the subject site consists of fill over alluvium and colluvium over bedrock. The foundation for the development should derive support from the alluvium or bedrock or a future compacted fill cap. Prior to the start of grading operations, utility lines within the project area, if any, should be located and marked in the field so they can be rerouted or protected during site development. All debris and perishable material should be removed from the site. Although currently not anticipated, all permanent cut and fill slopes should not be constructed steeper than 2:1.

If fill is to be placed, the upper six to eight inches of surface exposed by the excavation should be scarified; moisture conditioned to two to four percent over optimum moisture content, and compacted to 90 percent relative compaction². If localized areas of relatively loose soils prevent proper compaction, over-excavation and re-compaction will be necessary.

The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. The maximum density shall be determined by ASTM D 1557-12 or equivalent. On site fill is adequate for use as fill.

5.4.3 Excavation Characteristics

The test pits did encounter hard, cemented bedrock. Excavation difficulty is a function of the degree of weathering and amount of fracturing within the bedrock. The bedrock generally becomes harder and more difficult to excavate with increasing depth. Hard cemented layers are also known to occur at random locations and depths and may be encountered during foundation excavation. Should a hard cemented layer be encountered, coring or the use of jackhammers may be necessary.

² Relative compaction refers to the ratio of the in-place dry density of soil to the maximum dry density of the same material as obtained by the "modified proctor" (ASTM D1557-12) test procedure.

5.5 FOUNDATION SUPPORT

5.5.1 New Structures

All proposed structural foundations shall be embedded within the alluvium or bedrock or a future compacted fill cap in accordance with the recommendations presented below. Geologic conditions on the site are favorable for the proposed construction. For an individual structure, all footings should be embedded in the same material (alluvium, bedrock, or new fill).

Foundation support for the new structures could be derived by utilizing conventional shallow foundations embedded within the alluvium or bedrock or a future compacted fill cap. Allowable design parameters for foundations are provided below.

Minimum depth for interior and exterior footing (Measured from lowest adjacent grade)	2 feet
Minimum width.....	1.5 feet

For Bedrock:

Bearing pressure	
a. Sustained loads (lbs. per square foot)	3,000 psf
Resistance to lateral loads	
a. Passive soil resistance (lbs. per cubic ft.)	
Within bedrock	500 pcf
Maximum allowable	5,000 psf
b. Coefficient of sliding friction.....	0.45

For Compacted Fill:

Bearing pressure	
a. Sustained loads (lbs. per square foot)	2,000 psf
Resistance to lateral loads	
a. Passive soil resistance (lbs. per cubic ft.)	
Within compacted fill	250 pcf
Maximum allowable	2,500 psf
c. Coefficient of sliding friction.....	0.35

For Alluvium:

Bearing pressure	
a. Sustained loads (lbs. per square foot)	2,000 psf
Resistance to lateral loads	
a. Passive soil resistance (lbs. per cubic ft.)	
Within alluvium.....	250 pcf
Maximum allowable	2,500 psf
d. Coefficient of sliding friction.....	0.30

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 1.5.

Increases in the bearing value of the Bedrock are allowable at a rate of 400 pounds per square foot for each additional foot of footing width to a maximum of 4,000 pounds per square foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

All continuous footings should be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footing excavations should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geologist and geotechnical engineer prior to placing forms, steel or concrete.

Based on the anticipated building loads footings designed and constructed in accordance with the soil criteria included within the referenced report are expected to settle less than $\frac{1}{4}$ to $\frac{1}{2}$ inch in a distance of 20 feet. Differential settlement is expected to be less than $\frac{1}{4}$ inch. The total and differential settlements are within acceptable and allowable tolerances for conventional foundations.

5.6 RETAINING WALLS

5.6.1 Retaining Wall

Although not currently contemplated cantilevered retaining walls up to 12 feet high that support fill, alluvium, bedrock and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 30 pounds per cubic foot for level backfill. Restrained retaining walls that are pinned at the top by a non-yielding floor should be designed for an at-rest pressure. The design at-rest earth pressure on restrained basement walls is 60 pcf. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of $\frac{3}{4}$ inch crushed gravel.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls.

5.6.2 Waterproofing

Moisture affecting retaining walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite,

and/or halite (common salt). Efflorescence is common to retaining walls and generally does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls.

As aforementioned, the architect, structural engineer, or other qualified waterproofing consultant should develop the actual waterproofing details.

5.6.3 Retaining Wall Drainage

Retaining walls that use a subdrain should have the subdrain pipe surrounded with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The project structural engineer will incorporate an appropriately designed wall back-drain system for the purpose of mitigating potential for hydrostatic and/or seepage forces. Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing sub drainage pipe, the type and brand be verified and cleared with the proper municipal agencies. Sub drainage pipes should daylight and outlet to an acceptable location.

5.6.4 Retaining Wall Backfill

The onsite earth materials are acceptable for use as retaining wall backfill. Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90% (or 95%) of the maximum dry density obtained using test method ASTM D 1557-12 or equivalent. Flooding or jetting is not permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

Gravel or onsite earth materials will be utilized for backfill. If gravel is used, the upper 24 inches of backfill should consist of more cohesive material to minimize surface infiltration. Retaining wall backfill should be capped with a paved surface drain or pavement

It should be pointed out that the use of heavy compaction equipment in close proximity to retaining walls can result in excess wall movement and/or soil loadings exceeding design values. In this regard, care should be taken during backfilling operations.

5.7 TEMPORARY EXCAVATIONS

All vertical cuts shall be inspected by our office to verify geologic continuity.

Un-shored vertical cuts to a height of five feet (5') may be made in soil materials at the site. Un-shored cuts in excess of five feet (5') shall be sloped at a gradient of no steeper than 1:1 (horizontal to vertical) for the portion of the excavation above the vertical cut.

5.8 SLAB-ON-GRADE

If a slab-on-grade is used for the interior of the building it should be a minimum of four inches thick and reinforced with No. 4 bars at 16 inches on center, both ways. If it is desired to minimize vapor transmission through the slab, then the slab should be underlain by a 10-mil Visqueen plastic membrane sandwiched between two, two-inch thick layers of sand. The sand should contain sufficient fines to allow light compaction (e.g. drum roller) to an unyielding condition. The plastic Visqueen barrier should be sealed at all splices, around plumbing, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier and care should be taken to not puncture the membrane. The splices between layers should be generously staggered.

5.9 EXTERIOR FLATWORK

Whenever planned, exterior flatwork should be placed directly on alluvium or over at least a two-foot blanket of approved compacted fill. Five inch net sections with #4 bars at 18 inches o.c.e.w. are also advised. Control joints should be planned at not more than twelve foot spacing for larger concrete areas. Narrower areas of flatwork such as walkways should have control joints planned at not greater than 1.5 times the width of the walkway. Recommendations provided above for interior slabs can also be used for exterior flatwork, but without a sand layer or Visqueen moisture barrier. Additionally, it is also recommended that at least 12-inch deepened footings be constructed along the edges of larger concrete areas.

Movement of slabs adjacent to structures can be mitigated by doweling slabs to perimeter footings. Doweling should consist of No. 4 bars bent around exterior footing reinforcement. Dowels should be extended at least two feet into planned exterior slabs. Doweling should be spaced consistent with the reinforcement schedule for the slab. With doweling, 3/8-inch minimum thickness expansion joint material should be provided. Where expansion joint material is provided, it should be held down about 3/8 inch below the surface. The expansion joints should be finished with a color matched, flowing, flexible sealer (e.g., pool deck compound) sanded to add mortar-like texture. As an option to doweling, an architectural separation could be provided between the main structures and abutting appurtenant improvements.

5.10 CONCRETE

Testing of the soil indicates that sulfate levels are negligible (35 ppm) and therefore Type II concrete may be used. We recommend that the low permeable concrete be utilized at the site to limit moisture transmission through slab and foundation. For this purpose, the water/cement ratio to be used at the site should be limited to 0.5 (0.45 preferred). Limited use (subject to approval of mix designs) of a water reducing agent may be included to increase workability. The concrete should be properly cured to minimize risk of shrinkage cracking. The code dictates at least seven days of moist curing. Two to three weeks is preferred to minimize cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements provided the mix designs consider limiting shrinkage.

Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Mix designs proposed by the contractor should be considered subject to review by the project engineer.

5.11 DRAINAGE

Drainage should be directed away from structures via non-erodible conduits to suitable disposal areas. Two percent drainage is recommended directly away from structures however minimum building code requirements should be followed. All enclosed planters should be provided with a suitably located drain or drains and/or flooding protection in the form of weep holes or similar. Preferably, structures should have roof gutters and downspouts tied directly to the area drainage system.

5.12 PLAN REVIEW

When detailed grading and structural plans are developed, they should be forwarded to this office for review and comment.

5.13 AGENCY REVIEW

All soil, geologic, and structural aspects of the proposed development are subject to the review and approval of the governing agency(s). It should be recognized that the governing agency(s) can dictate the manner in which the project proceeds. They could approve or deny any aspect of the proposed improvements and/or could dictate which foundation and grading options are acceptable.

5.14 SUPPLEMENTAL CONSULTING

During construction, a number of reviews by this office are recommended to verify site geotechnical conditions and conformance with the intentions of the recommendations for construction. Although not all possible geotechnical observation and testing services are required by the governing agencies, the more site reviews requested, the lower the risk of future site problems. The following site reviews are advised, some of which will probably be required by the agencies.

Preconstruction/pregrading meeting	Advised
Cut and/or shoring observation	Required
Periodic geotechnical observations and testing during grading	Required
Reinforcement for all foundations	Advised
Slab subgrade moisture barrier membrane.....	Advised
Slab subgrade rock placement	Advised
Presaturation checks for all slabs in primary structure areas	Required
Presaturation checks for all slabs for appurtenant structures	Advised
Slab steel placement, primary and appurtenant structures	Advised
Compaction of utility trench backfill.....	Advised

Unless otherwise agreed to in writing, all supplemental consulting services will be provided on an as-needed, time-and-expense, fee schedule basis.

5.15 PROJECT SAFETY

The contractor is the party responsible for providing a safe site. This consultant will not direct the contractor's operations and cannot be responsible for the safety of personnel other than his own representatives on site. The contractor should notify the owner if he is aware of and/or anticipates unsafe conditions. If the geotechnical consultant at the time of construction considers conditions unsafe, the contractor, as well as the owner's representative, will be notified. Within this report the terminology safe or safely may have been utilized. The intent of such use is to imply low risk. Some risk will remain, however, as is always the case.

6.0

REMARKS

Only a portion of subsurface conditions have been reviewed and evaluated. Conclusions, recommendations and other information contained in this report are based upon the assumptions that subsurface conditions do not vary appreciably between and adjacent to observation points. Although no significant variation is anticipated, it must be recognized that variations can occur.

This report has been prepared for the sole use and benefit of our client. The intent of the report is to advise our client on geotechnical matters involving the proposed improvements. It should be understood that the geotechnical consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspect of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive the advice. Subsequent use of this report can only be authorized by the client. Any transferring of information or other directed use by the client should be considered "advice by the client."

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

APPENDIX ‘A’

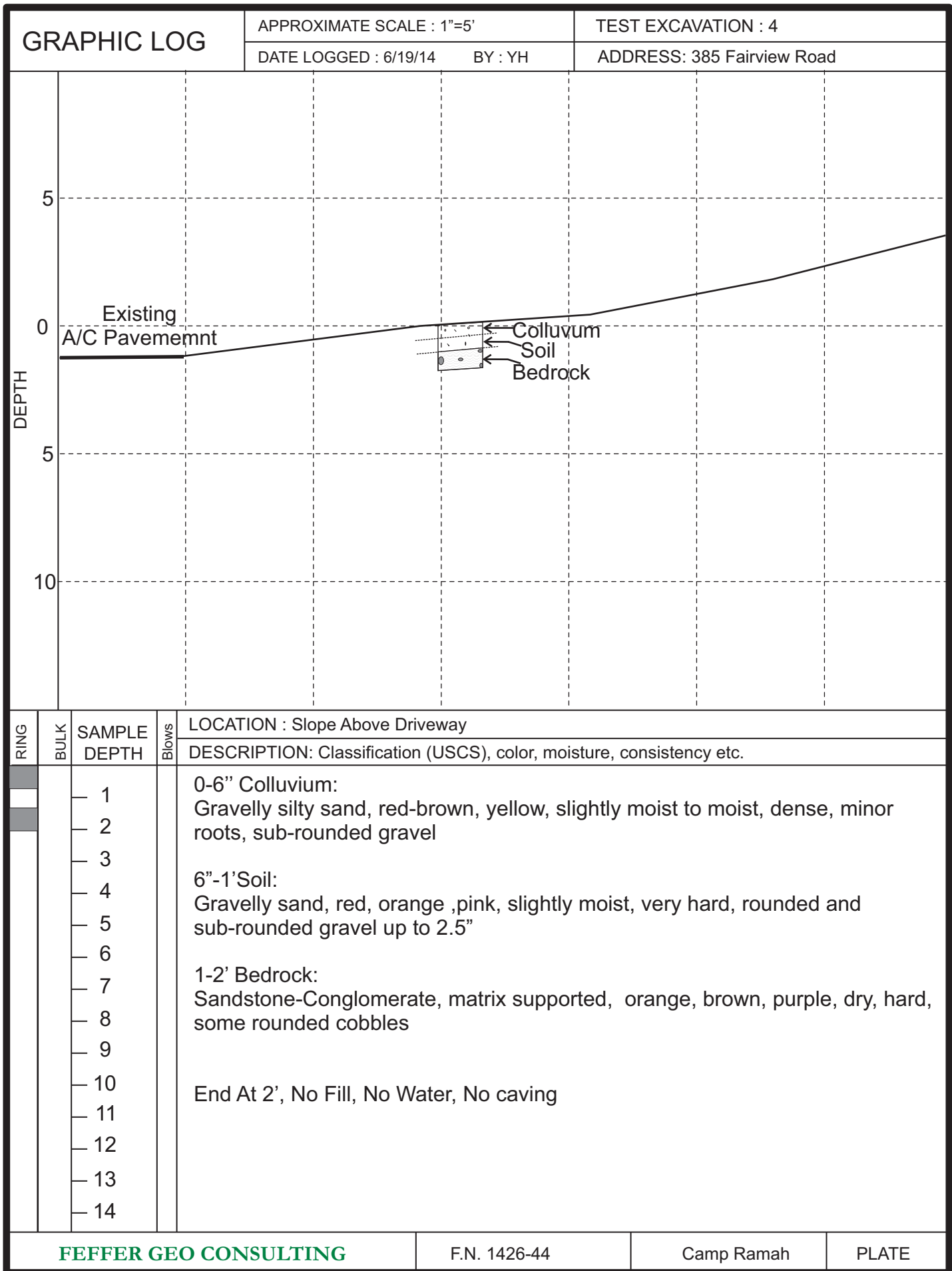
Test Pits

GRAPHIC LOG			APPROXIMATE SCALE : 1"=5'	TEST EXCAVATION : 1
			DATE LOGGED : 6/19/14 BY : YH	ADDRESS: 385 Fairview Road
DEPTH				
5				
0			Existing Soccer Field	
5				
10				
RING	BULK	SAMPLE DEPTH	LOCATION : Soccer Field	
Blows	DESCRIPTION: Classification (USCS), color, moisture, consistency etc.			
1		1	<p>0-3' Fill: Sandy silt, brown, slightly moist to moist, dense, roots</p> <p>3-5' Alluvium: Sandy silt, clay binder, red-brown, moist, dense</p> <p>End At 5', Fill to 3', No Water, No caving</p>	
2		2		
3		3		
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FEFFER GEO CONSULTING			F.N. 1426-44	Camp Ramah
			PLATE	

GRAPHIC LOG			APPROXIMATE SCALE : 1"=5'	TEST EXCAVATION : 2
			DATE LOGGED : 6/19/14 BY : YH	ADDRESS: 385 Fairview Road
<div style="display: flex;"> <div style="writing-mode: vertical-rl; transform: rotate(180deg); font-weight: bold; margin-right: 10px;">DEPTH</div> <div style="flex-grow: 1;"> </div> </div>				
RING	BULK	SAMPLE DEPTH	Blows	LOCATION : Soccer Field
				DESCRIPTION: Classification (USCS), color, moisture, consistency etc.
		1		<p>0-4" Fill:</p> <p>Top soil-Silt, brown, moist, loose, numerous roots</p> <p>Sandy silt, red-brown, slightly moist to moist dense, minor roots</p> <p>4"-3' Alluvium:</p> <p>Sandy silt, clay binder, red-brown, moist, dense</p> <p>End At 3', Fill to 4", No Water, No caving</p>
		2		
		3		
		4		
		5		
		6		
		7		
		8		
		9		
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		11		
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		14		

FEFFER GEO CONSULTING	F.N. 1426-44	Camp Ramah	PLATE
-----------------------	--------------	------------	-------

GRAPHIC LOG		APPROXIMATE SCALE : 1"=5'		TEST EXCAVATION : 3	
		DATE LOGGED : 6/19/14 BY : YH		ADDRESS: 385 Fairview Road	
<div style="display: flex;"> <div style="writing-mode: vertical-rl; transform: rotate(180deg); font-weight: bold; margin-right: 10px;">DEPTH</div> <div style="flex-grow: 1;"> </div> </div>					
RING	BULK	SAMPLE DEPTH	Blows	LOCATION : Soccer Field	
				DESCRIPTION: Classification (USCS), color, moisture, consistency etc.	
		1		<p>0-5" Fill: Silt, brown, moist, loose, numerous roots</p> <p>5"-2' Alluvium: Sandy silt, brown, slightly moist to moist, dense, minor roots</p> <p>2-4' Gravelly sand, brown to red-brown, moist, dense</p> <p>End At 4', Fill to 5", No Water, No caving</p>	
		2			
		3			
		4			
		5			
		6			
		7			
		8			
		9			
		10			
		11			
		12			
		13			
		14			
FEFFER GEO CONSULTING				F.N. 1426-44	Camp Ramah
				PLATE	



APPENDIX 'B'

Laboratory Testing



SL14.1670
June 25, 2014

Feffer Geological Consulting
1990 S. Bundy Drive
4th Floor
Los Angeles, California 90025

Attn: Joshua R. Feffer

Subject: Laboratory Testing

Site: 385 Fairview Road
Ojai, CA

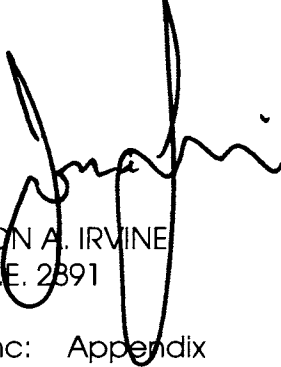
Job: FEFFER/CAMP RAMAH

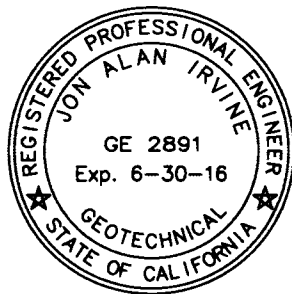
Laboratory testing for the subject property was performed by Soil Labworks, LLC., under the supervision of the undersigned Engineer in conjunction with a geotechnical investigation. Samples of the earth materials were obtained from the subject property by personnel of Feffer Geological Consulting and transported to the laboratory of Soil Labworks for testing and analysis. The laboratory tests performed are described and results are attached.

Services performed by this facility for the subject property were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions.

Respectfully Submitted:

SOIL LABWORKS, LLC


JON A. IRVINE
G.E. 2891
Enc: Appendix





SL14.1670
June 25, 2015

APPENDIX

Laboratory Testing

Sample Retrieval - Hand Labor

Samples of earth materials were obtained by driving a thin-walled steel sampler with successive blows of a drop hammer. The earth material was retained in brass rings of 2.416 inches inside diameter and 1.00 inch height. The samples were stored in closefitting, water-tight containers for transportation to the laboratory.

Moisture Density

The field moisture content and dry density were determined for each of the soil samples. The dry density was determined in pounds per cubic foot following ASTM 2937-10. The moisture content was determined as a percentage of the dry soil weight conforming to ASTM 2216-10. The results are presented below in the following table. The percent saturation was calculated on the basis of an estimated specific gravity. Description of earth materials used in this report and shown on the attached Plates were provided by the client.

Test Pit/Boring No.	Sample Depth (Feet)	Soil Type	Dry Density (pcf)	Moisture Content (percent)	Percent Saturation ($G_s=2.65$)
TP1	1½	Fill	107.3	9.1	45
TP1	4½	Alluvium	119.1	12.2	83
TP2	2	Alluvium	117.8	11.1	73
TP3	2	Alluvium	102.8	5.6	25
TP3	4	Alluvium	114.1	5.4	32
TP4	0-6"	Colluvium	106.2	6.3	30
TP4	1.5-2	Bedrock	118.6	5.7	38

Shear Strength

The peak and ultimate shear strengths of the colluvium, alluvium and bedrock were determined by performing consolidated and drained direct shear tests in conformance with ASTM D3080/D3080M-11. The tests were performed in a strain-controlled machine manufactured by GeoMatic. The rate of deformation was 0.01 inches per minute. Samples were sheared under varying confining pressures, as shown on the "Shear Test Diagrams," B-Plates. The moisture conditions during testing are shown on the following table and on the B-Plates. The samples indicated as saturated were artificially saturated in the laboratory. All saturated samples were sheared under submerged conditions.

2500 Townsgate Road, Suite E, Westlake Village, California 91361
(805) 370-1338 FAX (805) 371-4693



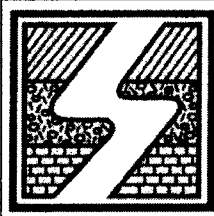
SL14.1670
June 25, 2014

Shear Strength (continued)

Test Pit/ Boring No.	Sample Depth (Feet)	Dry Density (pcf)	As-Tested Moisture Content (percent)
TP4	0-6"	106.2	20.7
TP2	2	117.8	16.4
TP4	1.5-2	118.6	17.8

Consolidation

One-dimensional consolidation tests were performed on samples of the alluvium in a consolidometer manufactured by GeoMatic in conformance with ASTM D2435/D2435M-11. The tests were performed on 1-inch high samples retained in brass rings. The samples were initially loaded to approximately ½ of the field over-burden pressure and then unloaded to compensate for the effects of possible disturbance during sampling. Loads were then applied in a geometric progression and resulting deformation recorded. Water was added at a specific load to determine the effect of saturation. The results are plotted on the "Consolidation Test," C-Plates.



**SOIL
LABWORKS LLC**

SHEAR DIAGRAM B-1

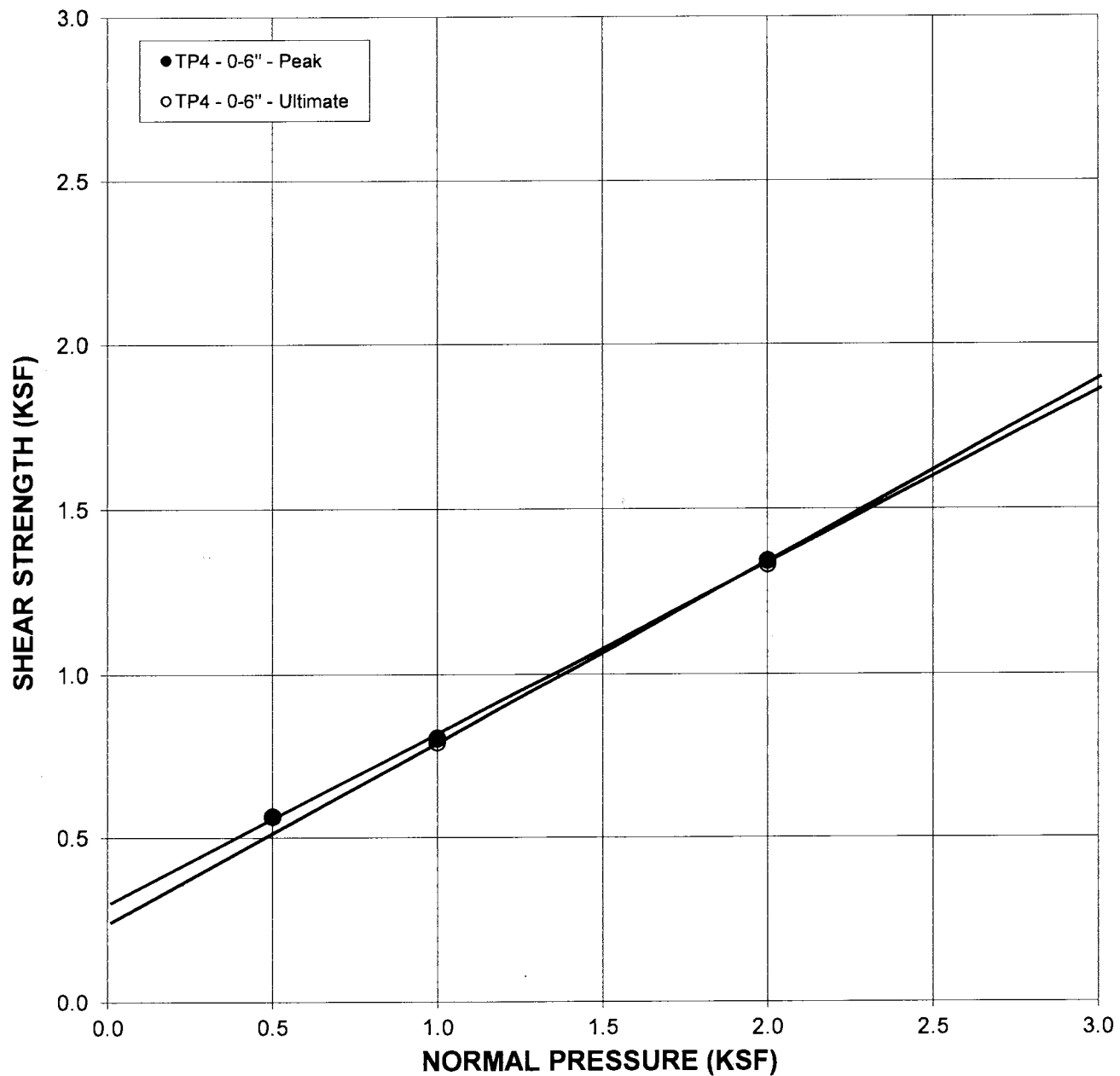
JN: SL14.1670 CONSULTANT JAI
CLIENT: Feffer/Camp Ramah - 385 Fairview Road

EARTH MATERIAL: COLLUVIUM

	PEAK	ULTIMATE	
Phi Angle	27	28	degrees
Cohesion	305	250	psf

Average Moisture Content	20.7%
Average Dry Density (pcf)	106.2
Percent Saturation	98.5%

DIRECT SHEAR TEST - ASTM D-3080





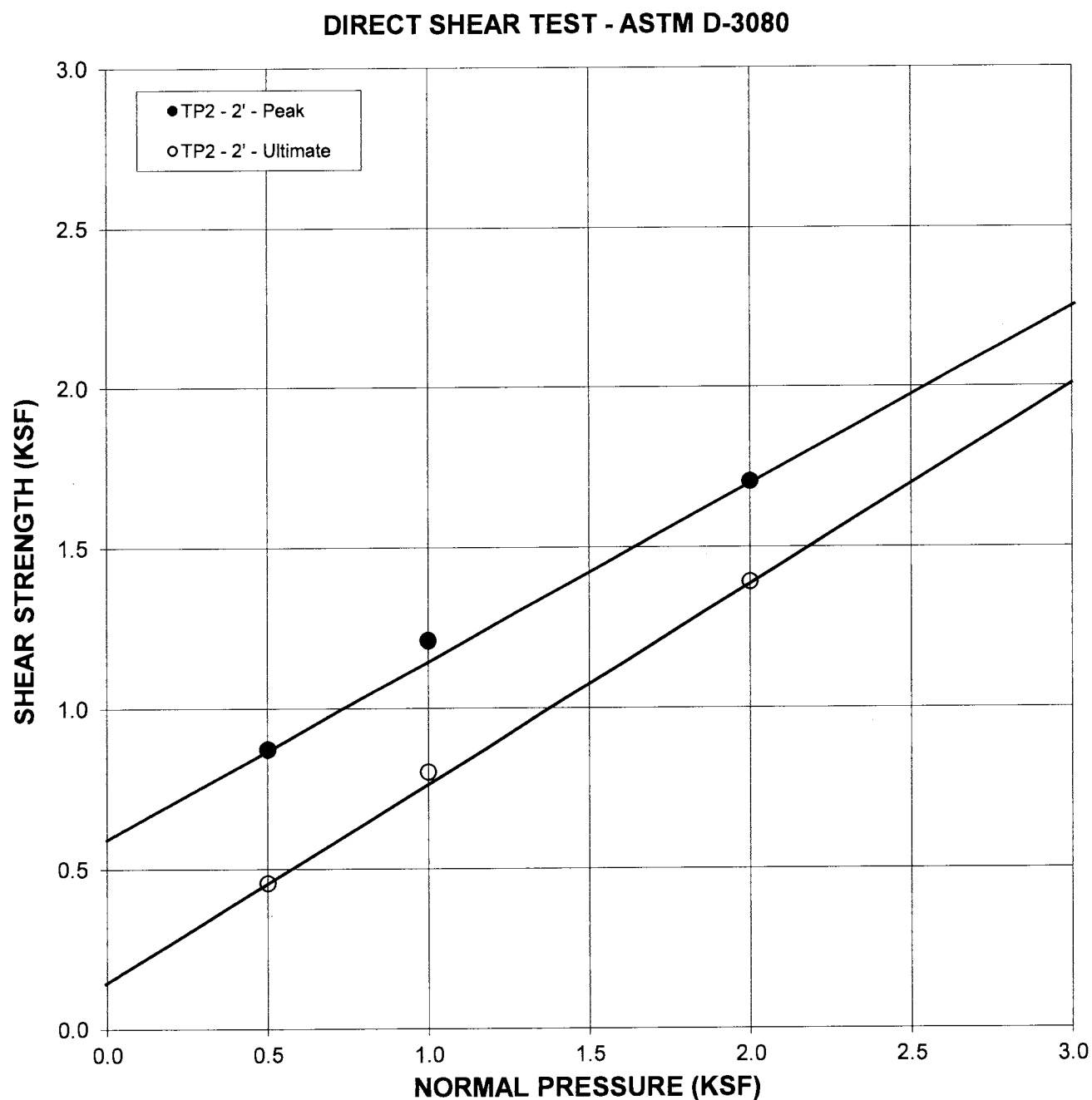
SHEAR DIAGRAM B-2

JN: SL14.1670 CONSULTANT JAI
CLIENT: Feffer/Camp Ramah - 385 Fairview Road

EARTH MATERIAL: ALLUVIUM

	PEAK	ULTIMATE	
Phi Angle	28.5	31.5	degrees
Cohesion	595	140	psf

Average Moisture Content	16.4%
Average Dry Density (pcf)	117.8
Percent Saturation	100.0%





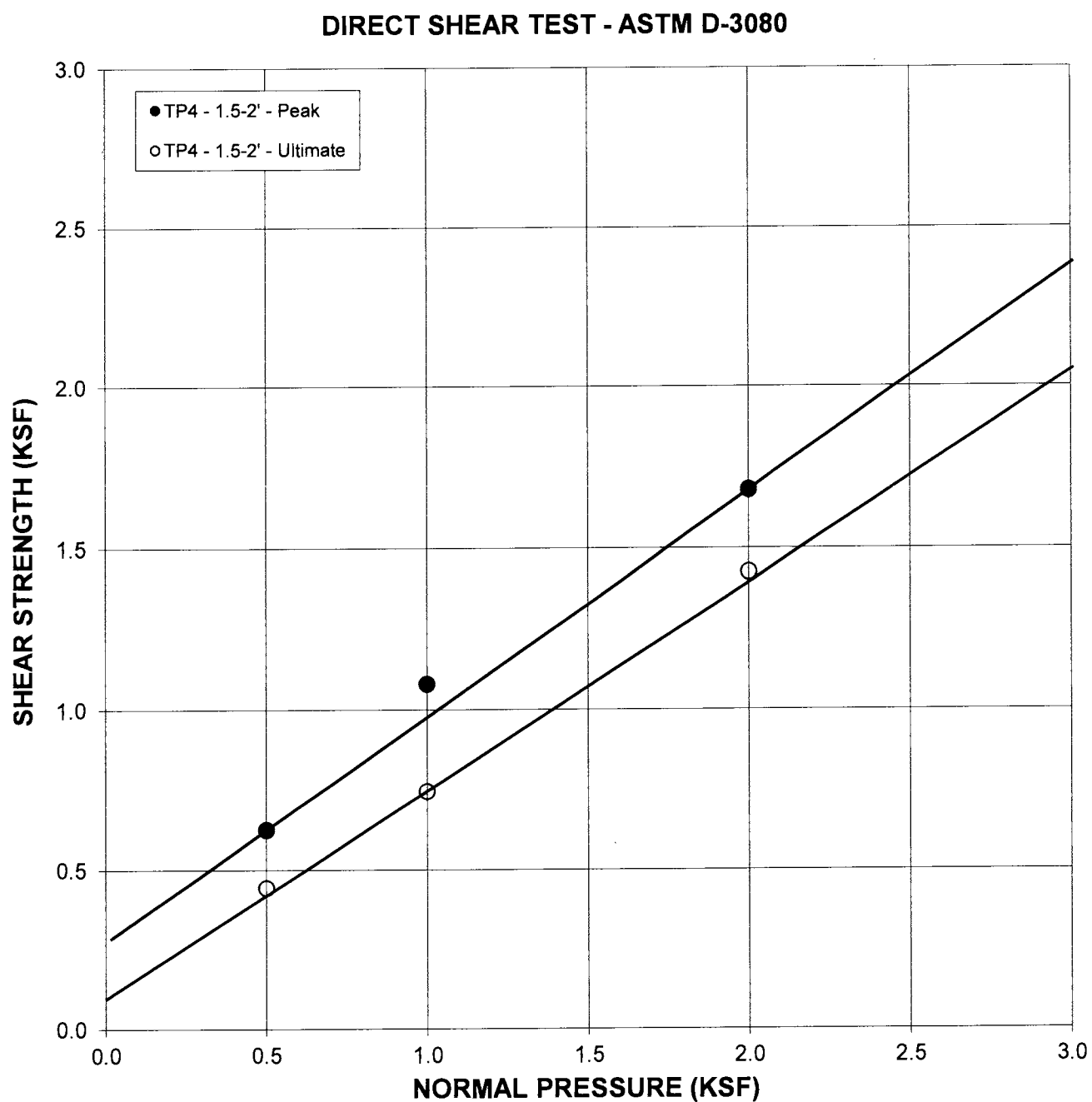
SHEAR DIAGRAM B-3

JN: SL14.1670 CONSULTANT JAI
CLIENT: Feffer/Camp Ramah - 385 Fairview Road

EARTH MATERIAL: BEDROCK

	PEAK	ULTIMATE	
Phi Angle	35	33	degrees
Cohesion	275	90	psf

Average Moisture Content	17.8%
Average Dry Density (pcf)	118.6
Percent Saturation	100.0%

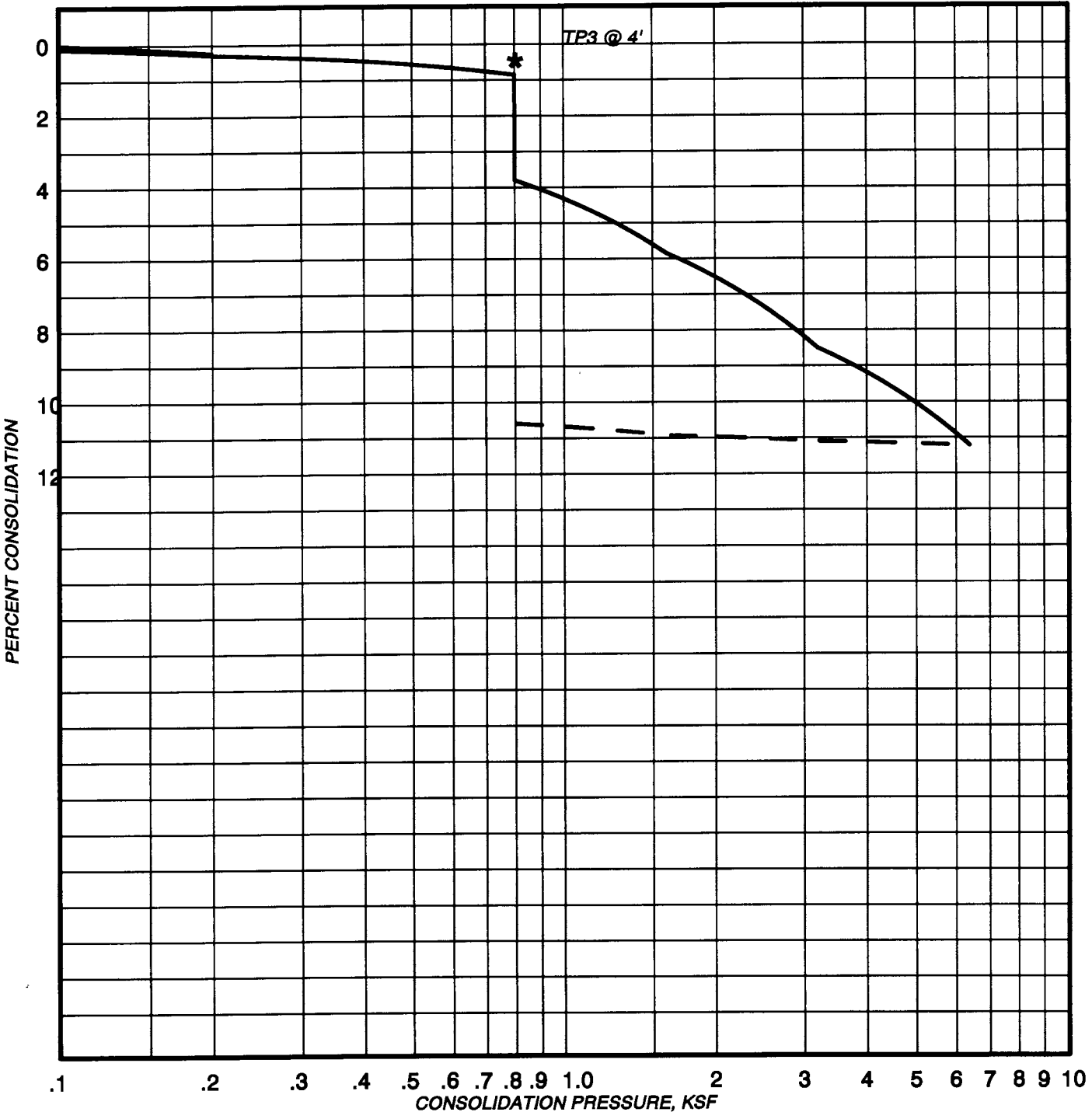


CONSOLIDATION TEST

PROJECT: FEFFER/CAMP RAMAH

SAMPLE: TP3 @ 4'

ALLUVIUM



* Water Added

PLATE: C-1



Table 1 - Laboratory Tests on Soil Samples

Feffer Geological
Camp Ramah
Your #1426-44, HDR Lab #14-0459LAB
30-Jun-14

Sample ID

@ 0-3'

Chemical Analyses

Cations

calcium	Ca ²⁺	mg/kg	na
magnesium	Mg ²⁺	mg/kg	na
sodium	Na ¹⁺	mg/kg	na
potassium	K ¹⁺	mg/kg	na

Anions

carbonate	CO ₃ ²⁻	mg/kg	na
bicarbonate	HCO ₃ ¹⁻	mg/kg	na
fluoride	F ¹⁻	mg/kg	na
chloride	Cl ¹⁻	mg/kg	na
sulfate	SO ₄ ²⁻	mg/kg	6.9
phosphate	PO ₄ ³⁻	mg/kg	na

Other Tests

ammonium	NH ₄ ¹⁺	mg/kg	na
nitrate	NO ₃ ¹⁻	mg/kg	na
sulfide	S ²⁻	qual	na
Redox		mV	na

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

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

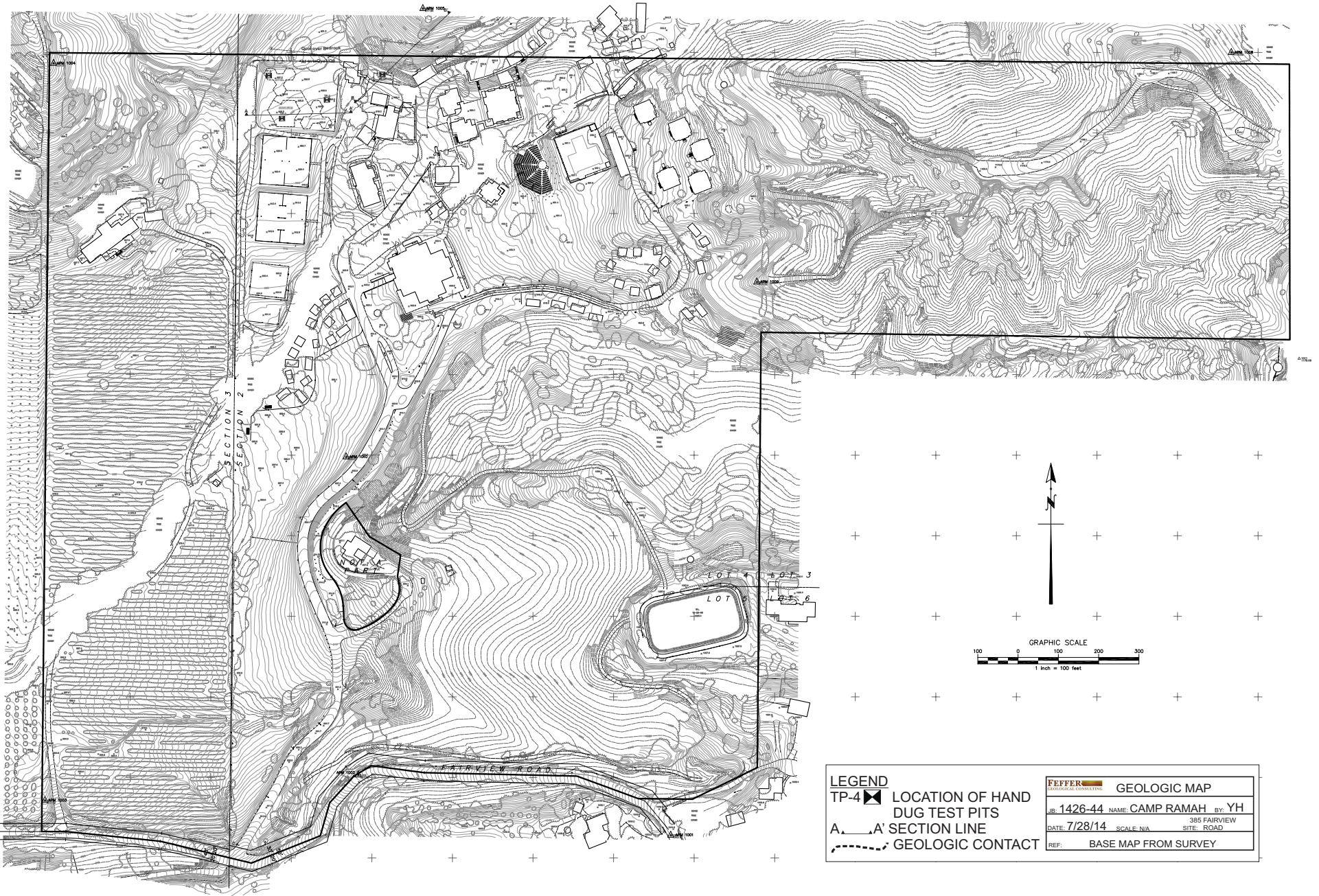
Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX ‘C’

Geologic Map & Cross Sections



LEGEND

TP-4  LOCATION OF HAND DUG TEST PITS

A-A'  SECTION LINE

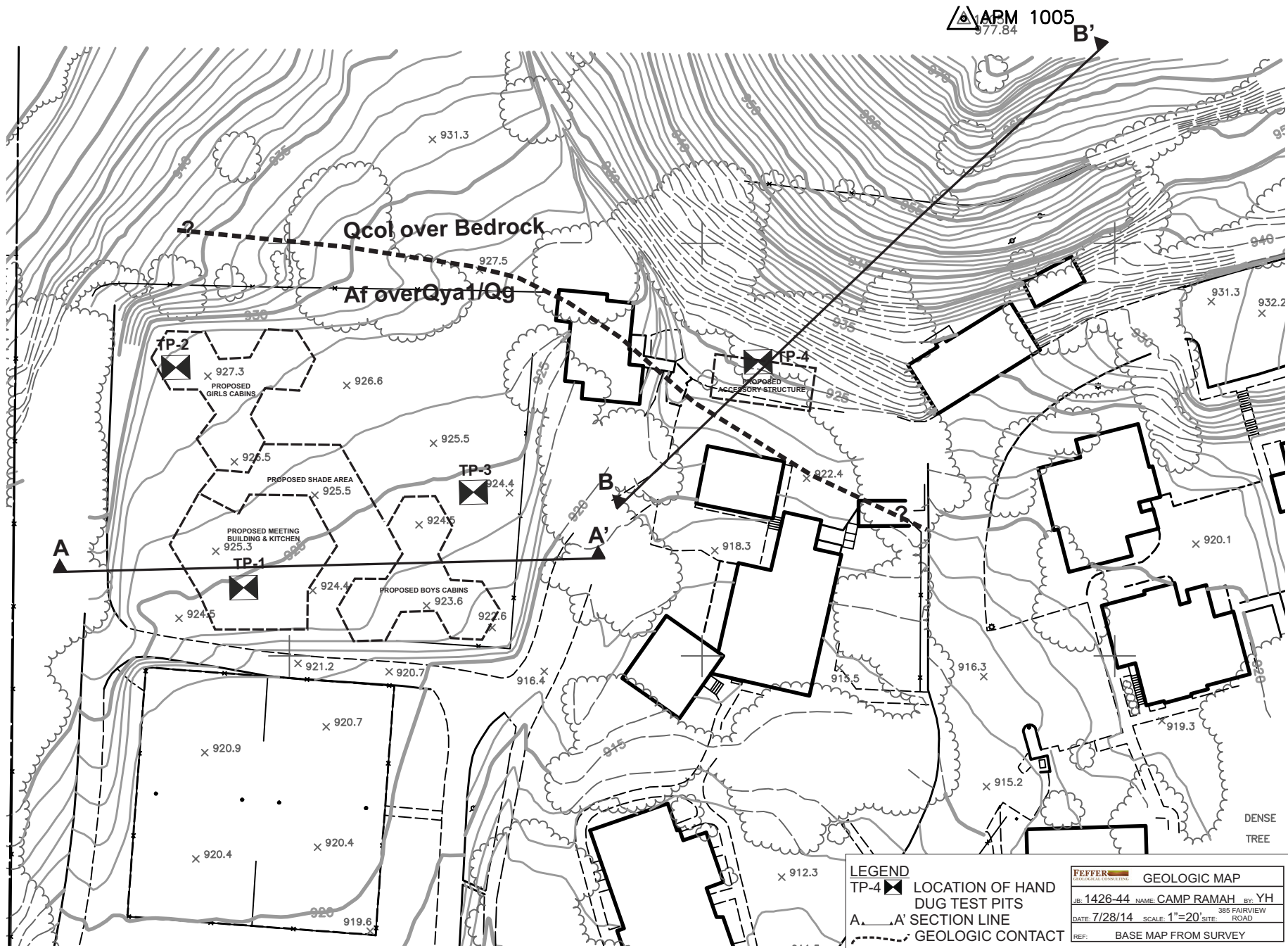
 GEOLOGIC CONTACT

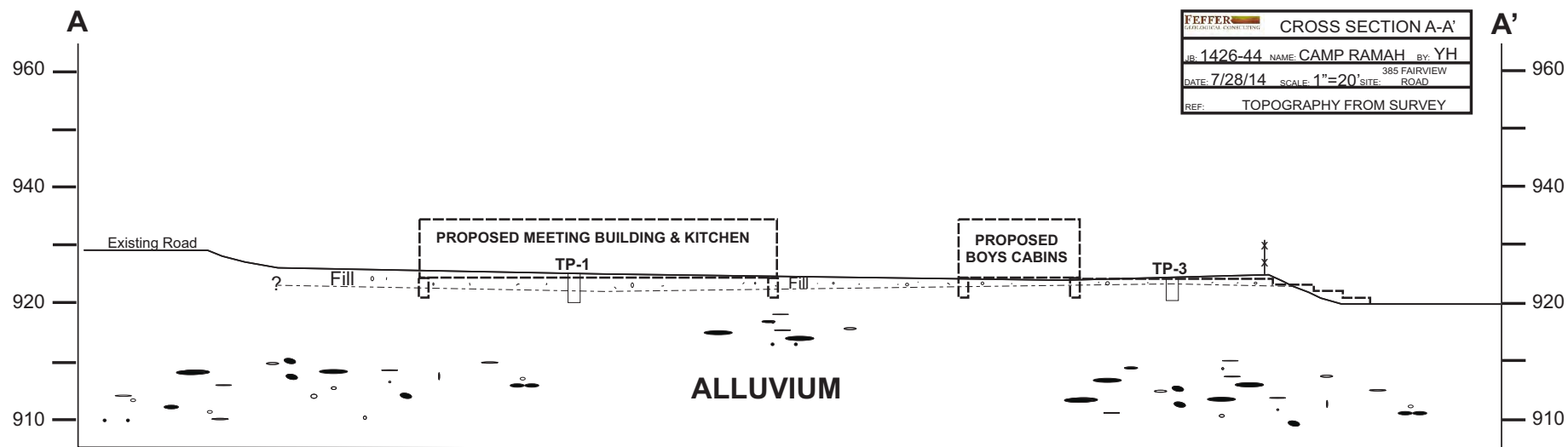
GEOLOGIC MAP

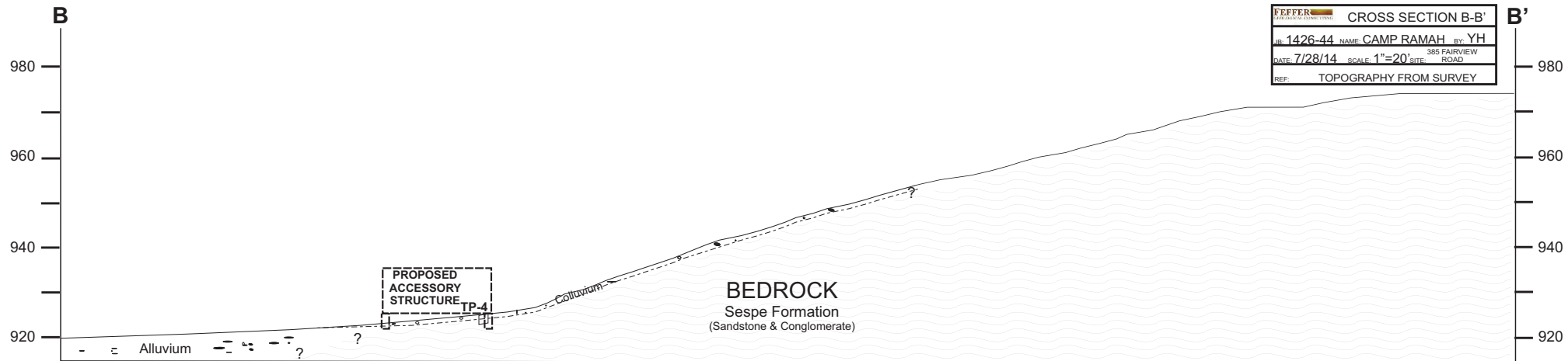
JB 1426-44 NAME: CAMP RAMAH BY YH

DATE: 7/28/14 SCALE: N/A SITE: ROAD

REF: BASE MAP FROM SURVEY







APPENDIX 'D'

Grading Specifications

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for grading operations performed under our supervision.

GENERAL

- 1) The Geotechnical Engineer and Engineering Geologist are the developer's representative on the project.
- 2) All clearing, site preparation or earth work performed on the project shall be conducted by the contractor under the supervision of the Geotechnical Engineer.
- 3) It is the contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Engineer and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Engineer. The contractor shall also remove all material considered unsatisfactory by the Geotechnical Engineer.
- 4) It is the contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction. Sufficient watering apparatus will also be provided by the contractor, with due consideration for the fill material, rate of placement and time of year.
- 5) A final report shall be issued by our firm outlining the contractor's conformance with these specifications.

SITE PREPARATION

- 1) All vegetation and deleterious materials such as rubbish shall be disposed of off-site. Soil, alluvium or rock materials determined by the Geotechnical Engineer as being unsuitable for placement in compacted fills shall be removed and wasted from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Engineer.
- 2) The Engineer shall locate all houses, sheds, sewage disposal systems, large trees or structures on the site or on the grading plan to the best of his knowledge prior to preparing the ground surface.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, or others not located prior to grading are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

3) After the ground surface to receive fill has been cleared, it shall be scarified, disced or bladed by the contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than twelve inches (12") in depth, the excess shall be removed and placed in lifts restricted to six inches (6").

Prior to placing fill, the ground surface to receive fill shall be inspected, tested and approved by the Geotechnical Engineer.

PLACING, SPREADING AND COMPACTION OF FILL MATERIALS

1) The selected fill material shall be placed in layers which when compacted shall not exceed six inches (6") in thickness. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to insure uniformity of material and moisture of each layer.

2) Where the moisture content of the fill material is below the limits specified by the Geotechnical Engineer, water shall be added until the moisture content is as required to assure thorough bonding and thorough compaction.

3) Where the moisture content of the fill material is above the limits specified by the Geotechnical Engineer, the fill materials shall be aerated by blading or other satisfactory methods until the moisture content is adequate.

COMPACTED FILLS

1) Any material imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches or other matter missed during clearing shall be removed from the fill as directed by the Geotechnical Engineer.

2) Rock fragments less than six inches (6") in diameter may be utilized in the fill, provided:

- a) They are not placed in concentrated pockets.
- b) There is a sufficient percentage of fine-grained material to surround the rocks.
- c) The distribution of the rocks is supervised by the Geotechnical Engineer.

3) Rocks greater than six inches (6") in diameter shall be taken off-site, or placed in accordance with the recommendations of the Geotechnical Engineer in areas designated as suitable for rock disposal. Details for rock disposal such as location, moisture control, percentage of rock placed, will be referred to in the "Conclusions and Recommendations" section of the geotechnical report.

If the rocks greater than six inches (6") in diameter were not anticipated in the preliminary geotechnical and geology report, rock disposal recommendations may not have been made in the "Conclusions and Recommendations" section. In this case, the contractor shall notify the Geotechnical Engineer if rocks greater than six inches (6") in diameter are encountered. The Geotechnical Engineer will then prepare a rock disposal recommendation or request that such rocks be taken off-site.

4) Representative samples of materials to be utilized as compacted fill shall be analyzed in the laboratory by the Geotechnical Engineer to determine their physical properties. If any materials other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as possible.

Material that is spongy, subject to decay or otherwise considered unsuitable shall not be used in the compacted fill.

5) Each layer shall be compacted to a minimum of ninety percent (90%) of the maximum density in compliance with the testing method specified by the controlling governmental agency (ASTM D-1557).

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soil conditions, the area to receive fill compacted to less than ninety percent (90%) shall either be delineated on the grading plan or appropriate reference made to the area in the geotechnical report.

6) Compaction shall be by sheeps foot roller, multi-wheeled pneumatic tire roller, or other types of acceptable rollers. Rollers shall be of such design that they will be able to compact the fill to the specified density. Rolling shall be accomplished while the fill material is at the specified moisture content. The final surface of the lot areas to receive slabs-on-grade should be rolled to a smooth, firm surface.

7) Field density tests shall be made by the Geotechnical Engineer of the compaction of each layer of fill. Density tests shall be made at intervals not to exceed two feet (2') of fill height provided all layers are tested. Where the sheeps foot rollers are used, the soil may be disturbed to a depth of several inches and density readings shall be taken in the compacted material below the disturbed surface. When these readings indicate the density of any layer of fill or portion thereof is below the required ninety percent (90%) density, the particular layer or portion shall be reworked until the required density has been obtained.

8) Buildings shall not span from cut to fill. Cut areas shall be over excavated and compacted to provide a fill mat of three feet (3').

FILL SLOPES

1) All fills shall be keyed and benched through all top soil, colluvium, alluvium, or creep material into sound bedrock or firm material where the slope receiving fill exceeds a ratio of five (5) horizontal to one (1) vertical, in accordance with the recommendations of the Geotechnical Engineer.

2) The key for side hill fills shall be a minimum of fifteen feet (15') within bedrock or firm materials, unless otherwise specified in the geotechnical report.

3) Drainage terraces and subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, or with the recommendations of the Geotechnical Engineer.

4) The Contractor will be required to obtain a minimum relative compaction of ninety percent (90%) out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either over-building

the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure which produces the required compaction.

5) All fill slopes should be planted or protected from erosion by methods specified in the geotechnical report and by the governing agency.

6) Fill-over-cut slopes shall be properly keyed through topsoil, colluvium, or creep material into rock or firm materials. The transition zone shall be stripped of all soil prior to placing fill.

CUT SLOPES

1) The Engineering Geologist shall inspect all cut slopes excavated in rock, lithified, or formation material at vertical intervals not exceeding ten feet (10').

2) If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints, or fault planes, are encountered during grading, these conditions shall be analyzed by the Engineering Geologist and Geotechnical Engineer; and recommendations shall be made to treat these problems.

3) Cut slope that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erosive interceptor swale placed at the top of the slope.

4) Unless otherwise specified in the geological and geotechnical report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of the controlling governmental agencies.

5) Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Engineer or Engineering Geologist.

GRADING CONTROL

1) Inspection of the fill placement shall be provided by the Geotechnical Engineer during the progress of grading.

2) In general, density tests should be made at intervals not exceeding two feet (2') of fill height or every five hundred (500) cubic yards of fill placed. These criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

3) Density tests should also be made on the surface materials to receive fill as required by the Geotechnical Engineer.

4) All clean-out, processed ground to receive fill, key excavations, subdrains, and rock disposal must be inspected and approved by the Geotechnical Engineer prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Engineer when such areas are ready for inspection.

CONSTRUCTION CONSIDERATIONS

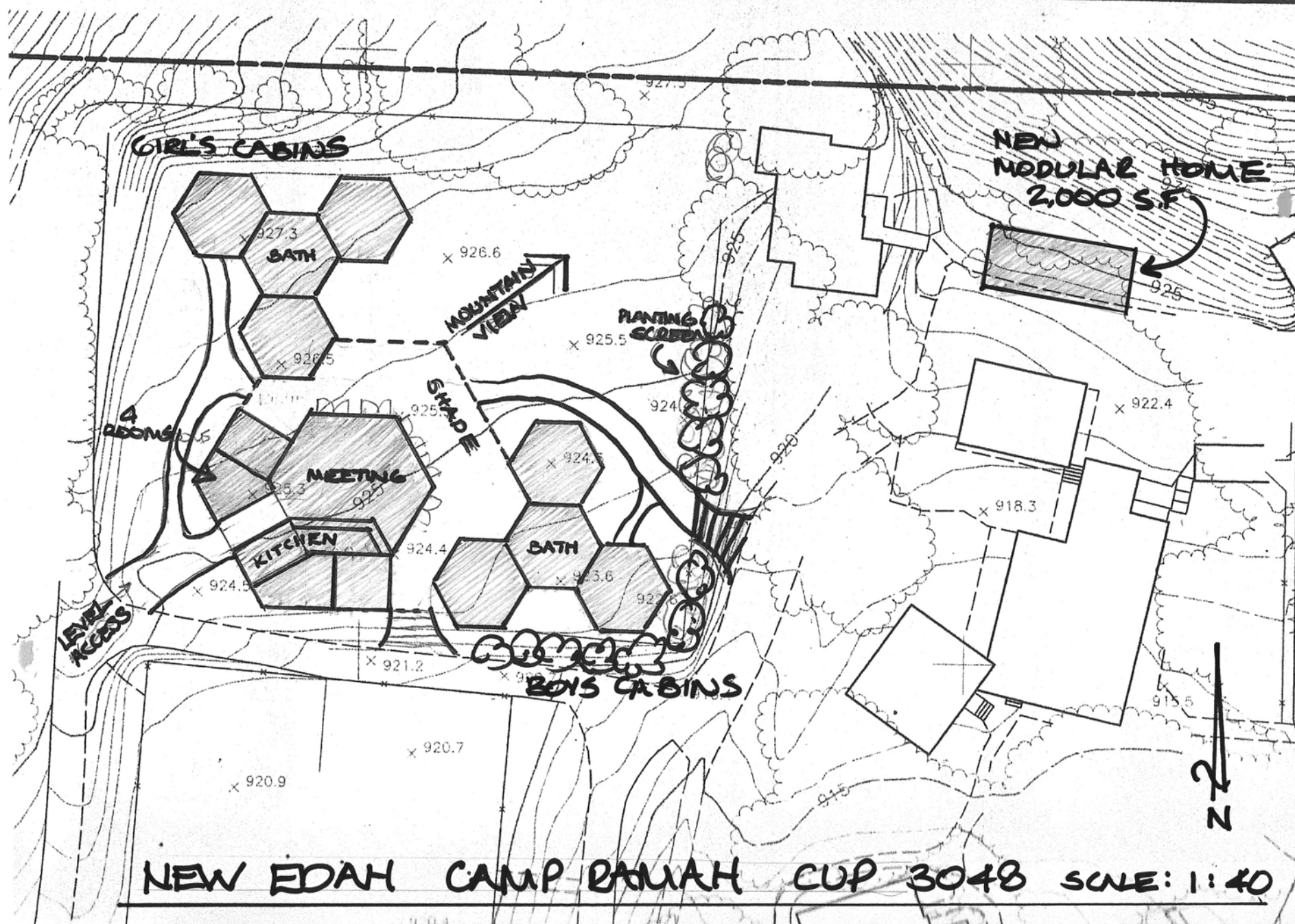
1) Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

2) Upon completion of grading and termination of inspections by the Geotechnical Engineer, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Engineer or Engineering Geologist.

3) Care shall be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

APPENDIX 'E'

Architectural Development Plans





NEW EDAM RADIATION SCALE 1:20
EAST

APPENDIX 'F'

Engineering Analysis



STABILITY - TAYLOR'S METHOD

IC: 1426-44 CONSULT: YMH
CLIENT: CAMP RAMAH

CALCULATION SHEET #

CALCULATE THE MAXIMUM HEIGHT TO WHICH UNIFORM SLOPES ARE GROSSLY STABLE USING TAYLOR'S METHOD FOR THE STABILITY OF EARTHEN EMBANKMENTS (*FUNDAMENTALS OF SOIL MECHANICS*).

CALCULATION PARAMETERS

EARTH MATERIAL:	BEDROCK	SAFETY FACTOR:	1.7
SHEAR DIAGRAM:	B3	SLOPE ANGLE:	26.6 degrees
COHESION:	275 psf	Cd Base (C/fs):	161.8 psf
PHI ANGLE:	35 degrees	PhiD = atan(tan(phi)fs) =	22.4 degrees
DENSITY (w):	139 pcf		

INTERPOLATE STABILITY NUMBER (sn) FROM TAYLOR'S CHARTS:

TAYLOR'S CHART

		SLOPE ANGLES						
Degrees		20	30	40	50	60	70	80
PhiD	5	0.090	0.110	0.130	0.145	0.160	0.185	0.210
	10	0.045	0.075	0.100	0.120	0.140	0.160	0.188
	15	0.020	0.045	0.070	0.095	0.115	0.140	0.168
	20	0.000	0.025	0.050	0.075	0.098	0.120	0.150
	25	0.000	0.010	0.033	0.055	0.080	0.105	0.130

FROM CHART sn = 0.012

SAFE SLOPE HEIGHT = $\frac{Cd}{w \times (sn)}$ 98.8 feet

CONCLUSIONS:

THE CALCULATION INDICATES THAT UNIFORM 2:1 SLOPES IN BEDROCK ARE STABLE (FS > 1.7) UP TO 98.8 FEET. THEREFORE, THE EXISTING 57 FOOT HIGH 3:1 SLOPE IS GROSSLY STABLE.

FEFFER

GEOLOGICAL CONSULTING

October 16, 2017

File No. 1426-44

Camp Ramah in California
17525 Ventura Blvd #201
Encino, CA 91316

Att: Randy Michaels, Director of Finance & Administration

Subject: **ADDENDUM LETTER FOR UPDATED PLANS**
385 Fairview Road, Ojai, CA 93024

Reference: **GEOTECHNICAL INVESTIGATION**
Proposed Construction of New Residence Buildings and Accessory Structure
385 Fairview Road, Ojai, CA 93024
By Feffer Geological Consulting, dated July 7, 2014

Dear Mr. Michaels,

Feffer Geological previously prepared a report for the construction of New Residence Buildings and Accessory Structure for the subject site. The proposed construction has not occurred at the subject site.

New updated plans have been prepared for an enlarged development that is now located farther to the north.

Three additional test pits were excavated on October 2, 2017 to a maximum depth of ten and a half feet the soils encountered in the test pits were consistent with the previous exploration; Quaternary Age Alluvium (Qyal) and Sespe Formation (Tsp) bedrock consisting of sandstone-conglomerate.

An updated geotechnical map and additional cross sections depicting the new development are appended to this addendum letter.

All of our previous recommendations remain the same from the report dated July 7, 2014.

Recommendations as provided in the original report are reiterated below.

All proposed footings shall be embedded within the alluvium or bedrock or a future compacted fill in accordance with the recommendations below.

New Structures

All proposed structural foundations shall be embedded within the alluvium or bedrock or a future compacted fill cap in accordance with the recommendations presented below. Geologic conditions on the site are favorable for the proposed construction. For an individual structure, all footings should be embedded in the same material (alluvium, bedrock, or new fill).

Foundation support for the new structures could be derived by utilizing conventional shallow foundations embedded within the alluvium or bedrock or a future compacted fill cap. Allowable design parameters for foundations are provided below.

Minimum depth for interior and exterior footing
(Measured from lowest adjacent grade).....2 feet
Minimum width.....1.5 feet

For Bedrock:

Bearing pressure
a. Sustained loads (lbs. per square foot)3,000 psf

Resistance to lateral loads
a. Passive soil resistance (lbs. per cubic ft.)
 Within bedrock..... 500 pcf
 Maximum allowable5,000 psf
b. Coefficient of sliding friction.....0.45

For Compacted Fill:

Bearing pressure
a. Sustained loads (lbs. per square foot)2,000 psf

Resistance to lateral loads
a. Passive soil resistance (lbs. per cubic ft.)
 Within compacted fill 250 pcf
 Maximum allowable2,500 psf
c. Coefficient of sliding friction.....0.35

For Alluvium:

Bearing pressure
a. Sustained loads (lbs. per square foot)2,000 psf

Resistance to lateral loads
a. Passive soil resistance (lbs. per cubic ft.)
 Within alluvium 250 pcf
 Maximum allowable2,500 psf
d. Coefficient of sliding friction.....0.30

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 3.0.

Increases in the bearing value of the Bedrock are allowable at a rate of 400 pounds per square foot for each additional foot of footing width to a maximum of 4,000 pounds per square foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

All continuous footings should be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footing excavations should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geologist and geotechnical engineer prior to placing forms, steel or concrete.

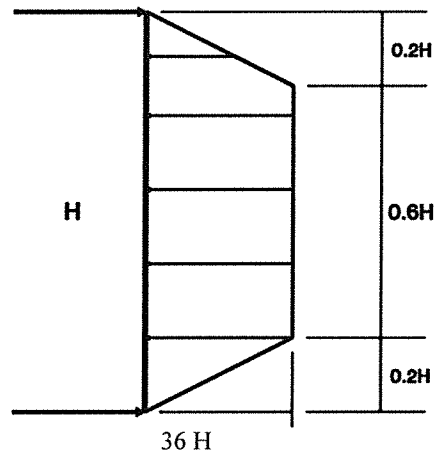
Based on the anticipated building loads footings designed and constructed in accordance with the soil criteria included within the referenced report are expected to settle less than $\frac{1}{4}$ to $\frac{1}{2}$ inch in a distance of 20 feet. Differential settlement is expected to be less than $\frac{1}{4}$ inch. The total and differential settlements are within acceptable and allowable tolerances for conventional foundations.

RETAINING WALLS

Cantilevered retaining walls up to 12 feet high that support fill, Older Alluvium, bedrock and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 43 pounds per cubic foot for level backslopes.

Restrained walls should be designed for an at-rest earth pressure of 60 pcf. Restrained braced retaining walls that are pinned at the top by a non-yielding floor should be designed for the trapezoidal pressure distribution noted in the figure below. The uniform trapezoidal pressure may be assumed over the central six tenths of the wall height. The pressure may be decreased to zero at the top and bottom of the wall.

TRAPEZOIDAL DISTRIBUTION OF PRESSURE



Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of $\frac{3}{4}$ inch crushed gravel.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls.

Cantilevered retaining walls higher than six feet need to consider a seismic surcharge from the Design Earthquake. According to the City of Los Angeles, the seismic surcharge should be calculated using a factor of safety of 1.0 with the PGA corresponding to $\frac{1}{2}$ of $\frac{2}{3}$ of the PGA_M . The PGA_M is 0.817 and therefore the corresponding seismic design value is 0.27g.

A seismic surcharge for retaining walls designed for active conditions is considered. For a 12 foot high retaining wall, the static design force is equal to 3.10 kips ($12ft^2 * 43 pcf / 2$).

For a ground motion of 0.27g and a FS of 1.0, the enclosed calculations indicate an unbalanced force under seismic conditions from the Maximum Considered Earthquake is 1814.5 pounds or 1.8 kips. Since the static design force is higher than the seismic force an additional seismic surcharge need not be added.

Waterproofing

Moisture affecting retaining walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water inside the building. Efflorescence is a process in which a powdery substance is produced on the surface of the concrete by the evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, and/or halite (common salt). Efflorescence is common to retaining walls and generally does not affect their strength or integrity.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A qualified waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to below grade walls.

As aforementioned, the architect, structural engineer, or other qualified waterproofing consultant should develop the actual waterproofing details.

Retaining Wall Drainage

Retaining walls that use a subdrain should have the subdrain pipe surrounded with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. The project structural engineer will incorporate an appropriately designed wall back-drain system for the purpose of mitigating potential for hydrostatic and/or seepage forces. Certain types of subdrain pipe are not acceptable to the various municipal agencies, it is recommended that prior to purchasing sub drainage pipe, the type and brand be verified and cleared with the proper municipal agencies. Sub drainage pipes should daylight and outlet to an acceptable location.

Retaining Wall Backfill

The onsite earth materials are acceptable for use as retaining wall backfill. Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90% (or 95%) of the maximum dry density obtained using test method ASTM D 1557-12 or equivalent. Flooding or jetting is not permitted. Proper compaction of the backfill will be necessary to reduce settlement of overlying walks and paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement, particularly at the points of entry to the structure.

Gravel or onsite earth materials will be utilized for backfill. If gravel is used, the upper 24 inches of backfill should consist of more cohesive material to minimize surface infiltration. Retaining wall backfill should be capped with a paved surface drain or pavement

It should be pointed out that the use of heavy compaction equipment in close proximity to retaining walls can result in excess wall movement and/or soil loadings exceeding design values. In this regard, care should be taken during backfilling operations.

TEMPORARY EXCAVATIONS

All vertical cuts shall be inspected by our office to verify geologic continuity.

Un-shored vertical cuts to a height of eight feet (8') may be made in earth materials at the site. Un-shored cuts in excess of eight feet (8') shall be sloped at a gradient of no steeper than 1:1 (horizontal to vertical) for the portion of the excavation above the vertical cut.

2016 CALIFORNIA BUILDING CODE CONSIDERATIONS

The proposed development may be designed in accordance with seismic considerations contained in the 2016 California Building Code, Section 1613, the following parameters may be considered for design:

Latitude and Longitude of Site (34.45866°N, 119.26489°W)

Mapped Spectral Response Acceleration Parameters:

	S_S	:	2.210
	S_1	:	0.835g
Site Class:	D	:	Stiff Soil
Site Coefficients:	F_a	:	1.0
	F_v	:	1.5

Maximum Considered Earthquake Spectral Response Acceleration Parameters:

S_{MS}	:	2.210g
S_{M1}	:	1.252g

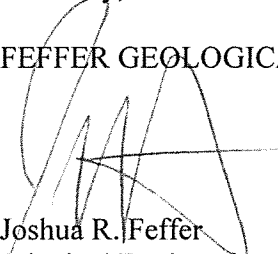
Design Spectral Response Acceleration Parameters:

S_{DS}	:	1.473g
S_{D1}	:	0.835g
PGA_M	:	0.817g

We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.


Sincerely,

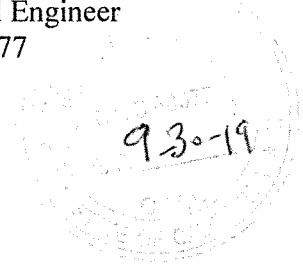
FEFFER GEOLOGICAL CONSULTING, INC.


Joshua R. Feffer
Principal Engineering Geologist
C.E.G. 2138



Distribution: Addressee- (1)


Dan Daneshfar
Principal Engineer
P.E. 68377



TEMPORARY EXCAVATION HEIGHT

IC: 1426-44 CONSULT: YMH
CLIENT: Camp Ramah

CALCULATION SHEET #

CALCULATE THE HEIGHT TO WHICH TEMPORARY EXCAVATIONS ARE STABLE (NEGATIVE THRUST).
THE EXCAVATION HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW.
ASSUME THE EARTH MATERIAL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE.

CALCULATION PARAMETERS

EARTH MATERIAL:	Bedrock	WALL HEIGHT:	12 feet
SHEAR DIAGRAM:	B2	BACKSLOPE ANGLE:	21 degrees
COHESION:	275 psf	SURCHARGE:	250 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	125 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	220.0 psf	FINAL TENSION CRACK:	30 feet
PHID = $ATAN(TAN(PHI)/FS)$ =	29.3 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	24.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	3056.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	5535 trials
LENGTH OF FAILURE PLANE	6.6 feet
DEPTH OF TENSION CRACK	4.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED HORIZONTAL THRUST	-49.1 pounds
CALCULATED EQUIVALENT FLUID PRESSURE	-1.5 pcf
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	8.0 feet

CONCLUSIONS:

THE CALCULATION INDICATES THAT THE TEMPORARY
EXCAVATIONS IN ALLUVIUM UP TO 8 FEET HIGH HAVE A NEGATIVE
THRUST AND ARE TEMPORARILY STABLE.

	RETAINING WALL																						
	IC: 1426-44 CONSULT: YMH CLIENT: Camp Ramah CALCULATION SHEET #																						
CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED TEMPORARY SHORING. THE HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOB-OKABE METHOD FOR SEISMIC FORCES.																							
CALCULATION PARAMETERS																							
EARTH MATERIAL: Bedrock SHEAR DIAGRAM: B2 COHESION: 275 psf PHI ANGLE: 35 degrees DENSITY 125 pcf SAFETY FACTOR: 1.5 WALL FRICTION 0 degrees CD (C/FS): 183.3 psf PHID = ATAN(TAN(PHI)/FS) = 25.0 degrees HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k _h) 0 %g VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k _v) 0 %g	WALL HEIGHT 12 feet BACKSLOPE ANGLE: 21 degrees SURCHARGE: 250 pounds SURCHARGE TYPE: U Uniform INITIAL FAILURE ANGLE: 30 degrees FINAL FAILURE ANGLE: 70 degrees INITIAL TENSION CRACK: 5 feet FINAL TENSION CRACK: 40 feet																						
CALCULATED RESULTS																							
<table style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 60%;">CRITICAL FAILURE ANGLE</td> <td style="text-align: right;">51 degrees</td> </tr> <tr> <td>AREA OF TRIAL FAILURE WEDGE</td> <td style="text-align: right;">80.5 square feet</td> </tr> <tr> <td>TOTAL EXTERNAL SURCHARGE</td> <td style="text-align: right;">1500.0 pounds</td> </tr> <tr> <td>WEIGHT OF TRIAL FAILURE WEDGE</td> <td style="text-align: right;">11564.1 pounds</td> </tr> <tr> <td>NUMBER OF TRIAL WEDGES ANALYZED</td> <td style="text-align: right;">1476 trials</td> </tr> <tr> <td>LENGTH OF FAILURE PLANE</td> <td style="text-align: right;">17.5 feet</td> </tr> <tr> <td>DEPTH OF TENSION CRACK</td> <td style="text-align: right;">2.6 feet</td> </tr> <tr> <td>HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK</td> <td style="text-align: right;">11.0 feet</td> </tr> <tr> <td>CALCULATED HORIZONTAL THRUST ON WALL</td> <td style="text-align: right;">2404.3 pounds</td> </tr> <tr> <td>CALCULATED EQUIVALENT FLUID PRESSURE</td> <td style="text-align: right;">33.4 pcf</td> </tr> <tr> <td>DESIGN EQUIVALENT FLUID PRESSURE</td> <td style="text-align: right;">43.0 pcf</td> </tr> </table>		CRITICAL FAILURE ANGLE	51 degrees	AREA OF TRIAL FAILURE WEDGE	80.5 square feet	TOTAL EXTERNAL SURCHARGE	1500.0 pounds	WEIGHT OF TRIAL FAILURE WEDGE	11564.1 pounds	NUMBER OF TRIAL WEDGES ANALYZED	1476 trials	LENGTH OF FAILURE PLANE	17.5 feet	DEPTH OF TENSION CRACK	2.6 feet	HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	11.0 feet	CALCULATED HORIZONTAL THRUST ON WALL	2404.3 pounds	CALCULATED EQUIVALENT FLUID PRESSURE	33.4 pcf	DESIGN EQUIVALENT FLUID PRESSURE	43.0 pcf
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RETAINING WALL

IC: 1426-44 CONSULT: YMH
CLIENT: Camp Ramah

CALCULATION SHEET #

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR PROPOSED TEMPORARY SHORING. THE HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. ASSUME THE BACKFILL IS SATURATED WITH NO EXCESS HYDROSTATIC PRESSURE. USE THE MONONOBÉ-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	Bedrock	WALL HEIGHT	12 feet
SHEAR DIAGRAM:	B2	BACKSLOPE ANGLE:	21 degrees
COHESION:	275 psf	SURCHARGE:	0 pounds
PHI ANGLE:	35 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	125 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION	0 degrees	INITIAL TENSION CRACK:	5 feet
CD (C/FS):	275.0 psf	FINAL TENSION CRACK:	40 feet
PHID = $ATAN(TAN(PHI)/FS)$ =	35.0 degrees		
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h)		0.27 %g	
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v)		0 %g	

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	70.1 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	8768.4 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1476 trials
LENGTH OF FAILURE PLANE	12.4 feet
DEPTH OF TENSION CRACK	5.5 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	8.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	1814.5 pounds

THE CALCULATION INDICATES THAT THE PROPOSED RETAINING WALL MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF
** POUNDS PER CUBIC FOOT.

GRAPHIC LOG			APPROXIMATE SCALE : 1"=5'	TEST EXCAVATION : 5
			DATE LOGGED : 10/2/17 BY : YH	ADDRESS: 385 Fairview Road
<div style="display: flex;"> <div style="writing-mode: vertical-rl; transform: rotate(180deg); font-weight: bold; margin-right: 10px;">DEPTH</div> <div style="flex-grow: 1;"> </div> </div>				
RING	BULK	SAMPLE DEPTH	LOCATION : Slope North Of Soccer Field	
			DESCRIPTION: Classification (USCS), color, moisture, consistency etc.	
1			<p>0-4' Alluvium: Gravelly silty sand, increase in gravel with depth, medium brown purple hue, increase in purple color with depth, slightly moist to dry, dense, roots in upper 1' rounded and sub-rounded gravel and cobbles up to 2", pores, organics</p> <p>4-4'8" Bedrock: Sandstone to matrix supported conglomerate, medium to coarse grained, purple, pink, brown, moist to dry, dense to hard, moderately weathered, friable</p> <p>End At 4'8", No Fill, No Water, No caving</p>	
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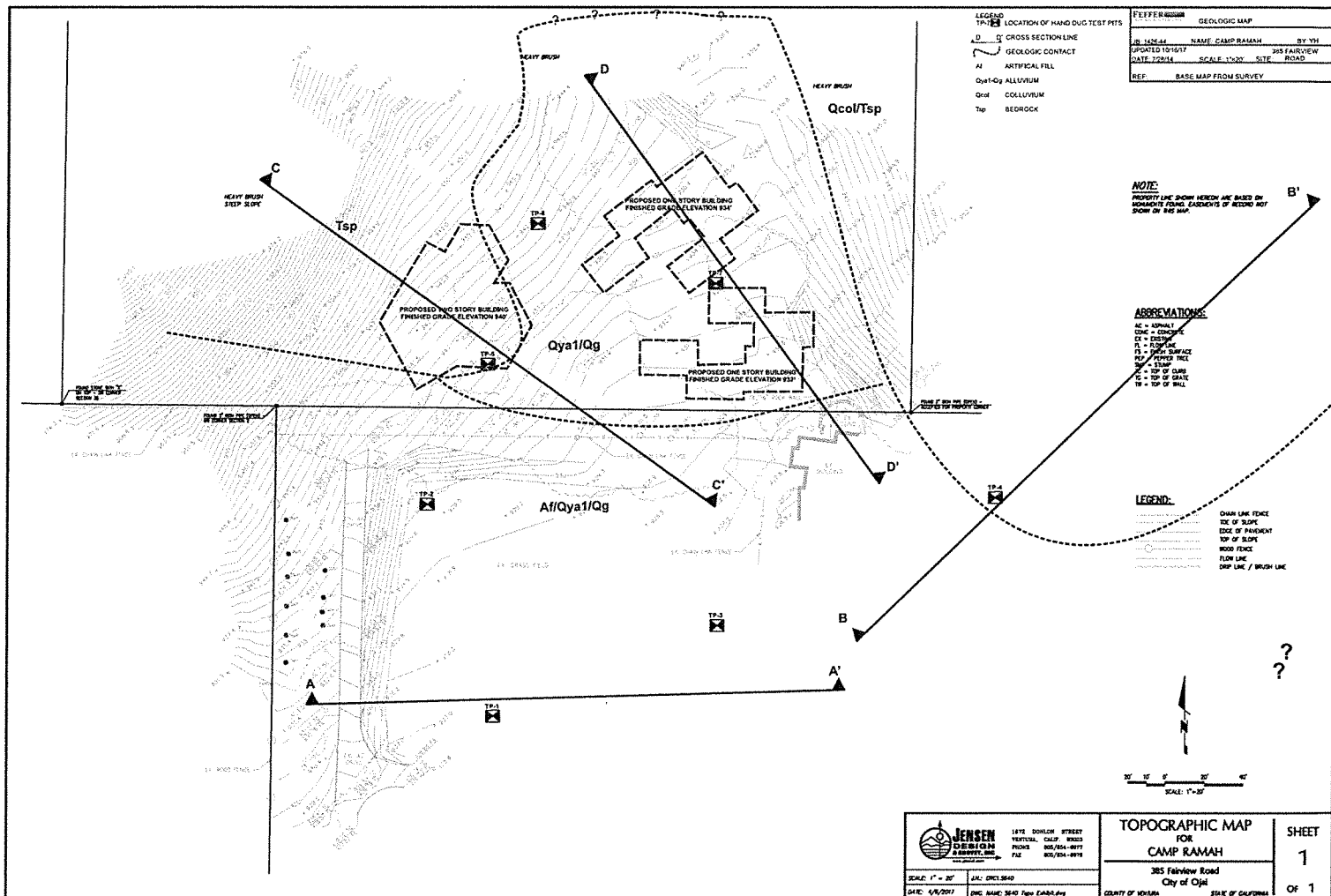
FEFFER GEO CONSULTING	F.N. 1426-44	Camp Ramah	PLATE
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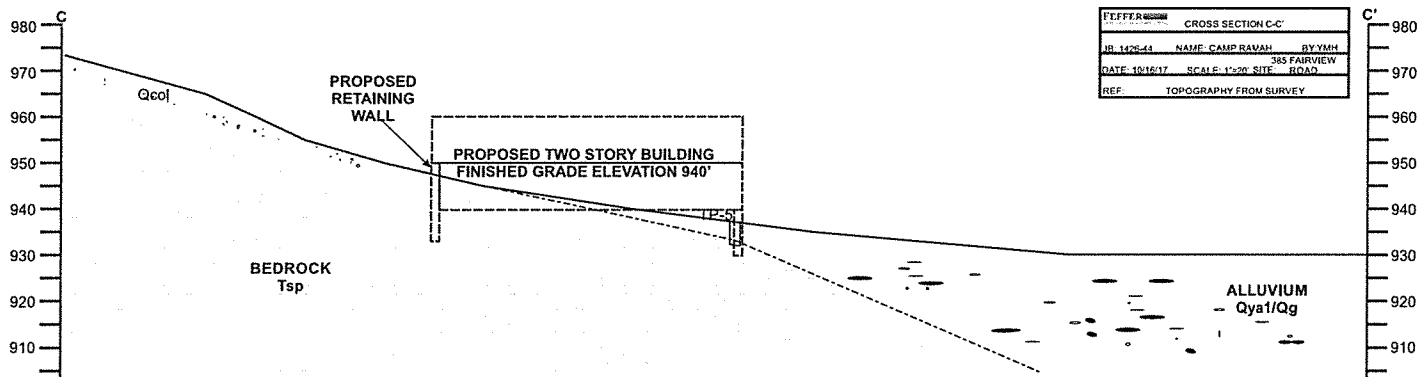
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		DATE LOGGED : 10/2/17 BY : YH		ADDRESS: 385 Fairview Road	
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RING	BULK	SAMPLE DEPTH	Blows	LOCATION : Slope North Of Soccer Field	
				DESCRIPTION: Classification (USCS), color, moisture, consistency etc.	
1				0-11' Alluvium:	
2				@2' Gravelly silty sand fine to medium grained, brown purple to pink hue, slightly moist to moist, dense, roots	
3					
4				@4' Gravelly silty sand, purple, slightly moist to moist, dense, caliche	
5					
6				@6' gravelly silty sand, dark purple, slightly moist to moist, dense, caliche	
7					
8				@8' Gravelly silty sand clay binder, purple, maroon, slightly moist to moist, dense, increase in gravel content, carbon	
9					
10				@10' Sandy layer slightly less gravel	
11				End At 11', No Fill, No Water, No caving	
12					
13					
14					

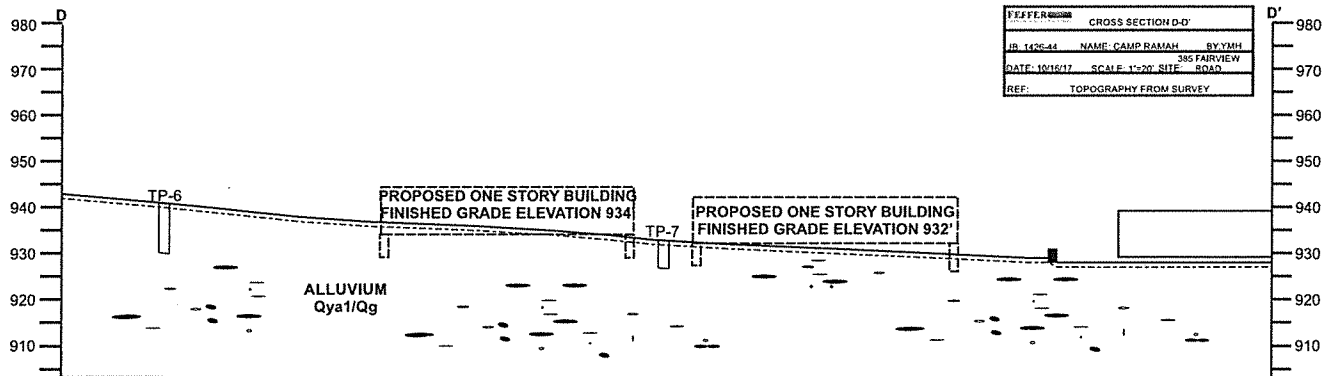
FEFFER GEO CONSULTING	F.N. 1426-44	Camp Ramah	PLATE
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GRAPHIC LOG			APPROXIMATE SCALE : 1"=5'	TEST EXCAVATION : 7
			DATE LOGGED : 10/2/17 BY : YH	ADDRESS: 385 Fairview Road
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RING	BULK	SAMPLE DEPTH	Blows	LOCATION : Slope North Of Soccer Field
				DESCRIPTION: Classification (USCS), color, moisture, consistency etc.
1				<p>0-6.5' Alluvium:</p> <p>@2' Silty sand with gravel, mottled, tan, brown, buff, moist, dense</p> <p>@4' Silty sand with gravel, tan, brown, slightly moist to dry, dense, subrounded gravel up to 4"</p> <p>@5' Change in color to purple, maroon</p> <p>@6' Gravelly silty sand clay binder, dark purple, maroon, slightly moist to moist, dense</p> <p>End At 6.5', No Fill, No Water, No caving</p>
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FEFFER GEO CONSULTING	F.N. 1426-44	Camp Ramah	PLATE
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FEFFER



GEOLOGICAL CONSULTING

October 29, 2018

File No. 1426-44

Camp Ramah in California
17525 Ventura Blvd #201
Encino, CA 91316

Att: Randy Michaels, Director of Finance & Administration

Subject: **RESPONSE TO VENTURA COUNTY**
 Determination of Application Incompleteness
 Major Modification to Conditional Use Permit (CUP) No. 3048
 Case No. PL18-0052, Camp Ramah

Reference: **GEOTECHNICAL INVESTIGATION**
 Proposed Construction of New Residence Buildings and Accessory Structure
 385 Fairview Road, Ojai, CA 93024
 By Feffer Geological Consulting, dated July 7, 2014

ADDENDUM LETTER FOR UPDATED PLANS
385 Fairview Road, Ojai, CA 93024
By Feffer Geological Consulting, dated October 16, 2017

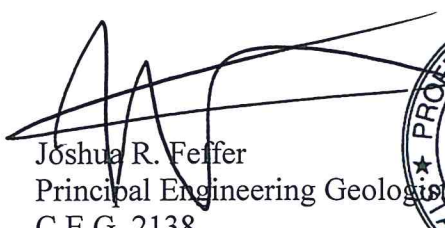
Dear Mr. Michaels,


The following is a response to questions by the County of Ventura. The items are reiterated below.

We appreciate the opportunity to be of service. Should you have any questions regarding the information contained in this report, please do not hesitate to contact us.

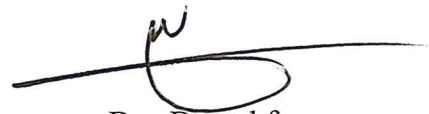
Sincerely,

FEFFER GEOLOGICAL CONSULTING, INC.


Joshua R. Feffer
Principal Engineering Geologist
C.E.G. 2138



Distribution: Addressee- (1)


Dan Daneshfar
Principal Engineer
P.E. 68377



Item 8

Geological Report: Please provide a signed copy of the report (Feffer Geological Consulting, dated July 7, 2014) by the responsible soil engineer and geologist. In addition, the plates in the back of the report do not appear to be at the correct scale. Please provide an updated, signed report, and plates to scale.

Response

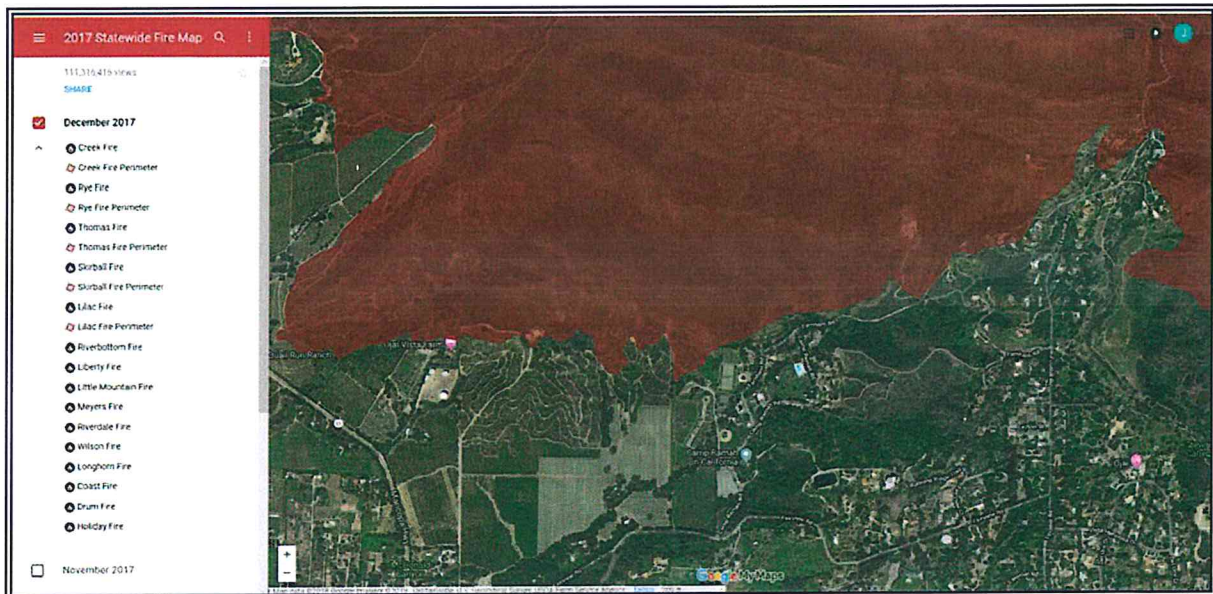
A copy of the 2014 report is attached to this response.

Item 9

Please address the hazard of debris flow with respect to the location of the proposed Machan village. The Feffer Report was prepared in 2017, prior to the occurrence of the Thomas Fire that affected the Camp Ramah site. Provide an updated report that addresses the debris flow impact and provide mitigation as necessary.

Response

Updated plans show the structures will be constructed into the base of the ascending slope. A Map showing the extent of the Thomas Fire Burn Area by Cal Fire and shown on <http://www.fire.ca.gov/general/firemaps> is shown below.



We visited the subject site and took photos of the area above the new proposed buildings. These photos are included at the back of this response. The slope above the proposed campground shows no evidence of recent burn, is in good condition, and covered with chaparral.

The subject site is not located within an "Area of Minimal Flood Zone" per FEMA maps and is outside of a flood zone per the County of Ventura GIS.

The proposed buildings are located several feet above the adjacent small channel and outside of the direct flow path. It is our opinion that the potential for debris flow is low and mitigation is not required.

Item 10

Please address the hydro-consolidation of the alluvial materials (3% upon saturation @ 800 pounds per square foot [psf]) of one of the higher in-place density alluvial materials and the recommendation to place 2,000 psf on this material. Please discuss and revise the 2014 and 2017 Feffer Geological Report, as necessary.

Response

The proposed two-story building is underlain by bedrock and all foundations should extend into the bedrock. For structures located in that area that is underlain by alluvium, the soil should be removed and recompacted to a minimum of 90% relative compaction to a depth of 3 feet below the proposed footings and five feet outside the building footprint.

Item 11

Please discuss why a City of Los Angeles standard has relevance on a project in the County of Ventura (Feffer Report dated October 16, 2017, page 4). Also, the seismic force is an added force to the static force. Thus, the last sentence under retaining walls discussion on page 4 should be explained and revised as necessary.

Response

Cantilevered retaining walls up to 12 feet high that support fill, Older Alluvium, bedrock and approved retaining wall backfill, may be designed for an equivalent fluid pressure of 43 pounds per cubic foot for level backslopes.

Restrained walls should be designed for an at-rest earth pressure of 60 pcf. The increase in lateral pressure due to earthquake loading can be estimated using the Mononobe-Okabe theory, as described by Seed and Whitman (1970). The estimated dynamic lateral force increase (due to seismic loading) for either restrained or unrestrained walls may be taken as $10H$ pounds per square foot of wall. The centroid of the dynamic lateral force increase should be applied at a distance of $0.6 \times H$ above the base of the wall. The distribution of the resultant dynamic lateral force can be assumed to be an invert triangle (base of the triangle at top of the wall).

To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using an active (not at-rest) lateral earth pressure of 43 pcf, equivalent fluid weight.



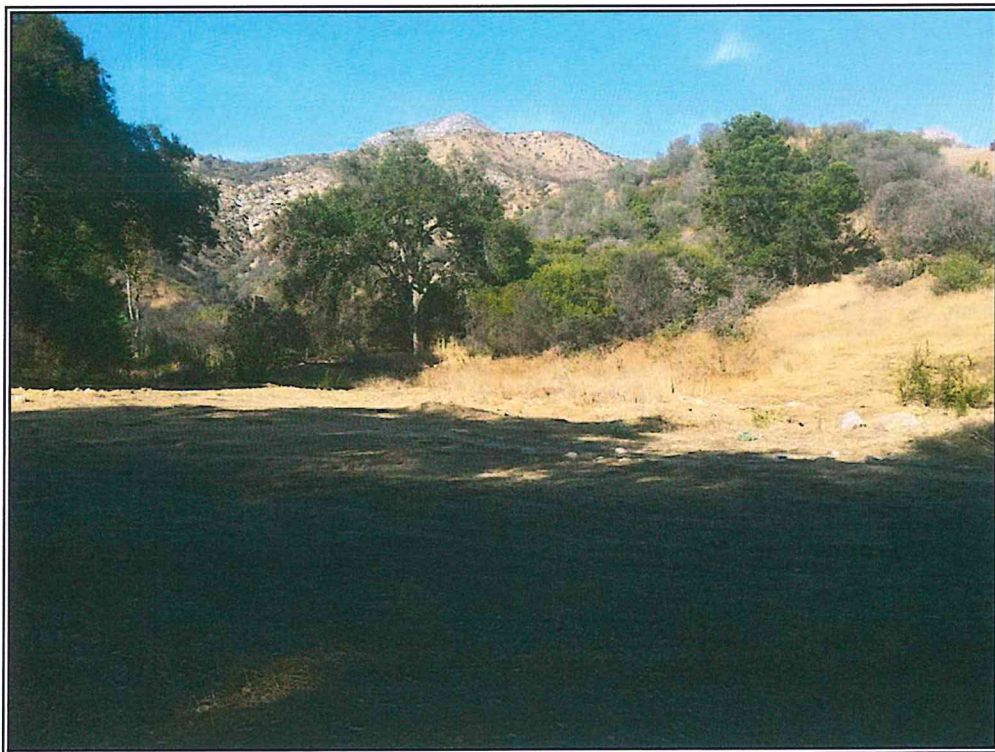
View of the slope above the proposed structures



View of the proposed construction area



View of the proposed construction area in relation to the small channel



View of the proposed construction area.

- c. Provide total water use from all sources from a representative base period of at least 10 years to allow for adequate water analysis. The water use data must be copies of the original Casitas water bills or printed on Casitas letterhead to serve as empirical evidence of actual water usage.
- d. Provide any metered groundwater extraction data if available.

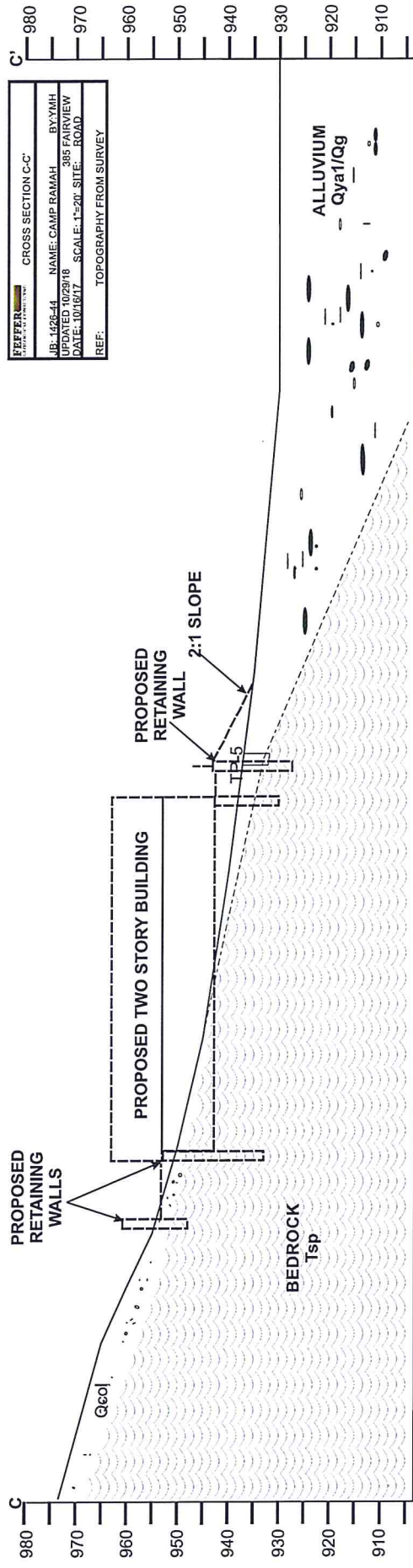
Ventura County Public Works Agency Engineering Services Department: Jim O'Tousa, (805) 654-2034 or jim.o'tousa@ventura.org

- 8. Geological Report: Please provide a signed copy of the report (Feffer Geological Consulting, dated July 7, 2014) by the responsible soil engineer and geologist. In addition, the plates in the back of the report do not appear to be at the correct scale. Please provide an updated, signed report and plates to scale.
- 9. Please address the hazard of debris flow with respect to the location of the proposed Machon village. The Feffer Report was prepared in 2017, prior to the occurrence of the Thomas Fire that affected the Camp Ramah site. Provide an updated report that addresses the debris flow impact and provide mitigation as necessary.
- 10. Please address the hydro-consolidation of the alluvial materials (3% upon saturation @ 800 pounds per square foot [psf]) of one of the higher in-place density alluvial materials and the recommendation to place 2,000 psf on this material. Please discuss and revise the 2014 and 2017 Feffer Geological Report, as necessary.
- 11. Please discuss why a City of Los Angeles standard has relevance on a project in the County of Ventura (Feffer Report dated October 16, 2017, page 4). Also the seismic force is an added force to the static force. Thus, the last sentence under retaining walls discussion on page 4 should be explained and revised as necessary.

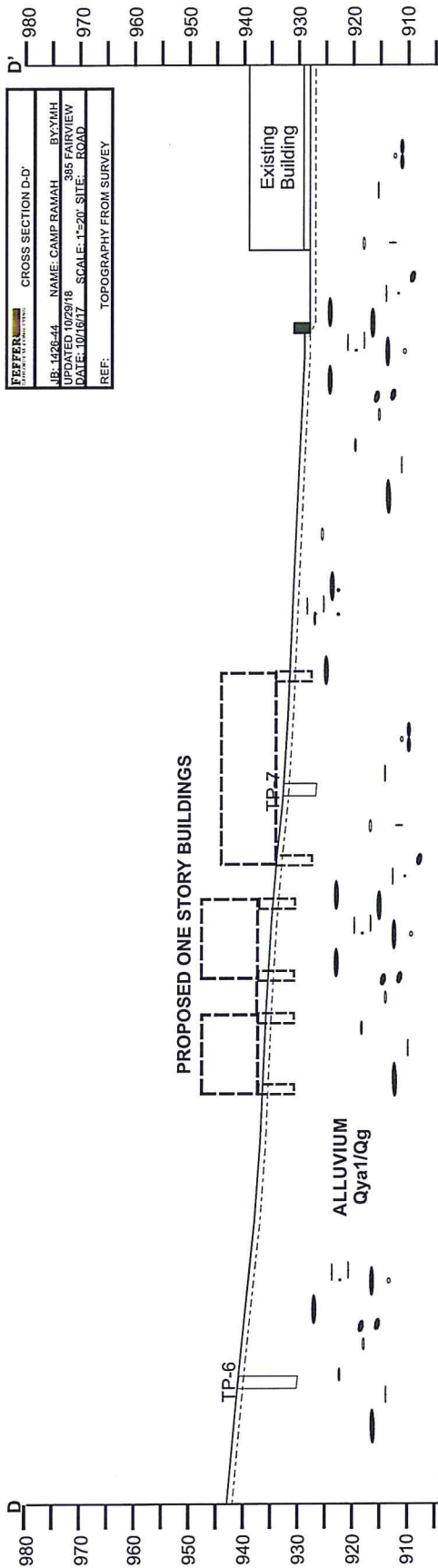
The Ventura County Public Works Agency Advanced Planning Floodplain Section is in the process of reviewing the CUP application. Any correspondence from this County Agency will be provided to you once their review of the project is complete.

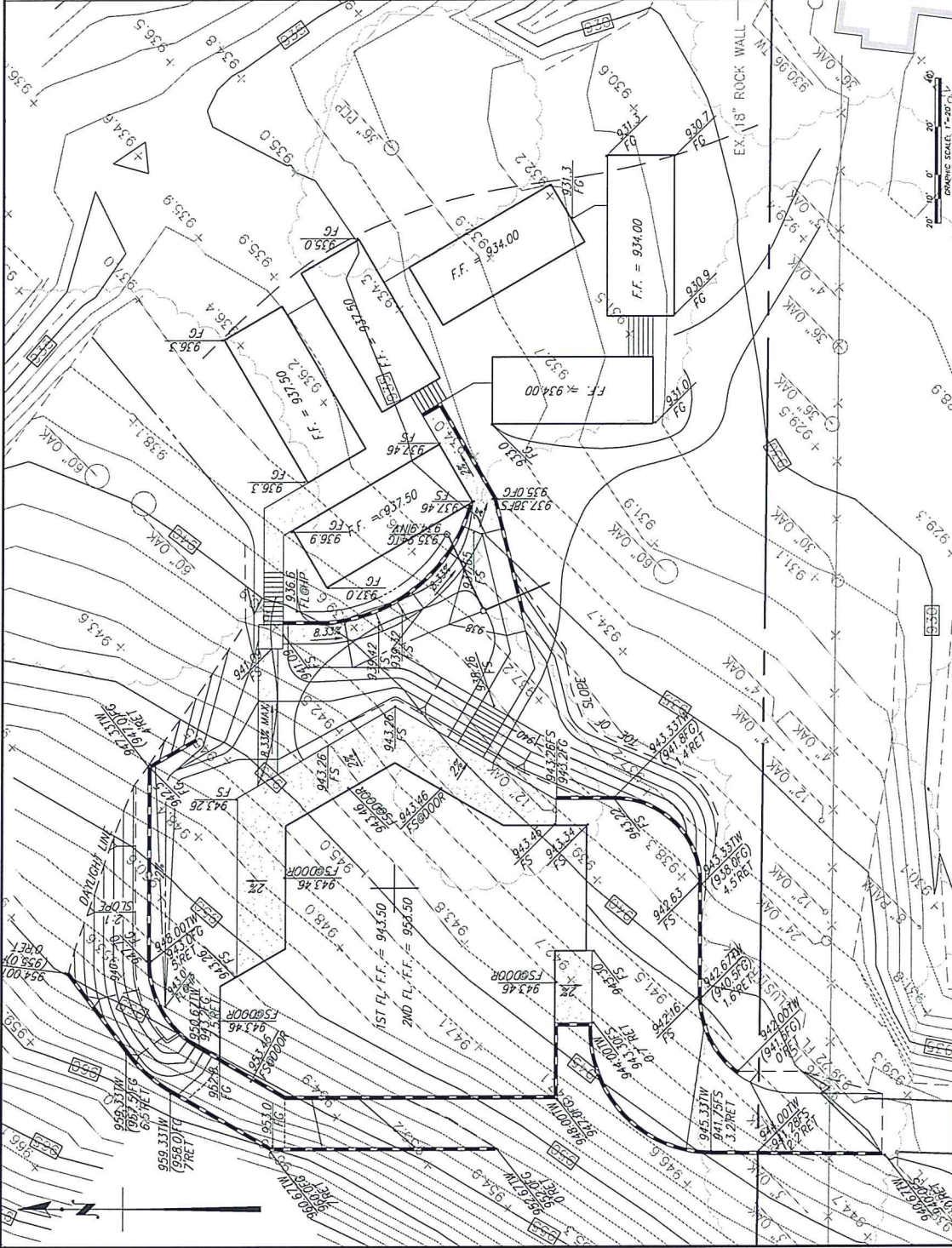
The Ventura County Public Works Agency Transportation Department is in the process of reviewing the CUP application. Any correspondence from this County Department will be provided to you once their review of the project is complete.

When you have gathered all of the information requested above, please submit the information to Kristina Boero, the case planner, to begin the next 30-day review period. Submittal directly to another department or agency may not start the second 30-day review period, resulting in processing delays for your permit application.



PREPARED BY	CROSS SECTION C-C'
J.B. 1428-44	NAME: CAMP RAMAH
UPDATED 10/29/18	BY: YMH
DATE: 10/16/17	385 FAIRVIEW
REF: TOPOGRAPHY FROM SURVEY	SCALE: 1"=20' SITE: ROAD





GENERAL NOTES

1. ALL CONSTRUCTION SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF THE STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (CALIFORNIA) AND THE STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION (FEDERAL).
2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR OBTAINING ALL NECESSARY PERMITS AND APPROVALS FROM THE APPROPRIATE AGENCIES.
3. THE CONTRACTOR SHALL MAINTAIN ACCESS TO ALL ADJACENT PROPERTIES AND SHALL BE RESPONSIBLE FOR REPAIRING ANY DAMAGE CAUSED BY THE CONSTRUCTION.
4. THE CONTRACTOR SHALL MAINTAIN ADEQUATE DRAINAGE DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE DRAINAGE TO ITS ORIGINAL CONDITION.
5. THE CONTRACTOR SHALL MAINTAIN ADEQUATE EROSION CONTROL DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE EROSION CONTROL TO ITS ORIGINAL CONDITION.
6. THE CONTRACTOR SHALL MAINTAIN ADEQUATE SAFETY DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE SAFETY TO ITS ORIGINAL CONDITION.
7. THE CONTRACTOR SHALL MAINTAIN ADEQUATE ACCESS TO ALL ADJACENT PROPERTIES AND SHALL BE RESPONSIBLE FOR RESTORING THE ACCESS TO ITS ORIGINAL CONDITION.
8. THE CONTRACTOR SHALL MAINTAIN ADEQUATE UTILITIES DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE UTILITIES TO ITS ORIGINAL CONDITION.
9. THE CONTRACTOR SHALL MAINTAIN ADEQUATE RECORDS DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE RECORDS TO ITS ORIGINAL CONDITION.
10. THE CONTRACTOR SHALL MAINTAIN ADEQUATE COMMUNICATION DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE COMMUNICATION TO ITS ORIGINAL CONDITION.
11. THE CONTRACTOR SHALL MAINTAIN ADEQUATE QUALITY CONTROL DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE QUALITY CONTROL TO ITS ORIGINAL CONDITION.
12. THE CONTRACTOR SHALL MAINTAIN ADEQUATE SCHEDULING DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE SCHEDULING TO ITS ORIGINAL CONDITION.
13. THE CONTRACTOR SHALL MAINTAIN ADEQUATE BUDGETING DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE BUDGETING TO ITS ORIGINAL CONDITION.
14. THE CONTRACTOR SHALL MAINTAIN ADEQUATE RISK MANAGEMENT DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE RISK MANAGEMENT TO ITS ORIGINAL CONDITION.
15. THE CONTRACTOR SHALL MAINTAIN ADEQUATE COMPLIANCE DURING CONSTRUCTION AND SHALL BE RESPONSIBLE FOR RESTORING THE COMPLIANCE TO ITS ORIGINAL CONDITION.

EARTHWORK QUANTITIES

QUANTITIES: CUT 1193 CY FILL 322 CY

EARTHWORK NOTE

THE EARTHWORK QUANTITIES ARE ESTIMATES ONLY. THEY ARE THE CALCULATED QUANTITIES BASED ON THE DIFFERENCE BETWEEN EXISTING GROUND ELEVATIONS AND PROPOSED GRADE ELEVATIONS AND THEY MAKE NO PROVISION FOR EXISTING OR PROPOSED OBSTRUCTIONS. THE ACTUAL VOLUME OF EARTH MOVED MAY BE DIFFERENT. THE CONTRACTOR SHALL BE RESPONSIBLE FOR VERIFYING THE EARTHWORK QUANTITIES AND FOR OBTAINING NECESSARY PERMITS AND APPROVALS FOR EARTHWORK. NO DIVERSIONS AS TO THE EARTHWORK BALANCE OR ACCURACY IS MADE.

DRAINAGE PLAN CAMP RAINBOW 3805 FAIRVIEW RD. QUAI, CA.	
COUNTY OF VENTURA PUBLIC WORKS	
APPROVE COUNTY OF VENTURA DATE: _____	
BY: MANAGER DEVELOPMENT SERVICES	
LEWIS ENGINEERING 10000 N. HIGHWAY 101 SUITE 100 P.O. BOX 1000 FRESNO, CA 93701 TEL: 432-234-2345	
DATE	DATE
APP.	DATE
SHEET 3 OF 3	
DRAWING NO.	

