PATEL & ASSOCIATES, INC.
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REPORT

PRELIMINARY GEOTECHNICAL INVESTIGATION
FOR
Proposed Commercial Development

LOCATED

**Off Harvard Road and I-15, APN: 0539-111-38,
Newberry Springs Area,
San Bernardino County, California**

**Prepared for;
Mr. Money Samra
10415 Edgebrook Way
Northridge, CA 91326**

June 05, 2017 **Project Number: 18810217**

Mr. Money Samra
10415 Edgebrook Way
Northridge, CA 91326

Attention: Mr. Money Samra

Subject: **P**reliminary **G**eotechnical **I**nvestigation for Proposed Commercial Development, Located Off Harvard Road and I-15, APN: 0539-111-38, Newberry Springs Area, San Bernardino, California.

In accordance with your authorization, **Patel and Associates, Inc**. is pleased to provide our geotechnical services on the subject project. The enclosed report contains the results of our field investigation, laboratory testing and classification, geotechnical considerations, conclusions, and recommendations.

We sincerely appreciate the opportunity of being service to you on this aspect of the project. Please do not hesitate to call us should you have any questions regarding the content of the reports.

Respectfully submitted,

PATEL AND ASSOCIATES, INC.

Stephen M. Poole

REGISTERED PROFESSIONAL ENGINEER
STEPHEN MICHAEL POOLE
No. 692
Exp.
GEOTECHNICAL
STATE OF CALIFORNIA

Stephen M. Poole, PE, GE

Principal Geotechnical Engineer

P&A INC.
COMMITMENT OF EXCELLENCE THROUGH ENGINEERED SOLUTIONS

Contents

1.0 INTRODUCTION AND SCOPE OF WORK

1.1 Introduction

This report presents the results of our **P**reliminary **G**eotechnical **I**nvestigation for the commercial development consisting of a convenience store with gas pumps, located off Harvard Road and I-15, APN: 0539-111-38, Newberry Springs Area, San Bernardino County, California. **Figure 1** shows the location. At present the project site is a undeveloped property. The topography on the site is relatively flat.

1.2 Scope of Work

Our scope of work included:

* Review of available soils data.
* Subsurface investigation by drilling.
* Perform laboratory testing.
* Geotechnical Engineering considerations.
* Report preparation with conclusions and recommendations.

2.0 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 Field Investigation

A field investigation using a truck mounted hollow-stem-auger drill rig was performed on April 28th, 2017. A total of five (5) borings were drilled throughout the site to a maximum depth of 31.5 feet from the existing ground surface. **Plate 1** shows the approximate location of the borings.

The purpose of our investigation was to ascertain the geotechnical properties of sub-surface soils for foundation recommendations, and was not intended to provide evidence of potential environmental conditions. **Appendix A** presents the logs. The logs and related data depict subsurface conditions only at the specific locations and time indicated.

2.2 Laboratory Testing

Laboratory testing on select representative samples included:

* Maximum Density Test (ASTM D 1557)

* Inplace Dry Density Tests
* Sieve Analysis Tests (ASTM D422)
* Expansion Index (ASTM 4829)
* Corrosion Analysis (CTM 643, CTM 417, and CTM 422)

Maximum Density (ASTM D 1557) testing is performed to determine the maximum dry density and optimum moisture content of a soil sample. Engineered fills are compacted to a dense state to improve their engineering properties for foundation soils. The test is utilized to estimate the relative compaction of the inplace soils during the grading operation. One (1) tests was performed on the bulk samples obtained during the field investigation.

Inplace dry density in conjunction with laboratory maximum dry density, provides an indication of relative density (or relative compaction). This inplace relative compaction is utilized in estimation of potential shrinkage factors during grading and recommendations for site preparations. Our test results indicated that relative compaction of the upper 5 feet of earth material was loose to medium dense.

Sieve Analysis is useful in classifying of soils in accordance with the Unified Soil Classification System, ASTM D2487. A total of two (2) Gradation/Sieve Analysis tests were performed. Gradation analysis can be utilized in qualitative determination of other engineering properties.

Expansion Index was evaluated using the guidelines of ASTM D 4829 for the onsite soil expansion potential

Corrosion Analysis was performed based on CTM 643, 417, and 422 in order to evaluate the onsite soil corosion potential.

Results of our laboratory testing are contained in **Appendix B**.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Conditions

The subject site is currently an undeveloped property with flat terrain adjacent to Interstate 15 (I-15).

3.2 Subsurface Conditions

Our field investigation and laboratory testing revealed that the near surface soils consist predominantly of **SILTY SAND (SM) and POORLY-GRADED SAND (SP),** mainly derived by alluvial, gravel, sand, and silt of valleys and floodplains. The soils in the upper 0 to 5 feet were dry to moist, and in a medium dense to dense state. **Appendix A** presents the detailed logs of soils encountered in our excavations.

4.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS

Our geotechnical engineering evaluations are based on the limited field investigation and laboratory testing performed for the subject project.

4.1 Foundation System Considerations

Allowable bearing pressure for foundations depends upon the shear strength, settlement characteristics of the underlying soils, types of foundation system, acceptable differential movement, and depth of embedment. We understand that the structure will have shallow foundations.

4.2 Settlement and Heaving Considerations:

An Expansion Index test was performed which indicated non-expansive soils. Based on dry soils, low field densities and our observations, the upper 0 to 5 feet of soils are likely to settle due to loading and introduction of water. In general, desert soils are cemented and undergo rapid consolidation due to saturation known as hydroconsolidation and settlement of dry sands due to earthquakes. Based on the site conditions, the site is susceptible to settlement in the upper 5 feet. This condition can be mitigated by overexcavation and recompaction of the existing soils.

4.3 Seismic Considerations

Seismic risk along active fault rupture zones represent real potential damage or property loss to structures due to ground shaking or motion. A review of the State California Alquist-Priolo Special Studies Zone maps indicate that the site is not located within any known or published active fault zone. A detailed geological study was not within the scope of this report.

According to section 1613.5 of the 2016 California Building Code (CBC) the site's soil profile may be characterized within the category of Site Class D. The Table below provides the seismic design parameters for this soil profile at the location of Latitude 34.96472222° N, -116.64611111° W, approximately at the center of the lot in accordance with section 1613.5 of the 2016 CBC. Any changes in the present code should be considered during the design.

Categorization/Coefficient	Design Value
Site Class	D
Mapped MCE Spectral Accelatation at Short (0.2 Second) Period S_s	1.148
Mapped MCE Spectral Accelatation at 1-Second Period S_1	0.426
Adjusted Spectural Response Acceleration at 0.2-Second Period, S_{ms}	1.195
Adjusted Spectural Response Acceleration at 1-Second Period, S_{m1}	0.671
Design (5% damped) Spectural Response Acceleration for Short (0.2 Second) Period, S_{ds}	0.797
Design (5% damped) Spectural Response Acceleration for 1.0 Second Period, S_{ds}	0.447

A mean peak ground acceleration was calculated to be 0.475g.

4.4 Seismically Induced Settlement/Liquefaction Potential

Ground movement and settlement can occur when relatively low density soils are subject to ground vibrations. These loose soils will be replaced with compacted fill.

5.0 CONCLUSIONS

In our opinion and based upon our geotechnical investigation findings of the soils encountered on this project site are suitable for the proposed development, provided recommendations contained in this report are followed during design and construction of the project.

5.1 Upper 3 to 5 feet of the soils are loose to medium dense and dry and prone to settlement. These materials are subject to settlement due to consolidation and ground vibrations. Overexcavation and recompaction of near surface soils, and other mitigating measures are discussed in **Section 6.0** of this report.

5.2 Our investigation and testing indicate that the near surface soils have a very low expansion potential.

5.3 Moderate to severe ground shaking should be expected during large magnitude earthquakes which can cause settlement.

5.4 Seismic considerations contained in **Section 4.3** should be considered during planning and design in conjunction with requirements of latest California Building Code.

6.0 RECOMMENDATIONS

6.1 Site Preparation

To achieve uniform support for shallow foundations and slab-on-grade, the site should be cleared of all vegetation, debris and any deleterious materials that fall within the grading limits prior to construction. Any existing utility lines, buried abandoned utilities or objects should be removed, capped and/or rerouted if they interfere with the proposed structure. The cavities resulting from removal of utility lines and any buried obstructions should be properly backfilled and compacted as recommended in **Section 6.5** of this report.

6.2 Shrinkage and Compaction Settlement During Grading

Our field investigation and field and laboratory testing determined that the near surface soils are loose to medium dense. Accordingly, we estimate the shrinkage factor to be approximately **12** to **17** percent during overexcavation and recompaction. Shrinkage is defined as the decrease in volume of soil due to artificial compaction, expressed as percentage of ratio of compacted dry density minus inplace density to compacted dry density. Shrinkage factors provided herein assumes an average relative compaction of **92** percent.

6.3 Overexcavation and Recompaction

If shallow foundations are used, the upper 3 to 5 feet of soil is relatively loose to medium dense and dry. To mitigate rapid settlement, we recommend that the building pad area, extending 5 feet beyond the outer most limits be overexcavated. To provide uniform consistent soil support and drainage, we recommend that the upper 3 to 5 feet below the existing or lowest cut grade be overexcavated. The competent bottom subgrade soils should be scarified to a minimum depth of 6 inches and uniformly moisture conditioned. The scarified surface should be observed by the geotechnical consultant prior to compaction.

Upon the approval of the geotechnical consultant, the bottoms should then be compacted per compaction criteria provided in **Secton 6.5** of this report. The excavated site is then backfilled and compacted in loose lifts not to exceed 8 inches of fill. Fill soils are to be uniformly moisture conditioned and compacted as per compaction criteria provided herein.

During the grading of subgrade soils, if loose, yeilding or otherwise unsuitable soils are exposed, further overexcavation should be made up to the competent soils.

6.4 Imported Fill

Imported fill should be free of all deleterious substances, and non-expansive. The source of the imported fill should verified by the Engineer prior to being brought to the site.

6.5 Compaction Criteria

Following compaction criteria should be observed:

Competent Sturtural Bottoms	90% or greater @ 0 to +2% of OMC
Structural Fill-Building areas extending at least 5' beyond the outermost building limit	90% or greater @ 0 to +2% of OMC
Backfill around retaining walls, Trench Backfill from 1' to 4' below the subgrade	90% or greater @ 0 to +2% of OMC

All compaction and moisture content criteria are relative to ASTM D1557 Maximum Dry Density (MDD) and Optimum Moisture Content (OMC).

6.6 Foundation Design - Shallow

Shallow footings are proposed provided proper bearing on dense soil per report guildlines are followed. All footings should be founded a minimum depth of 18 inches below existing grade. The bearing pressure can be increased by one-third for seismic or wind loading. The Structural Engineer should design the foundation system following the minimum requirements recommended herein.

The minimum depth of footings below the ground surface shall be no less than 18 inches. A minimum of two No. 4 rebar shall be place at the bottom and top of footings longitudinally with a minimum of 3 inches of cover.

An allowable vertical bearing capacity of 2,500 pounds-per-square-foot (psf) may be used for footing design incorperating the recommendations contain herein. The lateral bearing presure of 250 pcf/f below grade with a maximum allowable friction resistance of 0.36 may be used for design of concrete structures placed on properly dense soils.

6.7 Settlement

Based on the settlement characteristics of the earth materials that underlie the building site and the anticipated loading, we estimate that the maximum total static settlement of the footings will be less than one inch.. Differential static settlement is expected to be about ½ inch over a horizontal distance of approximately 20 feet, for an angular distortion ratio of 1:480. It is anticipated that the majority of the settlement will occur during construction or shortly after the initial application of loading.

The above settlement estimates are based on the assumption that the grading and construction are performed in accordance with the recommendations presented in this report and that the project geotechnical consultant will observe or test the earth material conditions in the footing excavations.

6.8 Lateral Resistance

Passive earth pressure of 250 psf per foot of depth to a maximum value of 2,500 psf may be used to establish lateral bearing resistance for footings. For areas coved with hardscape, passive earth pressure may be taken from the surface. For areas without hardscape, the first 3 feet of the soil profile must be neglected when calculating passive earth pressure. A coefficient of friction of 0.36 times the dead load forces may be used between concrete and the supporting earth materials to determine lateral sliding resistance. The above values may be increased by one-third when designing for short duration wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. In no case shall the lateral sliding resistance exceed one-half the dead load for clay, sandy clay, sandy silty clay, silty clay, and clayey silt.

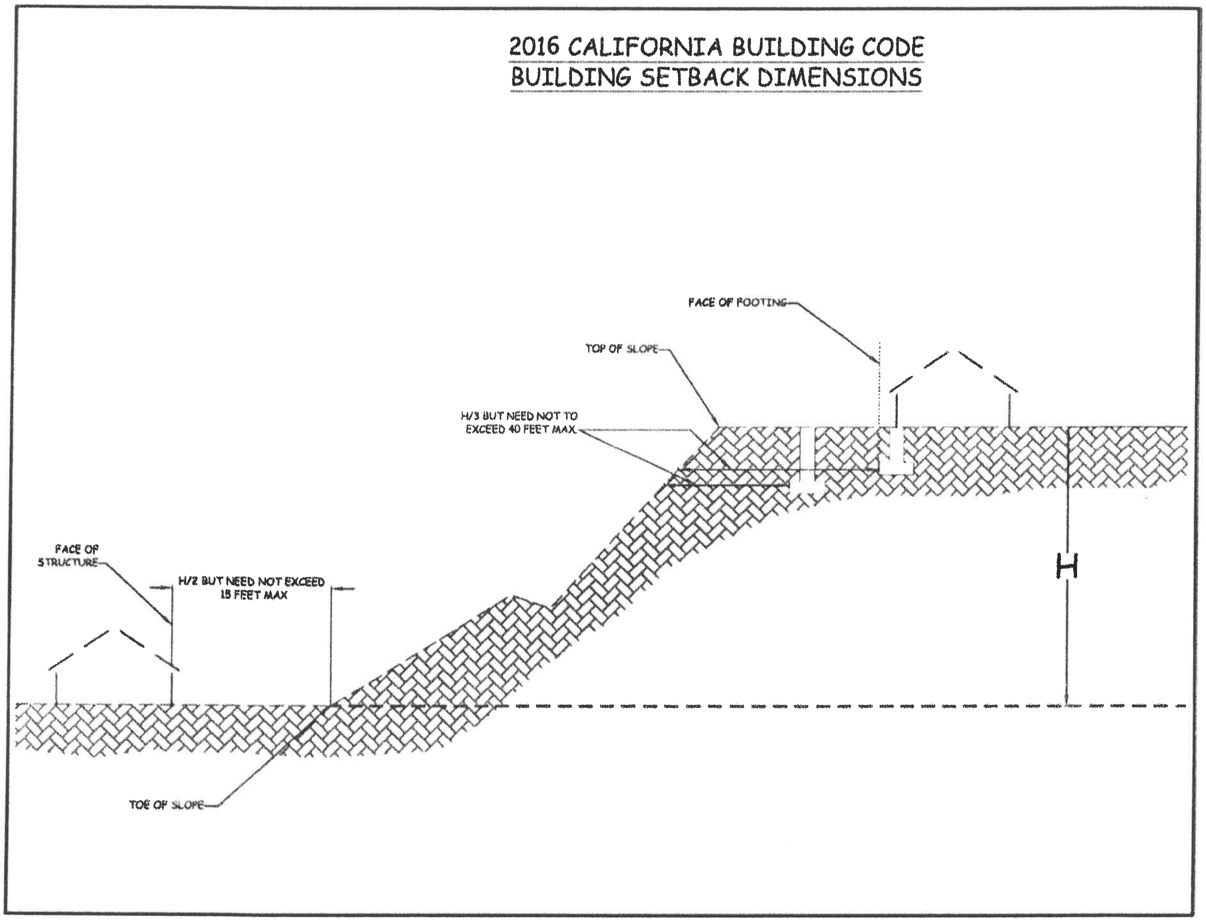
The above lateral resistance values are based on footings for an entire structure being placed directly against either compacted fill or competent alluvium.

6.9 Structural Setbacks and Building Clearance

Structural setbacks are required per the 2016 California Building Code (CBC). Additional structural setbacks are not required due to geologic or geotechnical conditions within the site. Improvements constructed in close proximity to natural or properly engineered and compacted slopes can, over time, be affected by natural processes including gravity forces, weathering, and long term secondary settlement. As a result, the CBC requires that buildings and structures be setback or footings deepened to resist the influence of these processes.

For structures that are planned near ascending and descending slopes, the footings should be embedded to satisfy the requirements presented in the CBC, Section 1808.7 as illustrated in the following Foundation Clearances from Slopes diagram.

FOUNDATION CLEARANCES FROM SLOPES



When determining the required clearance from ascending slopes with a retaining wall at the toe, the height of the slope shall be measured from the top of the wall to the top of the slope.

6.10 Observations Foundation

In accordance with the 2016 CBC and prior to the placement of forms, concrete, or steel, all foundation excavations should be observed by the geologist, engineer, or his representative to verify that they have been excavated into competent bearing materials. The excavations should be per the approved plans, moistened, cleaned of all loose materials, trimmed neat, level, and square. Any moisture softened earth materials should be removed prior to steel or concrete placement.

Earth materials from foundation excavations should not be placed in slab on grade areas unless the materials are tested for expansion potential and compacted to a minimum of 90 percent of the maximum dry density.

6.11 Slab on Grade

Slab on grade inside the proposed structure should be placed on properly compacted soil. As a minimum, we suggest that the thickness of concrete floor slabs supported directly on the ground shall not be less than 5 inches per CBC Section 1910A. The Sturctural Engineer should design the actual slab thickness and reinforcement based on structural load requirements.

A moisture retarder with joints overlaped not less than 6 inches of 10 mil Vapor Barrier or other approved equivalent membrane shall be placed on the subgrade soils.

Reinforcement for shrinkage and temperature stresses shall comply with provisions in CBC Chapeter 19A and ACI 318. We recommend of minimum of No. 4 bars, placed 12 inches on center both ways at mid-height of the slab. We recommend construction joints every 200 square feet.

6.12 Cement Type and Corrosion Potential

Based on our experience, we recommend Type II cement for all concrete works in contact with soils.

Corrosion is defined by the National Association of Corrosion Engineers (NACE) as "a deterioration of a substance or its properties because of a reaction with its environment." From a geotechnical viewpoint, the "substances" are the reinforced concrete foundations or buried metallic elements (not surrounded by concrete) and the "environment" is the prevailing earth materials in contact with them. Many factors can contribute to corrosivity, including the presence of chlorides, sulfates, salts, organic materials, different oxygen levels, poor drainage, different soil types, and moisture content. It is not considered practical or realistic to test for all of the factors which may contribute to corrosivity.

The potential for concrete exposure to chlorides is based upon the recognized Caltrans reference standard "Bridge Design Specifications", under Subsection 8.22.1 of that document, Caltrans has determined that "Corrosive water or soil contains more than 500 parts per million (ppm) of chlorides". Based on limited preliminary laboratory testing, the onsite earth materials have chloride contents *less* than 500 ppm. As such, specific requirements resulting from elevated chloride contents are not required.

Specific guidelines for concrete mix design are provided in 2016 CBC Section 1904.1 and ACI 318, Section 4.3 Table 4.3.1 when the soluble sulfate content of earth materials exceeds 0.1 percent by weight. Based on limited preliminary laboratory testing, the onsite earth materials are classified in accordance with Table 4.3.1 as having a *negligible* sulfate exposure condition.

Based on our laboratory testing of resistivity, the onsite earth materials in contact with buried steel should be considered *mildly corrosive*. Additionally, pH values below 9.7 are recognized as being corrosive to most common metallic components including, copper, steel, iron, and aluminum. The pH values for the earth materials tested were *lower* than

9.7. Therefore, any steel or metallic materials that are exposed to the earth materials should be encased in concrete or other measures should be taken to provide corrosion protection.

The preliminary test results for corrosivity are based on limited samples, and the initiation of grading may blend various earth materials together. This blending or imported material could alter and increase the detrimental properties of the onsite earth materials. Accordingly, additional testing for chlorides and sulfates along with testing for pH and resistivity should be performed upon completion of grading. Laboratory test results are presented in Appendix C.

6.13 Trench Backfill

Utility trench backfill material should be non-expansive, free of debris and any deleterious substances. Local onsite material is suitable for trench backfill. Granular bedding of 1 foot underneath the water and sewer line pipes and 6 inches above the pipes are recomended. The backfill should be compacted in loose lifts not exceeding six 6 inches to achieve relative compaction as set in **Section 6.4**

6.14 Surface Drainage and Landscaping

All grading should be such to direct surface runoff away from the building foundations. Roof runoff should also be directed away from the foundations. To mitigate settlement and potential swelling/collapse of near surface soils which could lead to distress to a structure, we recommend desert landscaping.

6.15 Active and At-Rest Earth Pressures

Foundations may be designed in accordance with the recommendations provided in the Tentative Foundation Design Recommendation section of this report. The following table provides the minimum recommended equivalent fluid pressures for design of retaining walls a maximum of 8 feet high. The active earth pressure should be used for design of unrestrained retaining walls, which are free to tilt slightly. The at-rest earth pressure should be used for design of retaining walls that are restrained at the top, such as basement walls, curved walls with no joints, or walls restrained at corners. For curved walls, active pressure may be used if tilting is acceptable and construction joints are provided at each angle point and at a minimum of 15 foot intervals along the curved segments. For earthquake loading, a load of 40 pcf using an inverted triangle.

MINIMUM STATIC EQUIVALENT FLUID PRESSURES (pcf)		
PRESSURE TYPE	BACKSLOPE CONDITION	
	LEVEL	2:1 (h:v)
Active Earth Pressure	40	63
At-Rest Earth Pressure	60	95

The retaining wall parameters provided do not account for hydrostatic pressure behind the retaining walls. Therefore, the subdrain system is a very important part of the design. All retaining walls should be designed to resist surcharge loads imposed by other nearby walls, structures, or vehicles should be added to the above earth pressures, if the additional loads are being applied within a 1:1 (h:v) plane projected up from the heel of the retaining wall footing. As a way of minimizing surcharge loads and the settlement potential of nearby buildings, the footings for the building can be deepened below the 1.5:1 (h:v)plane projected up from the heel of the retaining wall footing.

Upon request and under a separate scope of work, more detailed analyses can be performed to address equivalent fluid pressures with regard to stepped retaining walls, actual retaining wall heights, actual backfill inclinations, specific backfill materials, higher retaining walls requiring earthquake design motions, etc.

6.16 Preliminary Asphaltic Concrete Pavement Design

Laboratory testing of representative earth materials indicate an R-value of 24 may be used for preliminary pavement design. The following table includes our minimum recommended asphaltic concrete pavement sections calculatesd in accordance with the State of California design procedures using assumed Traffic Indices. Final pavement sections and calculation sheets have been provided within the appendices of this report. An overexcavation of three (3) feet is required.

PRELIMINARY ASPHALTIC CONCRETE PAVEMENT DESIGN				
PARAMETERS	AUTO PARKING	AUTO DRIVES	ENTRANCES/ TRUCK DRIVES	TRUCK COLLECTOR
Assumed Traffic Index	5.0	6.0	7.0	9.0
Design R-Value	24	24	24	24
AC Thickness (inches)	3	4	4	6
AB Thickness (inches)	7	8	11	14

6.17 Construction Observations and Testing

The recommendations contained in this report are based on the results of our limited preliminary investigation and our general experience with similar soil conditions. All grading and excavation should be performed under the observation and testing of the geotechnical consultant. The following stages of the construction activities include but may not be limited for observation are:

- Overexcavation and scarification.
- Fill placement and backfilling.
- During preparation of the building pad and the footing bottoms.
- Subgrade preparation
- Placement of aggregate base layer and asphalt concrete.
- Trench and Utility backfill

- When any unusual conditions are encountered.

Based on these observations and testing, it may be necessary to modify the recommendations contained herein.

6.18 Final Report

A final report should be prepared which will contain field observations, test results and additional recommendations, as warranted.

7.0 LIMITATIONS

Conclusions, recommendations and professional opinions resulting from our site observations, field investigation and laboratory testing are intended solely for **Mr. Money Samra.** Our conclusions and recommendations are based on our understanding of the project and consistent with the level of skill ordinarily exercised by other professional consultants under similar circumstances at the same time our services were provided. This report is exclusively prepared to assist **Mr. Money Samra** and their Engineer in the design of the footings and foundation support for the proposed residential development on site. **Patel & Associates, Inc**. should be consulted to provide written modification to the recommendations contained in this report, depending upon the project requirements.

APPENDIX A

Boring Logs

PATEL AND ASSOCIATES, INC.

BORING LOG NO. B-1

Project: APN 0539-111-38 **Project# :18810217**

Client: Money Samra **Date:** 4/28/2017

Depth Feet	Sample Type	Moisture Content %	Dry Density pcf	Lab Test Type	Blow Count per ft	Soil Profile	Geotechnical Description
--							
--							
2.5--		1.5%	116.6		30	**SM**	Silty Sand, Brown, Dry, Loose, fine to coarse sand.
--							
--							
5'--		0.5%			40	SP-SM	Poorly-graded silty SAND; brown, dry, dense, fine to coarse sand
--							
7.5'--					77	SM	Silty SAND; brown, dry, very dense, fine to coarse sand.
--							
10'--					50	SP	Poorly-graded SAND; brown, dry, dense, fine to coarse sand
--							
--							
--							
15'--					50/6	SM	Silty Sand; brown, dry, very dense, fine to coarse sand
--							
--							
--							
20'--		**2.8%**	**105.2**		50/5	SM	
--							
--							
--							
25'--		**0.6%**			75	SP	Poorly-graded SAND; brown, dry, dense, fine to coarse sand
--							
--							
--							
30'--							
--							
--							
--							
35'--							
--							
--							
--							
40'--							
--							
--							
--							
45'--							
--							
--							
--							
50'--							
--							**Bottom of Exploratory Boring: 31.5ft**
--							**No Groundwater Present**
--							

Figure A-1

PATEL AND ASSOCIATES, INC.

BORING LOG NO. B-2

Project: APN 0539-111-38 **Project #:**18810217

Client: Money Samra **Date:** 4/28/2017

Depth Feet	Sample Type	Moisture Content %	Dry Density pcf	Lab Test Type	Blow Count per ft	Soil Profile	Geotechnical Description
--							Topsoil:
--						SM	Silty SAND, Brown, Dry, Loose, fine to coarse sand
2.5'--		0.7			20	SP	Quaternary Alluvial (QA):
--							Poorly- graded SAND; Brown, dry, medium dense, fine to coarse sand
--							Sample Disturbed, No recovery @ 2.5 feet.
5'--		1.2	106.4		42	SP-SM	Poorly-graded silty SAND; brown, dry, dense, fine to coarse sand
--							
7.5'--		5	114.3		68	SM	Sily SAND; brown, dry, very dense, fine to coarse sand
--							
10'--		0.5			59	SP	poorly-graded SAND, brown, dry, medium dense, fine to coarse sand
--						SP-SM	Poorly-graded silty SAND, brown, dry, medium dense, fine to coarse sand
--							Sample Disturbed, No Recovery @ 10ft
--							
15'--		5.1	112.8		90/11	SM	Silty SAND, brown, dry, very dense, fine to coarse sand
--							
--							
--							
20'--			121		90/10		
--							
--							
--							
25'--		0.7			75	SP	
--							
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30'--							
--							
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35'--							
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40'--							
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--							
45'--							
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--							
--							
50'--							
--							**Bottom of Exploratory Boring: 26.5 ft**
--							**No Groundwater Present**
--							

Figure A-2

PATEL AND ASSOCIATES, INC

BORING LOG NO. B-3

Project: APN 0539-111-38 **Project #:**18810217

Client: Money Samra **Date:** 4/28/2017

Depth Feet	Sample Type	Moisture Content %	Dry Density pcf	Lab Test Type	Blow Count per ft	Soil Profile	Geotechnical Description
--							**Topsoil:**
2.5'--		1.2%	123.6		76	SM	Silty SAND, brown, dry, loose, fine to coarse sand
--							**Quaternary Alluvial(Qa)**
--						SM	Poorly-graded SAND; brown, dry, medium dense, fine to coarse sand
--							
5'--		1.0%	104.6		45	SP-SM	
--							
7.5'--		4.7%	117.1		90/11	SM	Silty SAND; brown, dry, very dense, fine to coarse sand
--							
10'--		0.7%	112		70		Poorly-graded SAND, brown, dry, very dense, fine to coarse sand
--							
--							
--							
15'--		0.9%	109.4		50/6	SM	Silty SAND, brown, dry, very dense, fine to coarse sand
--							
--							
--							
20'--		3.7%	113.4		50/5	SM	Silty SAND, brown, dry, very dense, fine to coarse sand
--							
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--							
25'--							
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30'--							
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35'--							
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40'--							
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45'--							
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--							
50'--							
--							**Bottom of Exploratory Boring: 26.5ft**
--							**No Groundwater Present**
--							

Figure A-3

PATEL AND ASSOCIATES, INC.

BORING LOG NO. B-4

Project: APN 0539-111-38 **Project #:**18810217

Client: Money Samra **Date:** 4/28/2017

Depth Feet	Sample Type	Moisture Content %	Dry Density pcf	Lab Test Type	Blow Count per ft	Soil Profile	Geotechnical Description
--							**Topsoil:**
--						SM	Silty SAND, brown, dry, loose, fine to coarse sand
2.5'--		3.3%	114.3		79		**Quaternary Alluvial(Qa)**
--						SM	Poorly-graded SAND; brown, dry, medium dense, fine to coarse sand
--							
5'--		0.8%	111.6		48	SP-SM	Poorly-graded silty SAND; brown, dry, dense, fine to coarse sand
--							
7.5'--		3.8%	111.9		46		
--							
10'--		0.8%			30	SP	Poorly-graded SAND, brown, dry, medium dense, fine to coarse sand
--							No Sample Recovery @ 10 ft
--							
--							
15'--		0.9%	116.4		50		Silty SAND, brown, dry, very dense, fine to coarse sand
--							
--							
--							
20'--		3.1%	106		42	SM	Poorly- graded silty SAND, brown, dry, medium dense, fine to coarse sand
--							
--							
--							
25'--		2.2%	113.4		50/6		
--							
--							
--							
30'--							
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35'--							
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--							
40'--							
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--							
--							
45'--							
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--							
--							
50'--							
--							**Bottom of Exploratory Boring: 26.5ft**
--							**No Groundwater Present**
--							

Figure A-4

PATEL AND ASSOCIATES, INC.

BORING LOG NO. B-5

Project: APN 0539-111-38 **Project #:**18810217

Client: Money Samra **Date:** 4/28/2017

Depth Feet	Sample Type	Moisture Content %	Dry Density pcf	Lab Test Type	Blow Count per ft	Soil Profile	Geotechnical Description
--							**Topsoil:**
--						SM	Silty SAND, brown, dry, loose, fine to coarse sand
2.5'--		2.5	113.9		43		**Quaternary Alluvial(Qa)**
--						SM	Poorly-graded SAND; brown, dry, medium dense, fine to coarse sand
--							
5'--		5	114.7		39	SP-SM	Poorly-graded silty SAND; brown, dry, dense, fine to coarse sand
--							
7.5'--		2.6	115.9		79	SM	Silty SAND; brown, dry, very dense, fine to coarse sand
--							
10'--					50/6	SP	Poorly-graded SAND, brown, dry, very dense, fine to coarse sand
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15'--							
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20'--							
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25'--							
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35'--							
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40'--							
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45'--							
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50'--							
--							**Bottom of Exploratory Boring:12.5 ft**
--							**No Groundwater Present**
--							

Figure A-5

APPENDIX B

LABORATORY PROCEDURES AND TEST RESULTS

APPENDIX B

Laboratory Procedures and Test Results

Laboratory testing provided quantitative and qualitative data involving the relevant engineering properties of the representative earth materials selected for testing. The representative samples were tested in general accordance with American Society for Testing and Materials (ASTM) procedures and/or California Test Methods (CTM).

Soil Classification: Earth materials encountered during exploration were classified and logged in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) of ASTM D 2488. Upon completion of laboratory testing, exploratory logs and sample descriptions were reconciled to reflect laboratory test results with regard to ASTM D 2487.

Moisture and Density Tests: For select samples moisture content was determined using the guidelines of ASTM D 2216 and dry density determinations were made using the guidelines of ASTM D 2937. These tests were performed on relatively undisturbed samples and the test results are presented on the exploratory logs.

Expansion Index: The expansion potential of representative samples was evaluated using the guidelines of ASTM D 4829. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
B-1 @ 0-2 feet	Silty SAND	8	Very Low

Minimum Resistivity and pH Tests: Minimum resistivity and pH Tests of select samples were performed using the guidelines of CTM 643. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	pH	MINIMUM RESISTIVITY (ohm-cm)
B-1 @ 0-2 feet	Silty SAND	9.2	3,400

Soluble Sulfate: The soluble sulfate content of select samples was determined using the guidelines of CTM 417. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	SULFATE CONTENT (% by weight)	SULFATE EXPOSURE
B-1 @ 0-2 feet	Silty SAND	No Detection	Negligable

Chloride Content: Chloride content of select samples was determined using the guidelines of CTM 422. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	CHLORIDE CONTENT (ppm)
B-1 @ 0-2 feet	Silty SAND	60

Maximum Density: The maximum dry density and optimum moisture content of representative samples were determined using the guidelines of ASTM D 1557. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
B-1 @ 0-2 feet	Silty SAND	119.0	8.5

R- Value: The R- value of a representative samples was determined using guidelines of CTM 301. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	R - VALUE
B-1 @ 0-2 feet	Silty SAND	24

APPENDIX C

CALIFORNIA DEPARTMENT OF
TRANSPORTATION

Caltrans ARS Online (v2.3.09)

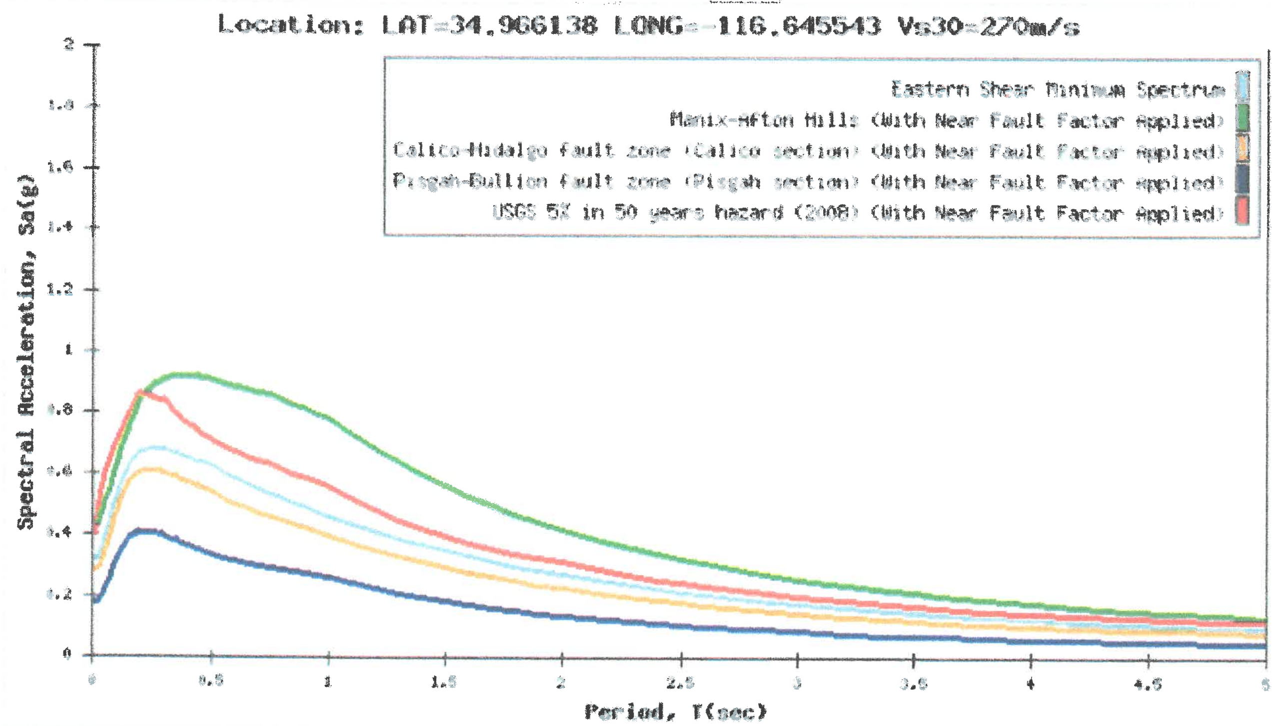
This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in *Appendix B of Caltrans Seismic Design Criteria*. More...

SELECT SITE LOCATION

Latitude: 34.966138 **Longitude:** -116.645543 ***Vs30*:** 270 m/s Calculate

CALCULATED SPECTRA

Display Curves: 3 ▼



Tabular Data | Envelope Only | Hide Near Fault | Axis Scale | Show Basin

Apply Near Fault Adjustment To:

NOTE: Caltrans SDC requires application of a Near Fault Adjustment factor for sites less than 25 km (Rrup) from the causative fault.

☑ Deterministic Spectrum Using

0.36 Km Manix-Afton Hills

12.11 Km Calico-Hidalgo fault zone (Calico section)

22.00 Km Pisgah-Bullion fault zone (Pisgah section)

☑ Probabilistic Spectrum Using

0.36 Km (Recommend Performing Deaggregation To Verify)

◉ Show Spectrum with Adjustment Only

○ Show Spectrum with and without near fault Adjustment

OK

2008 National Seismic Hazard Maps - Source Parameters

New Search

Distance in Kilometers	Name	State	Pref Slip Rate (mm/yr)	Dip (degrees)	Dip Dir	Slip Sense	Rupture Top (km)	Rupture Bottom (km)	Length (km)
12.07	Calico-Hidalgo	CA	1.8	90	V	strike slip	0	14	117
21.97	Pisgah-Bullion Mtn-Mesquite Lk	CA	0.8	90	V	strike slip	0	13	88
26.49	Landers	CA	0.6	90	V	strike slip	0	15	95
27.09	Gravel Hills-Harper Lk	CA	0.7	90	V	strike slip	0	11	65
35.13	Lenwood-Lockhart-Old Woman Springs	CA	0.9	90	V	strike slip	0	13	145
36.21	Blackwater	CA	0.5	90	V	strike slip	0	12	60
45.75	Johnson Valley (No)	CA	0.6	90	V	strike slip	0	16	35
49.38	So Emerson-Copper Mtn	CA	0.6	90	V	strike slip	0	14	54
58.29	Helendale-So Lockhart	CA	0.6	90	V	strike slip	0	13	114
68.68	North Frontal (West)	CA	1	49	S	reverse	0	16	50
69.09	North Frontal (East)	CA	0.5	41	S	thrust	0	16	27
69.69	Garlock;GE+GC+GW	CA	n/a	90	V	strike slip	0.3	12	256
69.69	Garlock;GE+GC	CA	n/a	90	V	strike slip	0	12	156

69.69	Garlock;GE	CA	3	90	V	strike slip	0	12	45
73.30	Garlock;GC+GW	CA	n/a	90	V	strike slip	0.4	12	210
73.30	Garlock;GC	CA	7	90	V	strike slip	0	12	111
73.95	Death Valley Connected	CA	4.6	82		strike slip	0	14	306
73.95	Death Valley (So)	CA	4	90	V	strike slip	0	13	42
74.12	Owl Lake	CA	2	90	V	strike slip	0	12	25
75.48	Hunter Mountain Connected	CA	2.5	90	V	strike slip	0	13	186
75.48	Panamint Valley	CA	2.5	90	V	strike slip	0	13	110
91.50	Cleghorn	CA	3	90	V	strike slip	0	16	25
93.14	Tank Canyon	CA	1	50	W	normal	0	8	16
94.92	Pinto Mtn	CA	2.5	90	V	strike slip	0	16	74
96.04	Burnt Mtn	CA	0.6	67	W	strike slip	0	16	21
96.36	Eureka Peak	CA	0.6	90	V	strike slip	0	15	19

Search Results

4 of 4 earthquakes in map area.

7.1	**16km SW of Ludlow, CA** 1999-10-16 09:46:44 (UTC)	13.7 km
6.3	**7km SSE of Big Bear City, CA** 1992-06-28 15:05:30 (UTC)	3.6 km
7.3	**10km N of Yucca Valley, CA** 1992-06-28 11:57:34 (UTC)	-0.1 km
6.5	**34km SSE of Fort Irwin, CA** 1947-04-10 15:58:05 (UTC)	6.0 km

Didn't find what you were looking for?

- Check your Settings.
- Which earthquakes are included on the map and list?
- Felt something not shown – report it here.

Earthquakes updated

USGS Design Maps Summary Report

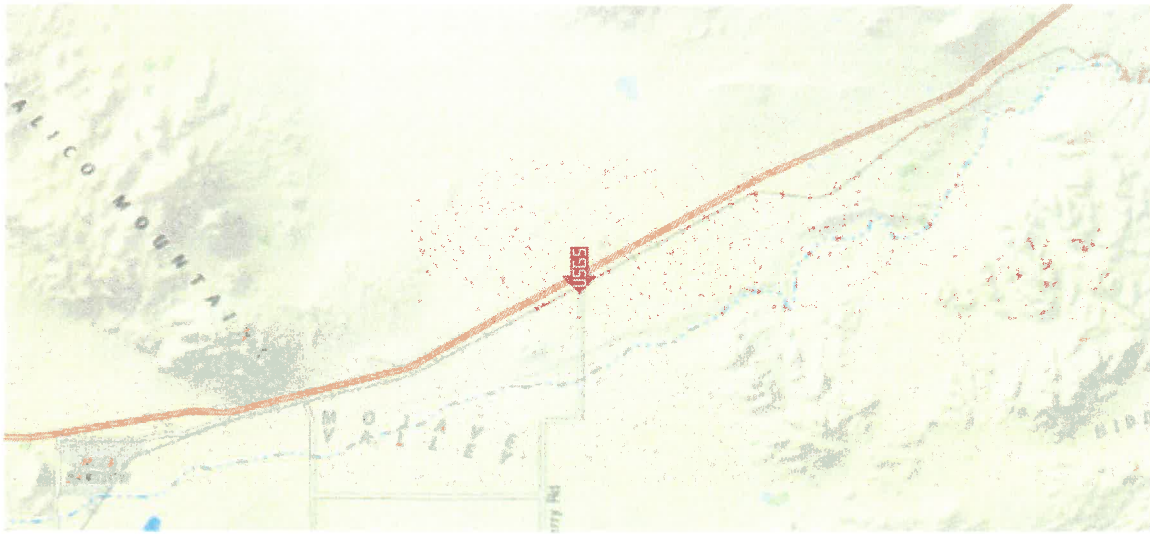
User–Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.96614°N, 116.64554°W

Site Soil Classification Site Class D – "Stiff Soil"

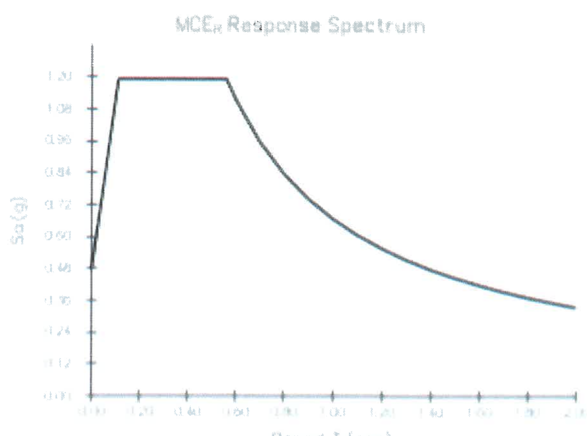
Risk Category I/II/III

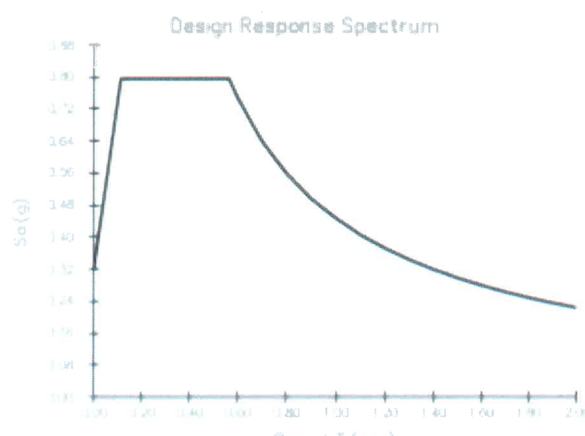


USGS–Provided Output

S_S =	1.146 g	S_{MS} =	1.194 g	S_{DS} =	0.796 g
S_1 =	0.425 g	S_{M1} =	0.670 g	S_{D1} =	0.447 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.





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 (34.96614°N, 116.64554°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] $S_S = 1.146$ g

From Figure 22-2 [2] $S_1 = 0.425$ 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				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
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	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 = 1.146 g, F_a = 1.042

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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	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 = 0.425 g, F_v = 1.575

Equation (11.4–1): $S_{MS} = F_a S_S = 1.042 \times 1.146 = 1.194$ g

Equation (11.4–2): $S_{M1} = F_v S_1 = 1.575 \times 0.425 = 0.670$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

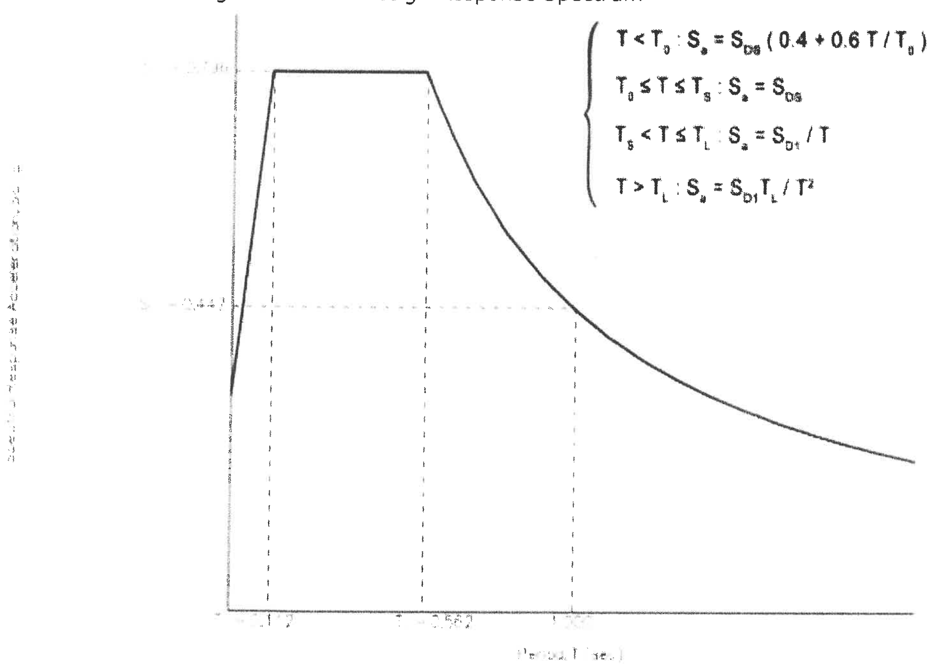
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.194 = 0.796$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.670 = 0.447$ g

Section 11.4.5 — Design Response Spectrum

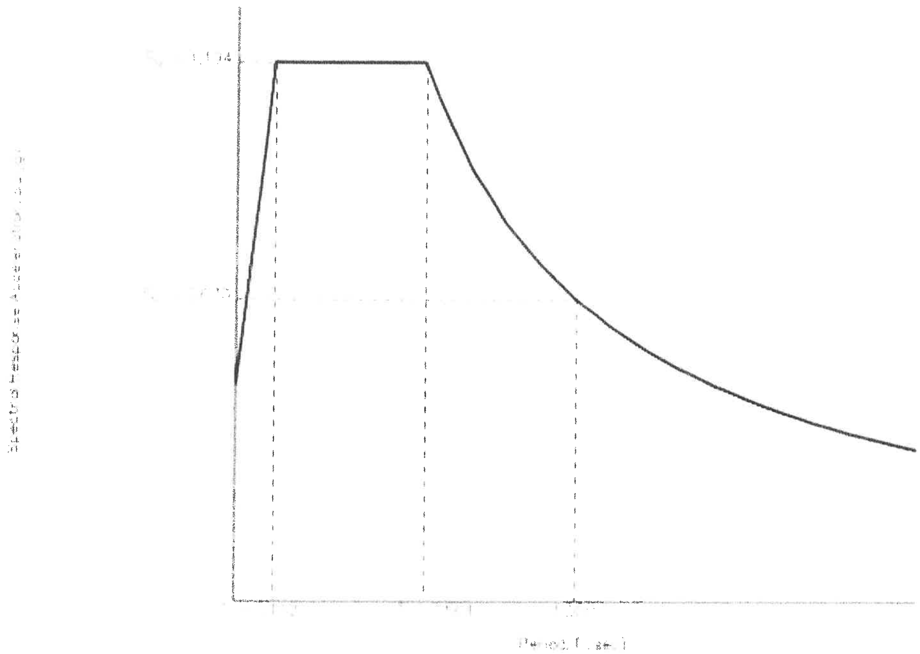
From Figure 22-12 [3] $T_L = 8$ seconds

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 = 0.453

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.047 \times 0.453 = 0.474$ 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	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 = 0.453 g, F_{PGA} = 1.047

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] $C_{R1} = 1.005$

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} = 0.796 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} = 0.447 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

PAVING DESIGN

JN: 18840217 CONSULT: SMP
PROJECT: Newberry Springs

CALCULATION SHEET # AutoParking

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	24	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	24	
Input Traffic Index (TI)	5	
Calculated Total Gravel Equivalent (GE)	1.216	feet
Calculated Total Gravel Equivalent (GE)	14.592	inches
Calculated Gravel Factor (Gf) for A/C paving	2.53	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent			INCHES		FEET	
GE (feet)	GE (inches)	Delta (inches)	A/C Section Thickness (inches)	Minimum Base (inches)	A/C Section Thickness (feet)	Minimum Base (feet)
0.63	7.60	6.99	3.0	6.6	0.25	0.55
0.74	8.87	5.72	3.5	5.4	0.29	0.45
0.76	9.13	5.47	3.6	4.8	0.30	0.40
0.84	10.14	4.45	4.0	4.2	0.33	0.35
0.89	10.65	3.95	4.2	3.6	0.35	0.30
0.95	11.41	3.19	4.5	3.0	0.38	0.25
1.01	12.17	2.43	4.8	2.4	0.40	0.20
1.06	12.67	1.92	5.0	1.8	0.42	0.15
1.27	15.21	-0.62	6.0		0.50	
2.11	25.35	-10.76	10.0		0.83	
2.53	30.42	-15.83	12.0		1.00	

PAVING DESIGN

JN: 18840217 CONSULT: SMP
PROJECT: Newberry Springs

CALCULATION SHEET # AutoDRIVES

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	24	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	24	
Input Traffic Index (TI)	6	
Calculated Total Gravel Equivalent (GE)	1.4592	feet
Calculated Total Gravel Equivalent (GE)	17.5104	inches
Calculated Gravel Factor (Gf) for A/C paving	2.31	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent GE (feet)	GE (inches)	Delta (inches)	INCHES A/C Section Thickness (inches)	INCHES Minimum Base (inches)	FEET A/C Section Thickness (feet)	FEET Minimum Base (feet)
0.58	6.94	10.57	3.0	9.6	0.25	0.80
0.67	8.10	9.41	3.5	8.4	0.29	0.70
0.69	8.33	9.18	3.6	8.4	0.30	0.70
0.77	9.26	8.25	4.0	7.8	0.33	0.65
0.81	9.72	7.79	4.2	7.2	0.35	0.60
0.87	10.41	7.10	4.5	6.6	0.38	0.55
0.93	11.11	6.40	4.8	6.0	0.40	0.50
0.96	11.57	5.94	5.0	5.4	0.42	0.45
1.16	13.88	3.63	6.0	3.0	0.50	0.25
1.93	23.14	-5.63	10.0		0.83	
2.31	27.77	-10.26	12.0		1.00	

PAVING DESIGN

JN: 18840217 CONSULT: SMP
PROJECT: Newberry Springs

CALCULATION SHEET # Entrances/TruckDRIVES

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	24	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	24	
Input Traffic Index (TI)	7	
Calculated Total Gravel Equivalent (GE)	1.7024	feet
Calculated Total Gravel Equivalent (GE)	20.4288	inches
Calculated Gravel Factor (Gf) for A/C paving	2.14	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent			INCHES		FEET	
GE (feet)	GE (inches)	Delta (inches)	A/C Section Thickness (inches)	Minimum Base (inches)	A/C Section Thickness (feet)	Minimum Base (feet)
0.54	6.43	14.00	3.0	12.6	0.25	1.05
0.62	7.50	12.93	3.5	12.0	0.29	1.00
0.64	7.71	12.72	3.6	11.4	0.30	0.95
0.71	8.57	11.86	4.0	10.8	0.33	0.90
0.75	9.00	11.43	4.2	10.2	0.35	0.85
0.80	9.64	10.79	4.5	9.6	0.38	0.80
0.86	10.28	10.15	4.8	9.0	0.40	0.75
0.89	10.71	9.72	5.0	9.0	0.42	0.75
1.07	12.85	7.58	6.0	6.6	0.50	0.55
1.79	21.42	-0.99	10.0		0.83	
2.14	25.71	-5.28	12.0		1.00	

PAVING DESIGN

JN: 18840217 CONSULT: SMP
PROJECT: Newberry Springs

CALCULATION SHEET # **TruckCOLLECTOR**

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	24	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	24	
Input Traffic Index (TI)	9	
Calculated Total Gravel Equivalent (GE)	2.1888	feet
Calculated Total Gravel Equivalent (GE)	26.2656	inches
Calculated Gravel Factor (Gf) for A/C paving	1.89	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent GE (feet)	Gravel Equivalent GE (inches)	Delta (inches)	INCHES A/C Section Thickness (inches)	INCHES Minimum Base (inches)	FEET A/C Section Thickness (feet)	FEET Minimum Base (feet)
0.79	9.45	16.82	5.0	15.0	0.42	1.25
0.94	11.34	14.93	6.0	13.8	0.50	1.15
1.10	13.23	13.04	7.0	12.0	0.58	1.00
1.26	15.11	11.15	8.0	10.2	0.67	0.85
1.42	17.00	9.26	9.0	8.4	0.75	0.70
1.57	18.89	7.37	10.0	6.6	0.83	0.55
1.73	20.78	5.48	11.0	4.8	0.92	0.40
1.89	22.67	3.59	12.0	3.0	1.00	0.25
2.05	24.56	1.70	13.0	1.8	1.08	0.15
2.20	26.45	-0.18	14.0		1.17	
2.36	28.34	-2.07	15.0		1.25	