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January 13, 2021

ALDI, Inc., Moreno Valley Division
12661 ALDI Place
Moreno Valley, California 92555

MTG$_L$ Project No: 5403A42
MTG$_L$ Log No: 21-1384
MTG$_L$ Branch: Riverside

Attention: Mr. Ben Vesper | Director of Real Estate

Subject: **UPDATE – GEOTECHNICAL REPORT**
ALDI Store
Route 62 (Twentynine Palms Highway) and Warren Vista Avenue
Yucca Valley, San Bernardino County, California

References: Professional Service Industries (PSI), Inc., 2008, "*Geotechnical Exploration Report, Proposed Commercial Development, Route 62 and Warren Vista Ave., Yucca Valley, California*", PSI Project No. 070-85007, dated March 25, 2008

INTRODUCTION

In accordance with your request, MTG$_L$, Inc. (MTGL) has prepared this report to provide an Update to the referenced Geotechnical Exploration Report dated March 25, 2008 for the subject site. The referenced report dated March 25, 2008 was prepared for the design and construction of a proposed commercial shopping center. The subject project is located within the proposed shopping center. The project site is located adjacent to the southwest corner of Twentynine Palms Highway (California State Highway 62) and Warren Vista Avenue in the unincorporated community of Yucca Valley, San Bernardino County, California (see attached Conceptual Site Plan, Figure 1).

PROJECT REVIEW

Based upon review of the Site Plan supplied (see Figure 1), MTGL understands that plans are to construct an approximate 20,473 square foot single-story retail building on the project site. Associated utility and pavement improvements are planned as part of the project development.

The proposed ALDI retail building is anticipated to be of wood frame/masonry construction with a concrete slab-on-grade floor and will have an attached truck loading dock. Estimated maximum loads for the proposed structure are approximately 2,500 plf for continuous foundations and 100 kips for isolated pad/column foundations. Sewage disposal is anticipated to be provided by the existing shopping center public sewer system. Due to the relatively flat site topography, maximum permanent slope heights of 10 feet are anticipated.

The referenced report dated March 25, 2008 was prepared for the design and construction of a proposed commercial shopping center. The subject project is located within the proposed shopping center. Four (4) of the borings conducted for that report (Borings B-8, B-9, B-10, and B-11) were performed within or directly adjacent to the proposed building (See Figure 1).

The test borings performed for the referenced report dated March 25, 2008 revealed the presence of approximately 1 foot of disturbed soil or topsoil underlain by native alluvial soil which extended to the maximum depth drilled of approximately 52 feet below the native ground surface. The soil within 10 feet of the ground surface was typically loose to medium dense. Below 10 feet the soils were found to be medium dense to dense. No groundwater was encountered to the maximum explored depth of approximately 52 feet below the native ground surface at the time of drilling.

As discussed in the referenced report dated March 25, 2008, the soil borings performed for that report encountered loose alluvial deposits near the ground surface with a moderate potential for collapse. These materials were not considered suitable for support of proposed new structures or pavements in their native condition. Remedial grading, consisting of over-excavation and recompaction of the soils beneath the proposed structure and pavements was recommended.

RECOMMENDATIONS

Based upon review of the referenced Geotechnical Investigation dated March 25, 2008 and current development plans, it is MTGL, Inc.'s opinion that the findings, conclusions, and recommendations contained within the referenced Geotechnical Investigation dated March 25, 2008, except as amended in this report, remain valid and should be followed and implemented during future project design and construction.

Seismic Design Parameters

The subsurface conditions of the site were found to be consistent with the characteristics of Site Class D. For a site class D, a site-specific ground motion analysis is required to be performed in accordance with the requirements of 2019 CBC and ASCE 7-16. As part of the site-specific analysis, base ground motions were evaluated in conjunction with both a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA) to characterize earthquake ground shaking that may occur at the site during future seismic events.

The PSHA is based on an assessment of the recurrence of earthquakes on potential seismic sources in the region and on ground motion prediction models of different seismic sources in the region. The United States Geological Survey (USGS) unified hazard analysis tool was used to develop a seismic hazard curve and the USGS risk targeted ground motion calculator was used to analyze ground motions for corresponding periods. Maximum directional scale factors were applied to the results to develop the probabilistic ground motion model specific to this site.

The DSHA is represented by the 84th percentile of the spectral accelerations for different periods using Pacific Earthquake Engineering Research Center's (PEER) Next Generation Attenuation West-2, Ground Motion Prediction Equations (NGA West 2 GMPE) tool. Fault parameters including the magnitude and width required for the NGA West 2 GMPE tool were obtained from the USGS Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) model. After applying maximum directional scale factors appropriate for each period, the maximum directional deterministic model specific to the site was developed.

Based on the PSHA and DSHA models, the Site-Specific Risk-Targeted Maximum Considered Earthquake (MCER) was taken as the lesser of the spectral response accelerations from the PSHA and DSHA. The design response spectrum and design acceleration parameters were calculated in accordance with the procedures of ASCE 7-16. The site coefficients and maximum considered earthquake spectral response acceleration parameters are presented below. Tabulated values and graphical plots are included in Appendix A of this report.

Ground Motion Parameter	**Design Value**
S_S	2.292 g
S_1	0.826 g
Site Class	D
F_a	1.2
F_v	2.5
S_{DS}	1.834 g
S_{D1}	1.318 g
S_{MS}	2.165 g
S_{M1}	1.976 g
T_L	8 Seconds
Site Specific PGA	0.865 g
Shear-Wave Velocity (V_{100})	970 ft/sec (estimated)
Rick Category	II

Remedial Site Grading

As discussed in the referenced report dated March 25, 2008, remedial site over-excavations were recommended for proposed site improvements. It is recommended that the existing soils within the proposed building areas, including a minimum distance of 5 feet beyond the lateral edge of perimeter footings, be over excavated to a minimum depth of 5 feet below existing grade, or the base of the proposed foundation, whichever is greater. The resulting surface should be scarified (ripped) eight (8) inches. The scarified soils should be moisture conditioned to near optimum moisture content (±2 percent) and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557 test procedures.

Backfill of all remedial excavations, and placement of fill to finish grades, should be performed with acceptable fill materials that have been properly moisture conditioned (±2 percent of optimum moisture content) and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D 1557 test procedures. All remedial grading and compaction operations should be observed by a representative of the geotechnical consultant. Compaction should be verified by testing.

Foundations

Provided the recommended building pad remedial grading is successfully performed, conventional shallow spread and/or continuous foundations may be used to support the proposed structure. Spread and/or continuous footings founded in compacted fill materials may be used to support the proposed structure. Foundations should be designed using an allowable bearing pressure of 3,000 psf. This allowable bearing pressure may be increased by 20% for each additional foot of width and/or depth, to a maximum value of 5,000 psf. The allowable bearing capacity may also be increased by one-third for considerations of short-term wind or seismic loads. The recommended minimum footing width and embedment depth below the lowest adjacent grade are as follows:

Foundation Type	Minimum Width	Minimum Depth*
Continuous (Interior)	15 inches	18 inches
Continuous (Perimeter)	15 inches	18 inches
Spread Footings (Interior)	24 inches	18 inches
Continuous (Wall Footing)	15 inches	18 inches

*The minimum depth of foundations is relative to the lowest adjacent soil subgrade for perimeter foundations and finished grade (top of concrete) for interior foundations.

Soil resistance developed against lateral structural movement can be obtained from the passive pressure value of 250 pcf. The upper one foot of passive pressure should be neglected unless confined by pavement or slab. For sliding resistance, a friction coefficient of 0.40 may be used at the concrete and soil interface. The passive pressure and the friction of resistance could be combined without reduction. In addition, the lateral passive resistance is applicable only if it is ensured that the soil against embedded structures will remain intact with time.

The near surface soils have an expansion index classification of "very low" (0-20). Therefore, nominal reinforcement consisting of two #4 bars placed within 3 inches of the top of footings and two #4 bars placed within 3 inches of the bottom of footings are recommended. However, the structural engineer may require heavier reinforcement.

Foundations should be designed to resist the anticipated settlements. Settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. It is estimated maximum settlement of foundations designed and constructed in accordance with the recommendations presented in this report to be on the order of 1 inch. Differential settlement between similarly loaded and adjacent footings are expected to be a maximum of approximately ½ inch across 40 feet, provided footings are founded on similar materials. Settlement of all foundations is expected to occur rapidly and should be essentially complete shortly after initial application of the loads.

Concrete Slabs-on-Grade and Miscellaneous Flatwork

Exterior concrete slabs-on-grade for buildings and miscellaneous flatwork that are not subjected to vehicular loads may be designed with a minimum (actual) thickness of 4.0 inches for normal loading conditions. Concrete slabs-on-grade for buildings and miscellaneous flatwork that are subjected to vehicular loads may be designed with a minimum (actual) thickness of 5.0 inches for normal loading conditions. However, if heavier loads are anticipated, a modulus of subgrade reaction of 150 pounds per cubic inch may be used when the slabs are supported by compacted fill.

All building slabs and miscellaneous flatwork should be reinforced with a minimum #4 bars, 12 inches on center, each direction, placed at the mid-height of the slab. The structural engineer may require heavier reinforcement. Special care should be taken so that reinforcement is placed at the slab mid-height. The floor slab should be separated from footings, structural walls, and utilities and provisions made to allow for settlement or swelling movements at these interfaces. If this is not possible from a structural or architectural design standpoint, it is recommended that the slab connection to footings be reinforced such that there will be resistance to potential differential movement.

Control joints should be constructed on all slabs on grade to create squares or rectangles with a maximum spacing of 12 feet on large slab areas. Where flatwork is adjacent to curbs, reinforcing bars should be placed between the flatwork and the curbs. Expansion joint material should be used between flatwork and curbs, and flatwork and buildings.

Subsurface moisture and moisture vapor naturally migrate upward through the soil and where the soil is covered by a building or pavement. To reduce the impact of the subsurface moisture and potential impact of future introduced moisture (such as landscape irrigation or precipitation) damp proofing should be provided under all slabs on grade with moisture sensitive floor coverings. The damp proofing should consist of a minimum 15 mil polyethylene liner placed with 2 inches of sand below and 2 inches of sand above the polyethylene liner. The liner should be carefully fitted around service openings with joints lapped not less than 6 inches.

Damp proofing typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards. Other factors such as surface grades, adjacent planters, the quality of slab concrete and the permeability of the on-site soils will affect slab moisture. In many cases, floor moisture problems are the result of either improper curing of floors slabs or improper application of flooring adhesives. We recommend contacting a flooring consultant experienced with concrete slab-on-grade floors for specific recommendations regarding the proposed flooring applications. MTGL makes no guarantee nor provide any assurance that use of a vapor retarder system will reduce concrete slab-on-grade floor moisture penetration to any specific rate or level, particularly those required by floor covering manufacturers. The builder and designers should consider all available measures for floor slab moisture protection.

Special precautions must be taken during the placement and curing of all concrete slabs. Excessive slump (high water-cement ratio) of the concrete and/or improper curing procedures used during either hot or cold weather conditions could lead to excessive shrinkage, cracking, or curling of the slabs. High water-cement ratio and/or improper curing also greatly increase the water vapor permeability of concrete. It is recommended that all concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) manual.

The subgrade soils beneath all concrete flatwork should be compacted to a minimum of 90% relative compaction for a minimum depth of 24 inches. The geotechnical engineer should monitor the compaction of subgrade soils and perform testing to verify that compaction has been obtained.

Prewetting Recommendation

Prior to placing concrete slabs and flatwork, the underlying soils should be brought to at least optimum moisture content for a depth of 12 inches prior to the placement of concrete. The geotechnical consultant should perform in-situ moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers. Once the slab subgrade soil has been pre-wetted and compacted, the soil should not be allowed to dry prior to concrete placement. If the subgrade soil is dry, the moisture content of the soil should be restored prior to placement of concrete and re-tested.

Proper moisture conditioning and compaction of subgrade soils prior to placement is very important prior to concrete placement. Even with proper site preparation, some soil moisture changes of the subgrade soils supporting the concrete flatwork due to edge effects (shrink/swell) may occur. Drying and/or wetting of subgrade soils adjacent to landscaped areas or open fields may increase the potential of shrink/swell effects beneath concrete flatwork areas. To help reduce edge effects, lateral cutoffs, such as inverted curbs are recommended. Control joints should be used to reduce the potential for flatwork panel cracks due to minor soil shrink/swell.

Soil Corrosivity

Soluble sulfate tests indicate that concrete at the subject site will have a "moderate" (Class S1) exposure to water soluble sulfate in the soil. Recommendations for concrete exposed to sulfate-containing soils are presented below.

RECOMMENDATIONS FOR CONCRETE EXPOSED TO SULFATE CONTAINING SOILS

Sulfate Exposure Severity	Class	Water soluble sulfate (SO_4) in soil (% by wt.)	Sulfate (SO_4) in water (ppm)	Max Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)	Cement Type	Calcium Chloride Admixture
Negligible	S0	0.00 - 0.10	0-150	---	2,500	---	No Restriction
Moderate	**S1**	**0.10 - 0.20**	**150-1,500**	**0.50**	**4,000**	**II/V**	**No Restriction**
Severe	S2	0.20 - 2.00	1,500-10,000	0.45	4,500	V	Not Permitted
Very Severe	S3	Over 2.00	Over 10,000	0.45	4,500	V Plus Pozzolan	Not Permitted

The upper site soils are anticipated to have a moderate corrosion potential. Protection of buried metal pipelines utilizing coatings along with using clean import sand bedding and a cathodic protection system should be used to reduce the soil corrosion potential. A qualified corrosion engineer should be consulted to further assess the corrosion potential, as necessary.

Retaining Walls

Embedded structural walls should be designed for lateral earth pressures exerted on the walls. The magnitude of these earth pressures will depend on the amount of deformation that the wall can yield under the load. If the wall can yield sufficiently to mobilize the full shear strength of the soils, it may be designed for the "active" condition. If the wall cannot yield under the applied load, then the shear strength of the soil cannot be mobilized, and the earth pressures will be higher. These walls such as basement walls, loading dock walls, and swimming pools should be designed for the "at rest" condition. If a structure moves towards the retained soils, the resulting resistance developed by the soil will be the "passive" resistance.

For design purposes, the recommended equivalent fluid pressure for each case for walls constructed above the static groundwater table and backfilled with non-expansive soils is provided below. Retaining wall backfill should be compacted to at least 90% relative compaction based on the maximum density defined by ASTM D1557. Retaining structures may be designed to resist the following lateral earth pressures. If any super-imposed loads are anticipated, this office should be notified so that appropriate recommendations for earth pressures may be provided.

- Allowable Bearing Pressure – 3,000 psf
- Coefficient of Friction (Soil to Footing) – 0.40
- Passive Earth Pressure - equivalent fluid weight of 300 pcf (Maximum of 2,500 psf)
- At rest lateral earth pressure – 55 pcf
- Active Earth Pressures – equivalent fluid weights:

Slope of Retained Material	Equivalent Fluid Weight (pcf)
Level	35
2:1 (H:V)	55

It is recommended that all retaining wall footings be embedded at least 18 inches below the lowest adjacent finish grade. In addition, the wall footings should be designed and reinforced as required for structural considerations. The wall areas should be over-excavated to a minimum depth of one (1) foot below the bottom of the proposed footings. The required horizontal limits of the over excavated area shall be defined as the area extending from the edge of the footing for a minimum distance of 2 feet.

Retaining structures should be drained to prevent the accumulation of subsurface water behind the walls. Back drains should be installed behind all retaining walls exceeding 3.0 feet in height. Alternatively, a pre-manufactured drainage product (i.e. Mira-Drain™ or equivalent) may be utilized instead of an aggregate drain. All back drains should be outlet to suitable drainage devices. Walls and portions thereof that retain soil and enclose interior spaces and floors below grade should be waterproofed and damp-proofed accordingly.

Seismically Induced Lateral Earth Pressures

A seismic lateral increment of 22 pcf (equivalent fluid weight) may be applied as an incremental force which should be applied to the back of the wall in the upper 1/3 of the wall and also applied as a reduction of force to the front of the wall in the upper 1/3 of the footing.

Pavement Recommendations

Recommended pavement structural sections are based on the procedures outlined in "Design Procedures for Flexible Pavements" of the Highway Design Manual, California Transportation Department. This procedure uses the principal that the pavement structural section must be of adequate thickness to distribute the load from the design traffic (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R value).

Pavement sections were designed based on an R-Value of 30 and assumed Traffic Index of 5.0 for light auto parking and drive lanes, 6.0 for commercial vehicles, and 8.0 for truck access/fire lanes. The recommend structural sections are as follows:

ASPHALT PAVEMENT STRUCTURAL SECTION

Pavement Area	Traffic Index	Asphalt Thickness	Aggregate Base Thickness
Light Auto Parking / Drive Lanes	5.0	3.0"	6.0"
Commercial Vehicles	6.0	4.0"	8.0"
Truck Access/Fire Lane (Heavy Truck Traffic)	8.0	5.0"	8.0"

Portland cement concrete (PCC) pavements for areas which are subject to traffic loads may be designed with a minimum thickness of 5.0 inches of Portland cement concrete (minimum compressive strength of 4,000 psi) on 6.0 inches of compacted aggregate base.

Prior to paving, the exposed subgrade soils should be scarified, adjusted to within 2% of optimum moisture and compacted to a minimum of 90% relative compaction for a minimum depth of 12 inches. All aggregate base courses should be compacted to a minimum of 95% relative compaction. Compaction should be confirmed by testing.

Construction Considerations

Moisture Sensitive Soils/Weather Related Concerns

The upper soils encountered at this site may be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and its support capabilities. In addition, soils that become excessively wet may be slow to dry and thus significantly delay the progress of the grading operations. Therefore, it will be advantageous to perform earthwork and foundation construction activities during the dry season. Much of the on-site soils may be susceptible to erosion during periods of inclement weather. As a result, the project Civil Engineer/Architect and Grading Contractor should take appropriate precautions to reduce the potential for erosion during and after construction.

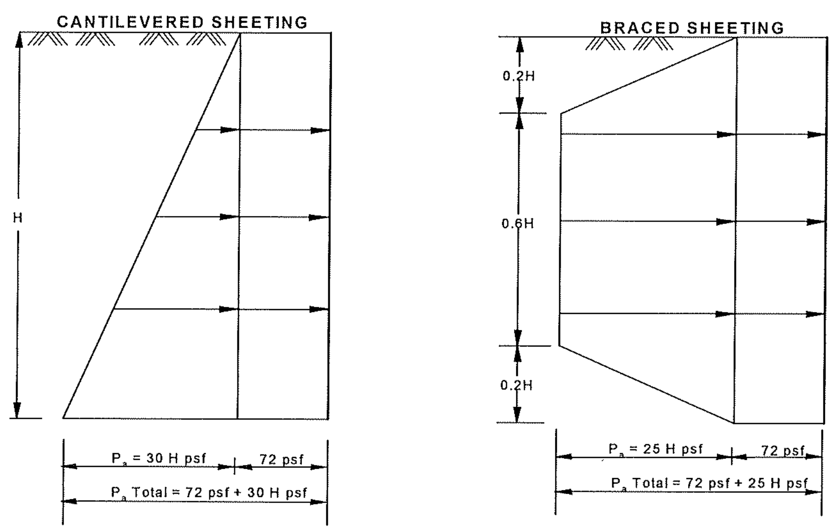
Drainage and Groundwater Considerations

Historic high groundwater levels in the immediate site vicinity are deeper than 50 feet below grade. Since this is below the anticipated depths of grading, the installation of subdrains is not expected to be necessary. However, variations in the ground water table may result from fluctuation in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors such as impermeable and/or cemented formational materials overlain by fill soils. In addition, during retaining wall excavations, seepage may be encountered. Therefore, it is recommended that a representative of MTG$_L$, Inc. be present during grading operations to evaluate areas of seepage. Drainage devices for reduction of water accumulation can be recommended should these conditions occur.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Undercut or excavated areas should be sloped to facilitate removal of any collected rainwater, groundwater, or surface runoff. Positive site drainage should be provided to reduce infiltration of surface water around the perimeter of the building and beneath the floor slabs. The grades should be sloped away from the building and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill and floor slab areas of the building.

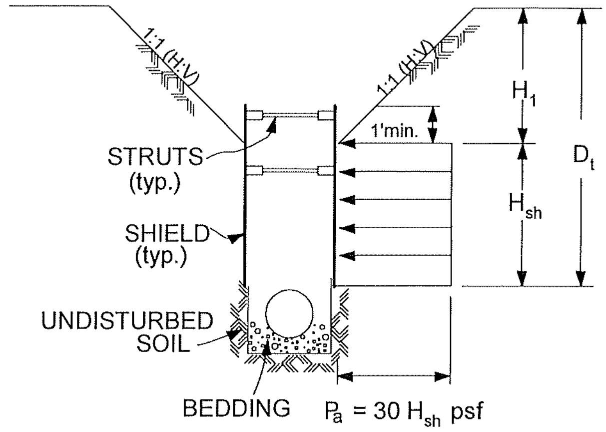
Temporary Excavations and Shoring

Short term temporary excavations in existing soils may be safely made at an inclination of 1:1 (horizontal to vertical) or flatter. If vertical sidewalls are required in excavations greater than 5 feet in depth, the use of cantilevered or braced shoring is recommended. Excavations less than 5 feet in depth may be constructed with vertical sidewalls without shoring or shielding. Our recommendations for lateral earth pressures to be used in the design of cantilevered and/or braced shoring are presented below. These values incorporate a uniform lateral pressure of 72 psf to provide for the normal construction loads imposed by vehicles, equipment, materials, and workmen on the surface adjacent to the trench excavation. However, if vehicles, equipment, materials, etc. are kept a minimum distance equal to the height of the excavation away from the edge of the excavation, this surcharge load need not be applied.



SHORING DESIGN: LATERAL SHORING PRESSURES

Design of the shield struts should be based on a value of 0.65 times the indicated pressure, Pa, for the approximate trench depth. The wales and sheeting can be designed for a value of 2/3 the design strut value.



HEIGHT OF SHIELD, H_{sh} = DEPTH OF TRENCH, D_t , MINUS DEPTH OF SLOPE, H_1

TYPICAL SHORING
DETAIL

Placement of the shield may be made after the excavation is completed or driven down as the material is excavated from inside of the shield. If placed after the excavation, some excavation may be required to allow for the shield width and advancement of the shield. The shield may be placed at either the top or the bottom of the pipe zone. Due to the anticipated thinness of the shield walls, removal of the shield after construction should have negligible effects on the load factor of pipes. Shields may be successively placed with conventional trenching equipment.

Vehicles, equipment, materials, etc. should be set back away from the edge of temporary excavations a minimum distance of 15 feet from the top edge of the excavation. Surface waters should be diverted away from temporary excavations and prevented from draining over the top of the excavation and down the slope face. During periods of heavy rain, the slope face should be protected with sandbags to prevent drainage over the edge of the slope, and a visqueen liner placed on the slope face to prevent erosion of the slope face.

Periodic observations of the excavations should be made by the geotechnical consultant to verify that the soil conditions have not varied from those anticipated and to monitor the overall condition of the temporary excavations over time. If at any time during construction conditions are encountered which differ from those anticipated, the geotechnical consultant should be contacted and allowed to analyze the field conditions prior to commencing work within the excavation. All Cal/OSHA construction safety orders should be observed during all underground work.

Utility Trenches

All Cal/OSHA construction safety orders should be observed during all underground work. All utility trench backfills within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 (horizontal to vertical) drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Utilities should be bedded and backfilled with clean sand or approved granular soil to a depth of at least 1-foot over the pipe. The bedding materials shall consist of sand, gravel, crushed aggregates, or native soils that are free draining with a sand equivalence of not less than 30. The bedding should be uniformly watered and compacted to a firm condition for pipe support.

The remainder of the backfill shall be typical on-site soil or imported soil which should be placed in lifts not exceeding 8 inches in thickness, watered, or aerated to near optimum moisture content, and mechanically compacted to at least 90% of maximum dry density (ASTM D1557).

The bedding and backfill materials and placement shall conform to the requirements of the latest Standard Specifications for Public Works Construction (Greenbook).

Site Drainage

The site should be drained to provide for positive drainage away from structures in accordance with the building code and applicable local requirements. Unpaved areas should slope no less than 2% away from structure. Paved areas should slope no less than 1% away from structures. Concentrated roof and surface drainage from the site should be collected in engineered, non-erosive drainage devices and conducted to a safe point of discharge. The site drainage should be designed by a civil engineer.

Geotechnical Observation/Testing of Earthwork Operations

The recommendations provided in this report are based on preliminary design information and subsurface conditions as interpreted from the investigation. Our preliminary conclusion and recommendations should be reviewed and verified during site grading and revised accordingly if exposed Geotechnical conditions vary from our preliminary findings and interpretations. The Geotechnical consultant should perform Geotechnical observation and testing during the following phases of grading and construction:

- During site grading and remedial excavations.
- During foundation excavations and concrete placement.
- Upon completion of retaining wall footing excavation prior to placing concrete.
- During excavation and backfilling of all utility trenches
- During processing and compaction of the subgrade for the access and parking areas and prior to construction of pavement sections.
- When any unusual or unexpected Geotechnical conditions are encountered during any phase of construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The findings, conclusions, and recommendations contained in this report are based on the site conditions as they existed at the time of MTG$_L$, Inc.'s review, and further assume that the subsurface conditions encountered during MTG$_L$, Inc.'s review are representative of conditions throughout the site. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

This report was prepared for the exclusive use and benefit of the owner, architect, and engineer for evaluating the design of the facilities as it relates to geotechnical aspects. It should be made available to prospective contractors for information on factual data only, and not as a warranty of subsurface conditions included in this report.

MTG$_L$, Inc.'s review was performed using the standard of care and level of skill ordinarily exercised under similar circumstances by reputable soil engineers and geologists currently practicing in this or similar localities. No other warranty, express or implied, is made as to the conclusions and professional advice included in this report.

This firm does not practice or consult in the field of safety engineering. MTG$_L$, Inc. does not direct the Contractor's operations, and are not responsible for their actions. The contractor will be solely and completely responsible for working conditions on the job site, including the safety of all persons and property during performance of the work. This responsibility will apply continuously and will not be limited to MTG$_L$, Inc.'s normal hours of operation.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge.

Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

All other findings, conclusions, and recommendations contained within the referenced Geotechnical Investigation report dated March 25, 2008, unless amended in this report, remain valid and apply to this report.

CLOSURE

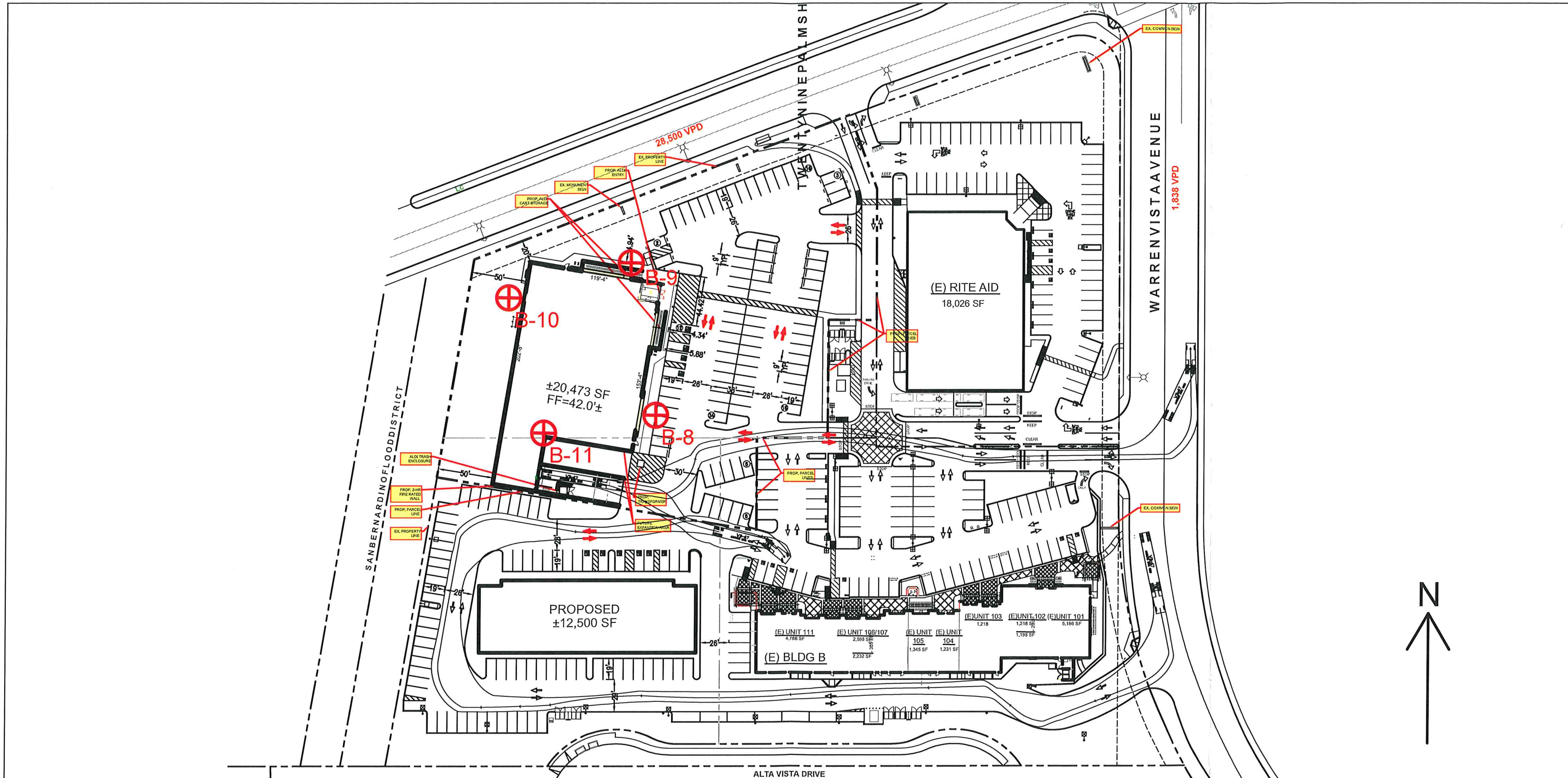
MTGL, Inc. appreciates this opportunity to be of continued service to you on this project. Should you have any questions regarding the information contained herein, or require additional services, please contact us at your earliest convenience.

Respectfully Submitted,
MTGL, Inc.

REGISTERED PROFESSIONAL ENGINEER
BRUCE A. HICK
No. GE2284
Exp. 12/31/~~16~~ 24
GEOTECHNICAL
STATE OF CALIFORNIA

Bruce A. Hick, P.E., G.E.
Vice President | Engineering Manager

Attachments: Figure 1 –Site Plan
Appendix A – Site Specific Ground Motion Analysis



LEGEND:
B-11 DENOTES APPROXIMATE LOCATION OF EXPLORATORY BORING FROM REPORT DATED MARCH 25, 2008

NO.	REVISION DESCRIPTION	AUTHOR	DATE

REVISIONS

DRAWN ON:	January 13, 2021
CHECKED BY:	
CLIENT:	

PROJECT:	ALDI STORE - YUCCA VALLEY	NO.	5403A42
DRAWING:	SITE PLAN		
FIGURE:	1	TWENTYNINE PALMS HWY & WARREN VISTA DRIVE, YUCCA VALLEY, CA	
SCALE:	NTS		

MTGL

APPENDIX A

Site Specific Ground Motion Analysis

SITE SPECIFIC GROUND MOTION ANALYSIS (ASCE 7-16)

Project: Aldi - Yucca Valley		**Latitude:**	34.125 deg	**Calculated:**	JR
Client: Aldi		**Longitude:**	-116.407 deg	**Checked:**	BH
Job No: 5403A42		**Vs_{30}:**	293 m/s	**Date:**	January, 2021

	PROBABILISTIC ANALYSIS				DETERMINISTIC ANALYSIS			CODE-BASED (LOWER LIMIT)			DESIGN RESPONSE SPECTRUM		
Period T (sec)	Uniform Hazard Ground Motion (g)	Risk Targeted Ground Motion (g)	Maximum Direction Scale Factor	Maximum Direction RTGM (g)	84th percentile Spectral Accelaration (g)	Maximum Direction Scale Factor	Maximum Directional Deterministic Sa (g)	Design S_a (g)	Code Based S_a (g)	80% of Code Based S_a (g)	Design S_{aM} (g)	Design S_a (g)	T x S_a (T>1s)
PGA	1.02	0.975	1.1	1.073	0.865	1.1	0.952	0.634	0.734	0.587	0.952	0.634	---
0.10	1.670	1.633	1.1	1.796	1.288	1.1	1.417	0.945	1.467	1.173	1.417	0.945	---
0.20	2.179	2.127	1.1	2.340	1.734	1.1	1.907	1.271	1.834	1.467	1.907	1.271	---
0.30	2.471	2.373	1.125	2.670	2.032	1.125	2.287	1.524	1.834	1.467	2.287	1.524	---
0.50	2.477	2.301	1.175	2.704	2.048	1.175	2.406	1.604	1.834	1.467	2.406	1.604	---
0.75	2.105	1.905	1.2375	2.357	1.728	1.2375	2.138	1.425	1.834	1.467	2.138	1.425	---
1.00	1.774	1.602	1.3	2.083	1.444	1.3	1.878	1.252	1.377	1.101	1.878	1.252	1.252
2.00	0.960	0.851	1.35	1.149	0.732	1.35	0.988	0.659	0.688	0.551	0.988	0.659	1.318
3.00	0.610	0.543	1.4	0.760	0.465	1.4	0.650	0.434	0.459	0.367	0.650	0.434	1.301
4.00	0.413	0.368	1.45	0.534	0.308	1.45	0.447	0.298	0.344	0.275	0.447	0.298	1.192
5.00	0.302	0.269	1.5	0.404	0.216	1.5	0.323	0.216	0.275	0.220	0.323	0.216	1.078

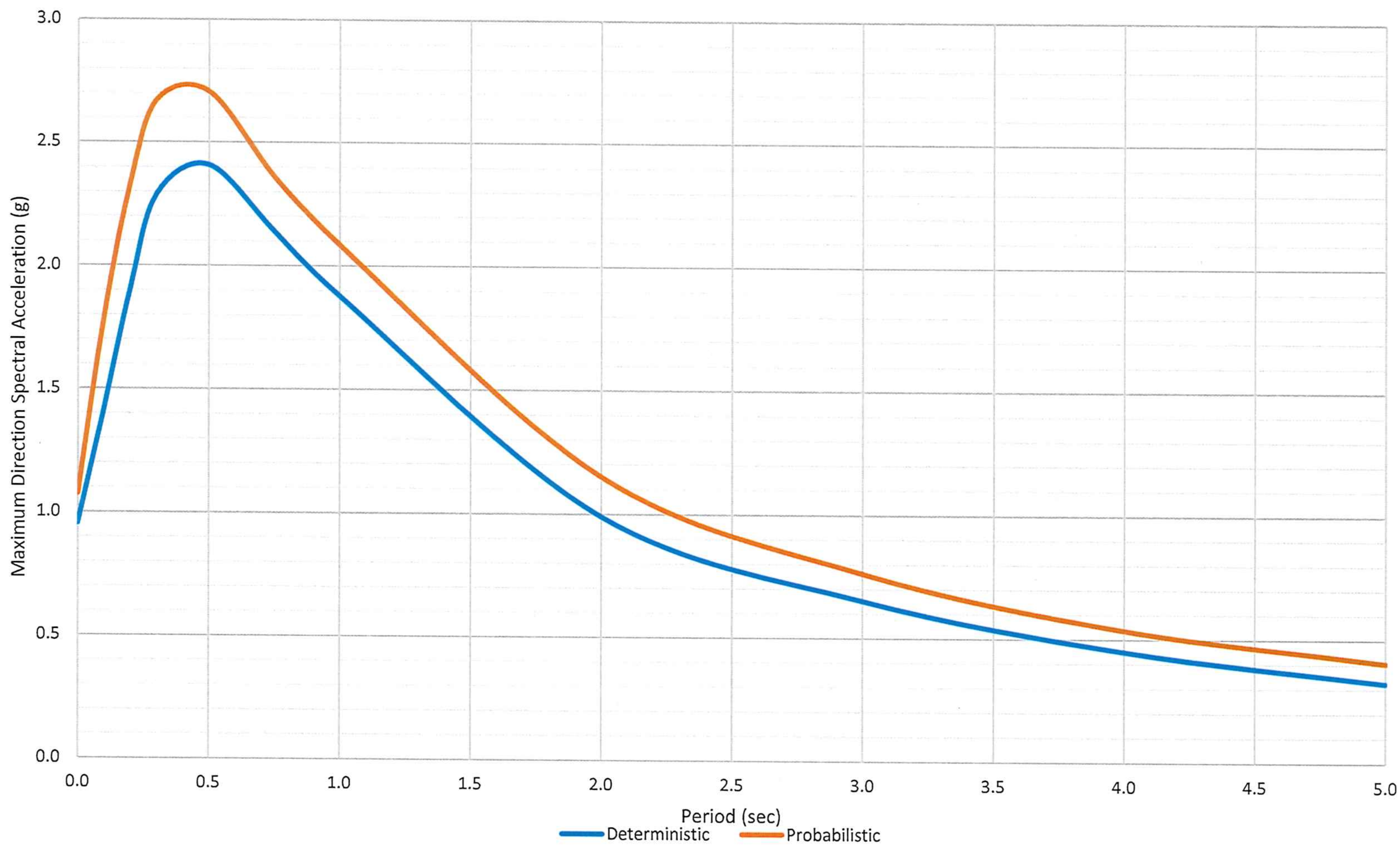
INPUT PARAMETERS - SEAOC (https://seismicmaps.org/)

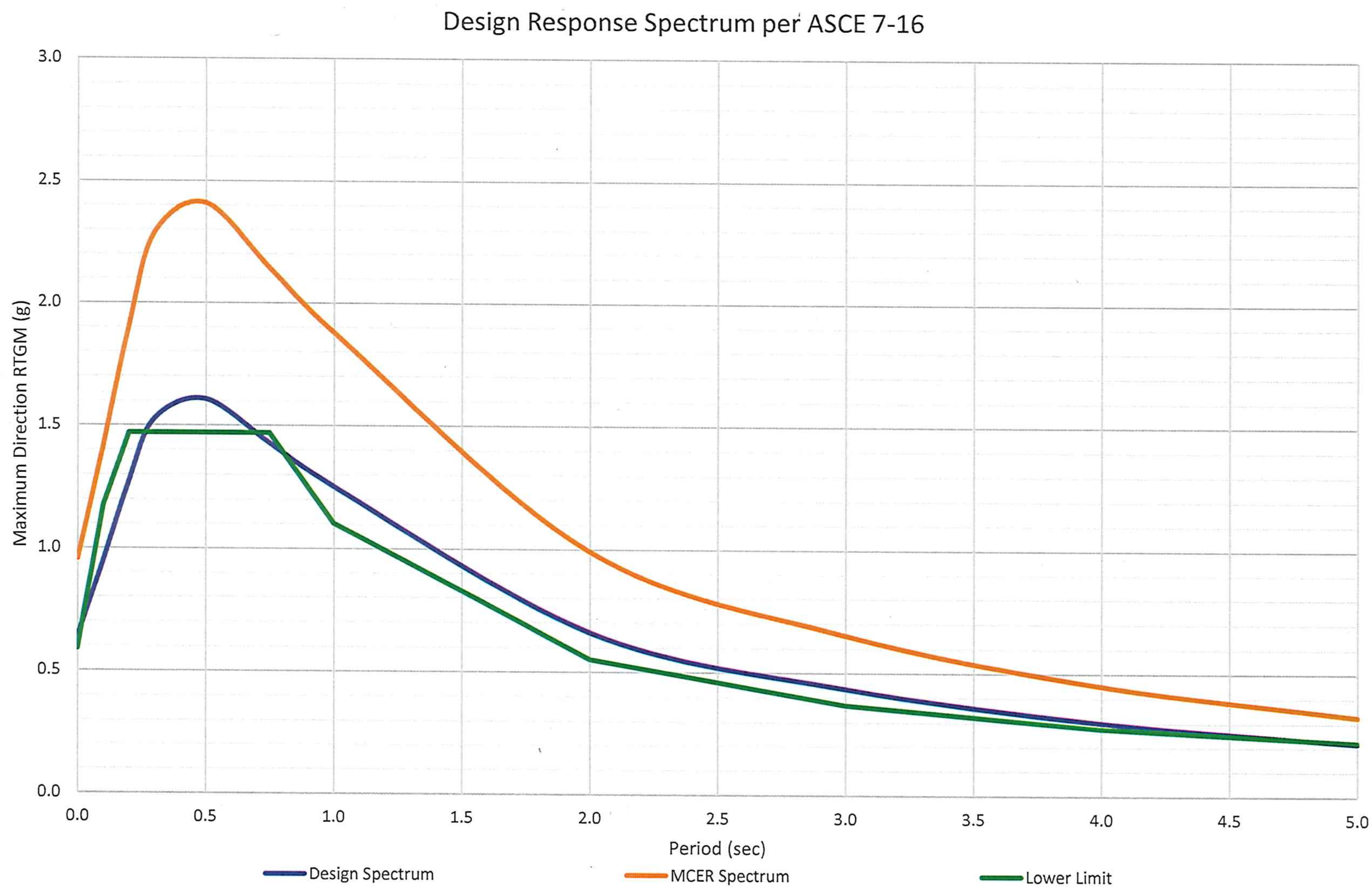
Site Class=	D	
F_a=	1.200	Short Period Site Coefficient
S_S=	2.292	Mapped MCE_R, 5% Damped at T=0.2s
S_1=	0.826	Mapped MCE_R, 5% Damped at T=1s
S_{DS}=	1.834	Design, 5% Damped at Short Periods
T_L (sec)=	8	Long Period Transition (Sect 11.4.6)
F_{PGA} (g)=	1.2	Site Coefficient for PGA
PGA_M (g)=	1.142	
S_{M1}=	2.065	The MCE_R, 5% Damped at T=1s
S_{D1}=	1.377	Design, 5% Damped at T=1s
T_o (sec)=	0.150	Defined in ASCE 7-16 Sect 11.4.6
T_S (sec)=	0.751	Defined in ASCE 7-16 Sect 11.4.6

SITE-SPECIFIC DESIGN PARAMETERS

S_{DS}=	**1.444**	90% of max S_a (ASCE 7-16 Sect 21.4)
S_{MS}=	**2.165**	MCE_R, 5% Damped, adjusted for Site Class
S_{D1}=	**1.318**	Design, 5% Damped, at T=1s (Sect 11.4.5)
S_{M1}=	**1.976**	MCE_R, 5% Damped, at T=1s, adjusted for Site
F_a=	**1.200**	Short Period Site Coefficient
F_v=	**2.500**	Long Period Site Coefficient (7-16 Sect 21.3)
S_S=	**1.805**	MCE_R, 5% Damped at T=0.2s
S_1=	**0.791**	MCE_R, 5% Damped at T=1s
$PGA_{Probabilistic}$ (g)=	**1.02**	Peak Ground Acceleration, Probabilistic
$PGA_{Deterministic}$ (g)=	**0.865**	Peak Ground Acceleration, Deterministic
F_{PGA} (g)=	**1.2**	Site Coefficient for PGA
$0.5*F_{PGA}$ (g)=	**0.600**	OK (Check $PGA_{Deterministic} > 0.5 \times F_{PGA}$)
Site Specific PGA (g) =	**0.865**	NG, Use PG (Check $PGA_{Site\ Specific} > 0.8 \times PGA_M$)

MCE_R Response Spectra Accelerations, ASCE 7-16




Design Response Spectrum per ASCE 7-16
Maximum Direction RTGM (g)
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
Period (sec)
Design Spectrum
MCER Spectrum
Lower Limit

OSHPD

Aldi - Yucca Valley

Latitude, Longitude: 34.1253, -116.4066

Date	11/19/2020, 3:00:22 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	2.292	MCE_R ground motion. (for 0.2 second period)
S_1	0.826	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.75	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.834	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.952	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.142	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.317	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.564	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.292	Factored deterministic acceleration value. (0.2 second)
S1RT	0.826	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.929	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.844	Factored deterministic acceleration value. (1.0 second)
PGAd	0.952	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.904	Mapped value of the risk coefficient at short periods
C_{R1}	0.888	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Latitude

Decimal degrees

34.1253

Longitude

Decimal degrees, negative values for western longitudes

-116.4066

Site Class

259 m/s (Site class D)

Spectral Period

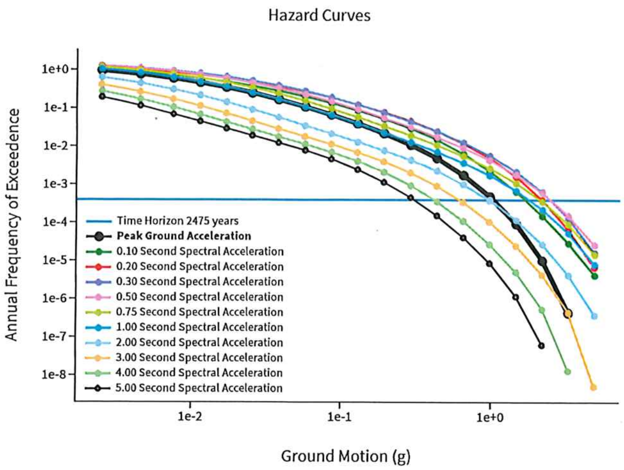
Peak Ground Acceleration

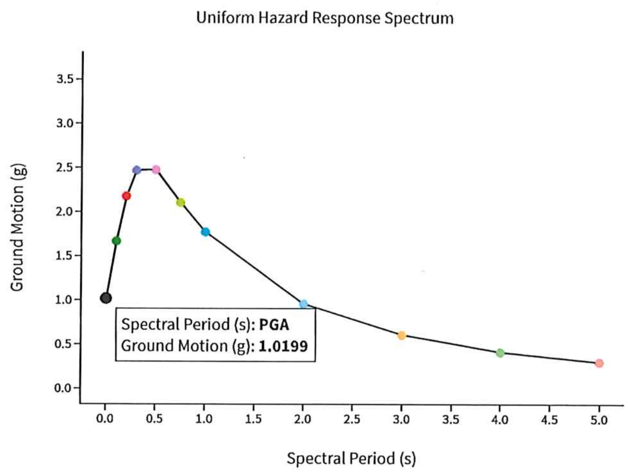
Time Horizon

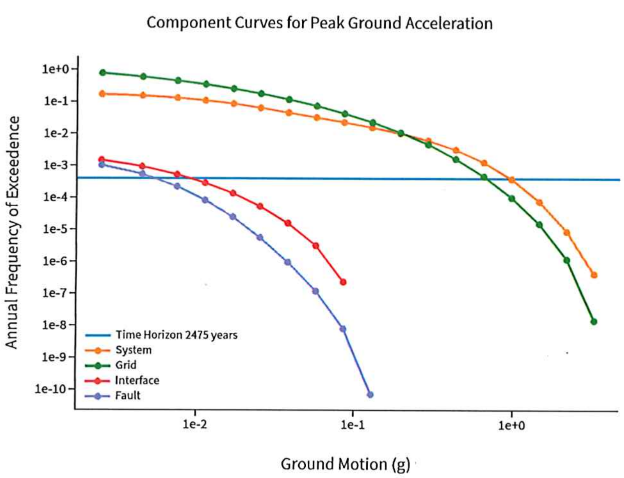
Return period in years

2475

Hazard Curve





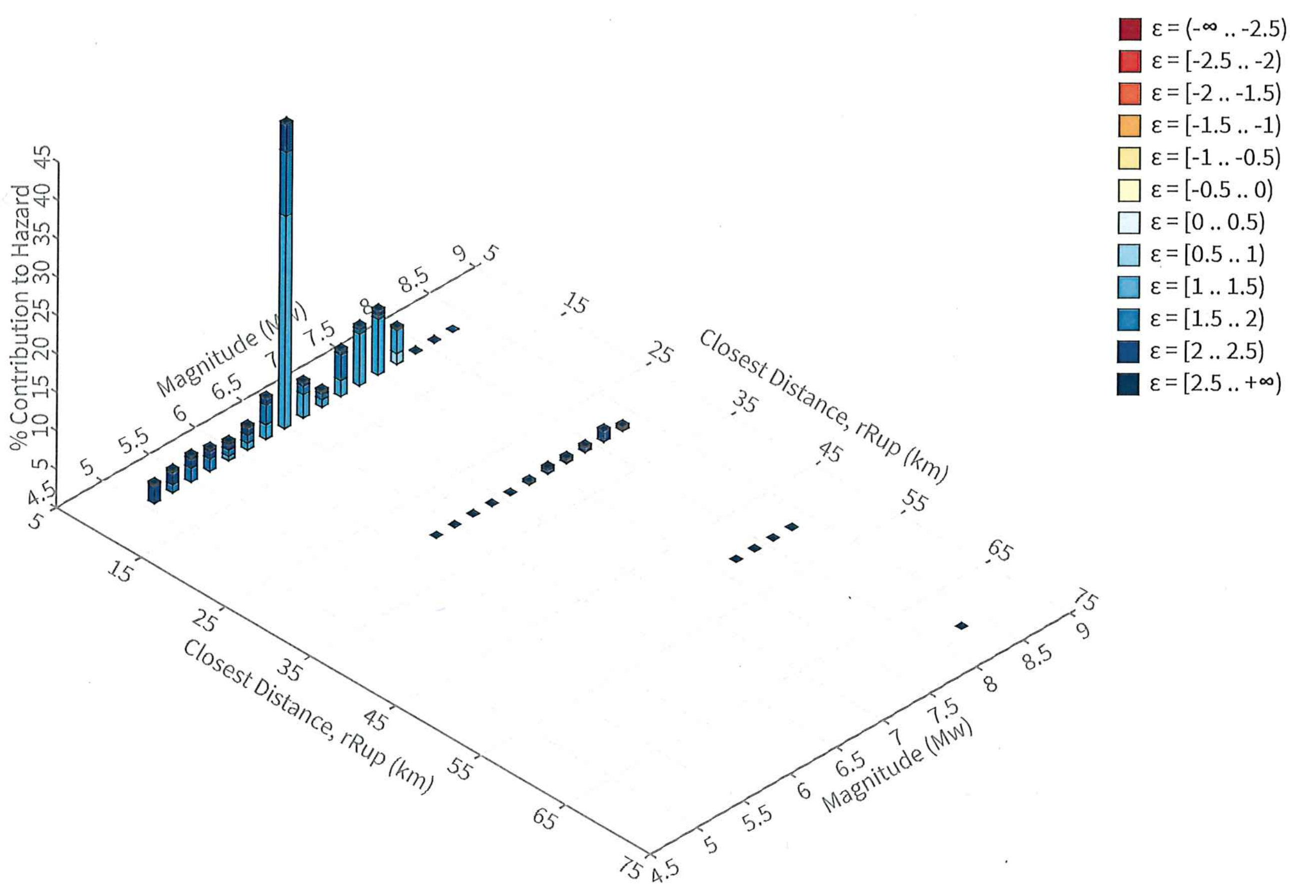


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.0198794 g

Recovered targets

Return period: 3042.1678 yrs
Exceedance rate: 0.00032871297 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.03 %

Mean (over all sources)

m: 6.66
r: 4.38 km
ε_0: 1.56 σ

Mode (largest m-r bin)

m: 6.52
r: 2.17 km
ε_0: 1.44 σ
Contribution: 39.68 %

Mode (largest m-r-ε_0 bin)

m: 6.52
r: 1.94 km
ε_0: 1.3 σ
Contribution: 27.61 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							41.42
Eureka Peak [2]		1.89	6.47	1.42	116.395°W	34.122°N	105.98	13.30
Pinto Mtn [3]		1.73	7.30	1.22	116.412°W	34.131°N	320.91	11.35
Burnt Mtn [0]		2.02	6.58	1.40	116.419°W	34.122°N	250.32	7.26
Johnson Valley (No) 2011 rev [0]		5.20	6.84	1.71	116.421°W	34.168°N	344.48	3.43
Pinto Mtn [4]		1.73	7.34	1.22	116.412°W	34.131°N	320.91	1.61
San Andreas (San Gorgonio Pass-Garnet HIll) [4]		24.28	7.90	2.31	116.517°W	33.893°N	201.58	1.54
UC33brAvg_FM31	System							40.49
Eureka Peak [2]		1.89	6.47	1.42	116.395°W	34.122°N	105.98	12.52
Pinto Mtn [3]		1.73	7.35	1.21	116.412°W	34.131°N	320.91	10.98
Burnt Mtn [0]		2.02	6.58	1.40	116.419°W	34.122°N	250.32	7.30
Johnson Valley (No) 2011 rev [0]		5.20	6.84	1.71	116.421°W	34.168°N	344.48	3.67
Pinto Mtn [4]		1.73	7.35	1.22	116.412°W	34.131°N	320.91	1.57
San Andreas (San Gorgonio Pass-Garnet HIll) [4]		24.28	7.90	2.32	116.517°W	33.893°N	201.58	1.54
UC33brAvg_FM31 (opt)	Grid							9.04
PointSourceFinite: -116.407, 34.148		5.77	5.57	1.87	116.407°W	34.148°N	0.00	3.43
PointSourceFinite: -116.407, 34.148		5.77	5.57	1.87	116.407°W	34.148°N	0.00	3.43
UC33brAvg_FM32 (opt)	Grid							9.04
PointSourceFinite: -116.407, 34.148		5.77	5.57	1.87	116.407°W	34.148°N	0.00	3.43
PointSourceFinite: -116.407, 34.148		5.77	5.57	1.87	116.407°W	34.148°N	0.00	3.43