GEOTECHNICAL ENGINEERING REPORT

Proposed Commercial Development

11279 Cedar Avenue

Bloomington, California

Prepared for

Harry Sidhu

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Prepared by

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1.0 INTRODUCTION

Geo-Cal, Inc. (**GCI**) has prepared this Geotechnical Engineering Report for the proposed commercial development located at 11279 Cedar Avenue in Bloomington, San Bernardino County, California (**Figures 1 and 3**).

This geotechnical investigation was performed concurrently with the infiltration testing for the subject project, thus the field and laboratory information were shared.

1.1 Project Considerations

A Proposed Architectural Site Plan (Sheet A0.1), prepared by Archmetrics, was provided for our use (**Figure 2**).

Based on the Proposed Site Plan, it is our understanding that the proposed Project will consist of a 4 island retail gas station with a convenience store and quick serve restaurant (QSR), a car wash, and a drive-through restaurant.

The anticipated construction includes multi-product fuel dispensers and steel canopy, fuel underground storage tanks (USTs) and associated piping, traffic access and parking pavements, walkways, landscaping, and signage.

Structures of wood or metal frame, reinforced masonry, or similar type construction with slab-on-grade were anticipated. Based upon the type of construction, foundation loads are not anticipated to exceed 1,500 pounds per linear foot for continuous footings and 20 kips for individual spread footings.

At the time of this investigation, the project grading plans were not yet completed. Conventional cut and fill site grading has been assumed with the maximum depth of both the proposed cut and fill to be less than five feet. An excavation depth of 15 to 20 feet has been assumed for the USTs.

The Proposed Architectural Site Plan did not include a proposed Best Management Practice (BMP) infiltration system. Based on experience with other gas station type commercial projects, it was assumed that BMP infiltration devices would likely be located within the landscape areas such as at the north east and south east corners of the project.

The above assumptions were used as the basis for the exploration, testing, and analysis programs, and for the recommendations contained in this Report. If the anticipated foundation loading or other Site improvements vary significantly from those stated herein, then the recommendations should be reconfirmed prior to completing Project plans.

1.2 Purpose and Scope of Services

The purpose of **GCI**'s services was to explore and evaluate the subsurface soil conditions at the Site in order to provide preliminary geotechnical engineering conclusions and recommendations relative to the proposed development. **GCI**'s scope of services included a geotechnical Site reconnaissance, drilling and sampling of three exploratory borings (50-ft max) and three test pits, laboratory testing including corrosivity, geotechnical engineering analyses of the boring and test data, seismic design values, and a discussion of findings and recommendations in this Report.

This Report provides geotechnical recommendations for design and construction of the proposed development, including Site preparation and grading criteria, foundation design and lateral earth pressures, estimated settlements, expansive soils, soil corrosivity, and preliminary on-site pavement structural section design.

2.0 SITE DESCRIPTION

The subject Site is located on the north east corner of Cedar Avenue and Jurupa Avenue in Bloomington, California. It is approximately rectangular in shape and was vacant with chain link fencing.

At the time of this investigation, topography was near planar and flat with a very slight slope to the south as shown on **Figure 1**. With the exception of a mature tree located midway across the southern Site margin, the Site was essentially free of vegetation. A minor amount of trash was scattered across the Site.

The adjacent property to the north consisted of a trucking facility. A single family residence with a horse barn in use was adjacent along the east. Additional single family residences were east of the Site. Commercial properties were to the south and a concrete tilt-up building was under construction across Cedar Avenue to the west. Sidewalk and curb and gutter improvements were not complete along the streets.

3.0 FIELD INVESTIGATION

As part of the field investigation, a geotechnical field reconnaissance of the Site and surrounding areas was performed by the project engineer. The general configuration of the Site, Site topography and drainage characteristics, and surface conditions were noted and photographs were taken.

Subsurface exploration consisted of drilling and sampling three exploratory hollow-stem auger test borings to a maximum depth of 51.5 feet below the existing ground surface with a drill rig equipped with an automatic hammer for soil sampling. Three test pits, a maximum of 6 feet deep, were also excavated, sampled and logged for the project. The approximate locations of the exploratory borings and test pits are shown on **Figures 2 and 3**.

Bulk (disturbed) samples of the subsurface soils were obtained from spoil generated during drilling for classification and testing purposes. They represent mixtures of soils within the noted depth intervals.

Standard Penetration Test (SPT) samplers were utilized at 5-foot intervals to the full depth of Borings 1 and 2 to provide appropriate SPT data for geotechnical evaluations. California ring samplers were utilized in Borings 3 to provide relatively undisturbed ring test specimens for dry density determinations and other potential tests. The samplers were driven by an automatic lift 140-pound hammer falling 30 inches (ASTM D 1586). The raw number of blows required to drive the sampler 18 inches was noted in six-inch increments, or portion thereof, and recorded on the boring logs.

The materials and conditions exposed in the test pits were visually/manually classified (USCS) and evaluated by the project engineer. The soil samples were logged, labeled, and placed in sealed containers for transportation to our office and the laboratories for testing and further evaluation. The bore holes were backfilled with drill spoils and the test pits were loosely backfilled without compaction.

Logs of the exploratory borings and test pits are included in **Appendix A**. They represent **GCI**'s interpretation of the field logs prepared for each location by the project engineer, along with an interpretation of soil conditions between samples. While the noted stratification lines represent approximate boundaries between soil types, the actual transitions may be gradual.

4.0 LABORATORY TESTING

Included in the laboratory testing program were field moisture content determinations of all samples and field dry densities of all relatively undisturbed ring and SPT tube samples (ASTM D 2937 and ASTM D 2216). The results are included on the exploratory logs in **Appendix A**.

Sieve analysis and percent fines tests were conducted on selected samples for classification and correlation purposes.

A combined bulk sample was subjected to maximum dry density-optimum moisture content testing (ASTM D 1557) to evaluate the relative compaction and recompaction characteristics of the soils encountered.

The graphs of the laboratory test results are included in **Appendix B**.

A selected sample of soil was delivered to HDR for soil corrosivity testing including soluble sulfates (CTM 417) and chlorides (CTM 422), minimum resistivity (CTM 643), pH, and for various additional cations and anions.

The corrosivity test results are included in **Appendix C**.

5.0 SUBSURFACE SOIL CONDITIONS

Data from the exploratory borings and test pits indicate that the soil profile at the Site generally consists of loose and disturbed native alluvial fine to medium grained Silty Sand (SM) with varying amounts of subrounded gravel up to 2 inches in size and pockets of trashy fill to a depth of about 5 feet, underlain by undisturbed alluvial clean poorly graded Sand (SP) and well graded Sand with silt (SW-SM) both with varying amounts and sizes of sub-angular to subrounded gravel, additional Silty Sand (SM), and non-plastic Silt (ML) were encountered to the maximum depth of 51.5 feet attained.

Gravelly clast supported lenses with cobble up to about 6 inches in maximum dimension were encountered in the test pits.

The SPT and density data indicate that the soils are in place in medium dense to very dense states.

The materials encountered at the Site were cohesionless and non-cemented. Moderate sidewall caving was experienced in the test pits.

Compressible soil conditions or soils prone to hydro-consolidation when inundated with water and subjected to surcharge loading were not encountered below a depth of about 5 feet.

The materials encountered at the Site were granular non-plastic and considered to be non-expansive.

The soil corrosivity test results indicate that the soils tested exhibit a "negligible" anticipated exposure to sulfate attack of concrete.

Refusal was not experienced and no groundwater or bedrock was encountered.

For seismic design, the appropriate Site soil profile classification is D, "stiff soil", according to the California Building Code (CBC). The USGS Design Maps Beta (2015 NEHRP Provisions) seismic design values for the Site are included in **Appendix D.**

6.0 GROUNDWATER

No ground water or evidence of previous shallow groundwater (mottles) was encountered within any of the exploratory borings or test pits to the maximum depth of 51.5 feet attained. A review of groundwater information for the area indicated a depth to groundwater of over 200 feet (CDMG Special Report 113). Therefore, groundwater is not anticipated to be encountered during construction and liquefaction is not a hazard.

7.0 GEOLOGIC HAZARDS

Geologic hazards that may affect the proposed development include seismic shaking and other earthquake-related hazards.

The Site is not located within a currently delineated CGS Special Studies Zone (formerly known as Alquist-Priolo fault hazard zone). No known or suspected active faults were identified on or near the Site. Therefore, the potential for active fault rupture is considered to be very low.

Potential secondary seismic hazards related to ground shaking include liquefaction, water storage facility failure, ground deformation, areal subsidence, seismically-induced landsliding or slope failure, rockfalls, tsunamis, and seiches. Due to the inland location and elevation of the Site, hazards from tsunamis are not of concern. No water storage reservoirs are located in the immediate vicinity of the Site; therefore, there are no seiche hazards.

Inspection of the raw SPT blow count data indicate that the clean sands tested at the Site are sufficiently dense to preclude significant seismic settlement and liquefaction.

8.0 DISCUSSION AND CONCLUSIONS

Based upon the results of the field and laboratory investigations, it is the opinion of **GCI** that the proposed development is feasible from a geotechnical standpoint, provided the recommendations contained in this Report are followed and maintained during design, and construction.

A minimum mandatory removal and recompaction of the upper 5 feet of natural soils is recommended because of the loose and disturbed surface conditions observed and encountered at the Site including areas of old deleterious fills. The minimum mandatory removal should help to identify any buried structures and areas of deeper fill or disturbance associated with past land use and the previous demolition of previous structures at the Site. By virtue of the minimum mandatory removal and recompaction of the upper 5 feet of natural soils, a continuous compacted fill surface across the Site will result to provide uniform support for the proposed improvements.

If the Site is prepared and graded as recommended, conventional spread foundations may be used to support the proposed structures. The building pad areas will be overexcavated and recompacted to provide to provide at least 36 inches of properly compacted and tested fill beneath footings. Foundations for the proposed fuel canopy, pole signs, and UST's should be deep enough to bear in competent natural soils which shall be observed, approved, and documented by the geotechnical consultant.

If the site is properly prepared and the preliminary recommendations for foundation design and construction are followed, we would anticipate maximum settlements on the order of 3/4 inch. Differential settlement may be assumed to be fifty percent of the total settlement.

Based on the density and SPT data the clean sands tested at the Site are sufficiently dense to preclude significant seismic settlement and due to the deep depth to groundwater (over 200 feet) liquefaction is not a hazard at the Site.

9.0 RECOMMENDATIONS

The following recommendations and applicable portions of the CBC as well as any local ordinances should be followed during Site preparation, design, and construction of the proposed commercial development. An on-Site pre-grade meeting with the developer/owner, contractor, inspector, design civil, and the geotechnical consultant should occur prior to beginning site preparation.

9.1 Initial Site Preparation

All vegetation, undocumented fill, trash piles, pavements, abandoned underground utilities (if any), and other debris should be removed from the Site. Underground utilities (water, sewer, storm drain, electric, gas, cable, etc.) may be present within or adjacent to the proposed construction area. These utilities should be identified and relocated as required prior to performing excavations for any Site grading or foundation excavations. Depressions resulting from such removals should have debris and loose soils removed and filled with suitable soils placed as recommended below.

Any underground structures (e.g. seepage pits, cesspools, cellars, underground storage tanks), if any, should be removed in their entirety, including any brick lining and any liquids or sediment remaining at the bottom of the pits. The void resulting from removal of the seepage pits should be backfilled with suitable soils placed as recommended below. This may require ramping and/or laying back side slopes to an angle to allow safe entry of personnel and equipment. Alternatively, seepage pit excavations may be backfilled with a low-cement concrete slurry mix or "self-compacting" gravel to within 5 feet of proposed final grade or proposed footing elevations. The final 5 feet should consist of compacted engineered fill as described below.

In order to minimize potential settlement problems associated with structures supported on a non-uniform thickness of compacted fill, the geotechnical consultant should be consulted for grading recommendations relative to backfilling large and/or deep depressions resulting from such removals.

To provide more uniform bearing conditions for the proposed structure foundations and slab-on-grade construction, **GCI** recommends the following:

Undocumented fill should be carefully examined by the geotechnical consultant to determine if the material is suitable for re-use as engineered fill. Materials with significant organics, debris, clay or soluble sulfate contents should be deemed "unsuitable" by the geotechnical consultant and all such materials should be removed from the Site to prevent them from being incorporated in the fill.

A minimum mandatory **removal and recompaction** of the upper 5 feet of natural soils is recommended across the entire Site with exceptions for landscape and infiltration areas. The **bottom** of the removal excavation shall remain open for the geotechnical consultant to observe, approve, and document prior to any fill placement.

Once approved, the bottom of the removal excavation should be **scarified** (ripped) 6 inches, brought to between optimum moisture content and 3 percent above, and be compacted to at least 90 percent relative compaction (ASTM D 1557).

The excavated soils may be reused as compacted fill provided they are processed to remove any deleterious or **oversize** (6" max) materials.

Fill materials should be mixed and moisture treated to between optimum moisture content and 3 percent above and be uniformly compacted to at least 90% relative compaction (ASTM D 1557). To help compaction, fill should be spread in horizontal 8-inch thick loose lifts or less. Compaction testing shall be performed by a geotechnical consultant to verify compaction and moisture content.

Import soils should be equal to, or better than, the on-Site soils in strength, expansion, compressibility, and soil chemistry characteristics. In general, import material should be free of organic matter and deleterious substances, have 100% passing a two inch sieve, 60% to 100% passing a #4 sieve, no more than 20% passing a #200 sieve, an Expansion Index less than 20, a Liquid Limit less than 35 and a Plasticity Index less than 12. Import soils shall be observed, (tested if needed), and approved by the geotechnical consultant prior to their use.

Backfill around or adjacent to confined areas (i.e. interior utility trench excavations, etc.) may be performed with a lean sand/cement slurry (minimum two sacks of cement per cubic yard) or "flowable fill" material (a mixture of sand/cement/fly ash). The fluidity and lift placement thickness of any such material should be controlled in order to prevent "floating" of any "submerged" structure or may be performed using "self-compacting" gravel subject to approval by the geotechnical engineer.

Shrinkage due to excavation and compaction of the upper Site soils is estimated to be between approximately 10 to15 percent. In addition, subsidence on the order of 0.1 foot may occur due to densification of the underlying natural soils. Losses from Site clearing operations should also be considered when estimating earthwork quantities.

9.2 Excavations

Standard construction techniques should be sufficient for Site excavations. All excavations should be made in accordance with applicable regulations (including CAL/OSHA). The Site soil conditions are classified as Type "C" according to CAL/OSHA. Project safety is the responsibility of the contractor. **GCI** will not be responsible for project safety.

Cohesionless (non-cemented) sands with the tendency to cave or flow were encountered and should be considered with means of mitigation prior to excavation.

Open excavations may be cut vertically to a maximum depth of no more than four feet. Excavations extending between four and ten feet deep should be shored or sloped back from the base of the excavation to at least a 1.5 horizontal to one vertical (1.5H:1V) slope or flatter. If excavations dry out, sloughing may occur. No excavation should be made within a 1:1 line projected outward from the toe of any existing footing or structure.

During the time excavations are open, no heavy grading equipment or other surcharge loads should be allowed within a horizontal distance from the top of any slope equal to the depth of the excavation. Adequate measures should be taken to protect any structural foundations, pavements, or utilities adjacent to any excavations.

9.3 Utility Trenches

Standard construction techniques should be sufficient for utility trench excavations. The surface of utility trench backfill frequently settles even when backfill is placed under optimum conditions. Structural units or pavement placed over such backfill should be designed to accommodate such movements.

It is recommended that utility trench backfill should be mixed and moisture conditioned (brought to between optimum moisture content and three percent above) outside of the trench, and be uniformly compacted to at least 90% relative compaction (ASTM D 1557). In pavement areas, the top 6 inches of trench backfill and all base material shall be brought to near optimum moisture content and compacted to at least 95% relative compaction. To help obtain compaction, trench backfill should be placed in horizontal 6-inch loose lifts or less. Thinner lifts should be utilized with hand operated equipment. Jetting of utility trench backfill is not recommended.

Backfill operations should be observed and tested by the geotechnical consultant to verify conformance with these recommendations.

9.4 Foundation Preparation

Foundations for the proposed building structures shall be supported by a minimum 3-foot thickness of compacted soils prepared as recommended in this Report. In areas where the minimum mandatory removal and recompaction of the upper 5 feet of natural soil does not meet the minimum compacted fill matt thickness, the building pad areas shall be further subexcavated to provide at least 3 feet of compacted fill beneath footings to a lateral over-excavation distance of 5 feet beyond footing lines, where possible.

Foundations for the proposed canopy, pole signs, and UST's should be deep enough to bear in competent natural soils which **shall be observed, approved, and documented by the geotechnical consultant**. Cohesionless (non-cemented) sands with the tendency to cave or flow were encountered, as such, the need for mitigation measures should be anticipated for deep foundation excavations.

Excavations for foundations should be cleaned of all loose or unsuitable soils and debris prior to placement of concrete. Soil generated from the foundation excavations should not be placed below the floor slab unless properly moisture conditioned and compacted, and only after the area to receive fill has been properly prepared and approved.

9.5 Foundation Design

The proposed building structures may be safely supported by conventional shallow foundations, either continuous wall footings and/or individual spread footings bearing on a minimum 36-inch thickness of compacted soils prepared as recommended in this Report.

Foundations for the proposed fuel canopy, pole signs, and UST's should be deep enough to bear in competent observed and approved natural soils.

Footings should be at least a minimum of 12 inches wide and should bear at a minimum depth of at least 18 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) for dead plus sustained live loads. The allowable bearing capacity may be increased by 250 psf for each additional foot of width and by 500 psf for each additional foot of depth to a maximum safe soil bearing pressure of 4,000 psf for dead plus live loads. These values may be increased by 1/3 when transient loads (such as wind and seismic forces) are included.

For footings designed and constructed as recommended, we would anticipate a maximum settlement on the order of 3/4 inch. Differential settlement can be assumed to be approximately half the total settlement.

9.6 Slab-on-Grade Construction

Interior and exterior building concrete slab-on-grade construction should be supported by compacted soils prepared as recommended in this Report. The minimum thickness of concrete floor slab supported directly on the ground shall not be less than 6 inches.

It is recommended that all interior and exterior building concrete slab-on-grade construction be reinforced with at least #4 bars on 16-inch centers, each way. Reinforcement should be placed at mid-depth of the slab. The floor slabs should be quarter-sawn and isolated from stem wall foundations with a minimum 3/8-inch thick felt expansion joint.

Nominal eight-inch (8") thick (minimum) concrete slabs should be provided for traffic aprons, island slabs, and driveways and reinforced and isolated in the same manner as building floors. In addition, a grade beam at least 12 inches in width and at least 18 inches below the lowest adjacent soil grade should be provided across the traffic entrances.

Actual reinforcement requirements will be dependent on the governing building code, and requirements of the structural engineer.

A modulus of subgrade reaction ("k" value) of 350 psi/inch may be assumed for design of slab-on-grade provided the subgrade soils are prepared and compacted as recommended in this Report.

In areas of moisture sensitive floor coverings, an appropriate **vapor retarder** should be installed in order to minimize vapor transmission from the subgrade soil to the slab. The vapor retarder should be centered within a 4-inch thick sand layer. The vapor retarder should be evaluated for holes and/or punctures, and the edges overlapped and taped, prior to placement of sand. Any holes or punctures observed should be properly repaired. The 2 inches of sand cover should be lightly moistened and densified just prior to placing the concrete.

Relatively impervious floor coverings (i.e. vinyl, linoleum, etc.) that cover concrete slab-on-grade may block the passage of moisture vapor through the concrete slab, which could result in damage to the floor covering. It is suggested that after the concrete slab has sufficiently cured, the concrete slab surface be sealed with a commercial sealant prior to placing the floor covering. The compatibility and recommendations for placing of the concrete sealer, mastic, and floor covering should be verified by the floor covering manufacturer prior to sealing the concrete or placing of the floor covering. Cracks that develop in concrete slab-on-grade should be filled and sealed prior to placing floor coverings. Frequent control joints should be incorporated into the slab construction, particularly in the areas of re-entrant corners, to help control cracking.

9.7 Lateral Earth Pressures, Shoring and Retaining Walls

Resistance to lateral loading will be provided by passive earth pressure and friction acting along the foundation base. For footings bearing against properly compacted fill, a passive earth pressure of 350 psf per foot of depth may be utilized. A base friction coefficient of 0.35 may be used with dead loads. Base friction and passive resistance may be combined without reduction.

For preliminary retaining wall and shoring design, an "active" equivalent fluid pressure of 35 pcf may be assumed for cantilever (unrestrained) conditions and an "at-rest" lateral equivalent fluid pressure of 55 pcf may be assumed for braced conditions. These values should be verified prior to construction when the actual retained materials and conditions have been determined and are applicable only to properly drained level backfill with no additional surcharge loading.

Cohesionless (non-cemented) sands (C=0 psf, phi = 33 degrees) with the tendency to cave or flow should be considered in the shoring design for the UST excavation.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended.

9.8 Expansive Soil

Because the materials encountered at the Site were generally granular non-plastic and considered to be non-expansive, design and construction measures specifically to mitigate the effects of expansive soils are not anticipated at this time.

Additional evaluation of soils for expansion potential should be conducted by the geotechnical consultant during construction.

9.9 Preliminary Pavement Section

Based on the gradation results and visual/manual evaluation of the near-surface soils encountered at the Site, a preliminary asphalt concrete structural section design of 3 inches AC over 6 inches of aggregate base (R=78) may be considered. The actual pavement section should be determined during construction and based on R-value testing of the actual subgrade soil.

The pavement structural section design is predicated upon proper site preparation and compaction of utility trench as recommended with the upper 6 inches of subgrade soils and all base materials being compacted to at least 95 percent maximum dry density (ASTM D 1557).

9.10 Soil Corrosivity

A selected sample of near-surface soil was delivered to HDR and subjected to a suite of Caltrans corrosivity tests. The test results for soluble sulfate and chloride are in the "negligible" range according to the CBC, of which no special design considerations or specific concrete types are needed for corrosion protection or sulfate attack of concrete.

The soil corrosivity test results provided in **Appendix C** should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials with the soils.

Additional testing should be conducted during construction on the actual soils to be in contact with the item or material of concern, especially if fill is imported.

10.0 LIMITATIONS AND CONSTRAINTS

The conclusions and recommendations submitted in this Report relative to the proposed development are based, in part, upon the data obtained from Site observations during the field exploration operations, and past experience. The nature and extent of variations between the borings may not become evident until construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this Report.

In the event of any change in the assumed nature or design of the proposed Project as planned, the conclusions and recommendations contained in this Report shall not be considered valid unless the changes are reviewed and the conclusions of this Report modified or verified in writing. This Report is issued with the understanding that it is the responsibility of **Harry Sidhu**, or of his representatives, to insure that the information and recommendations contained in this Report are called to the attention of the architects and engineers for the Project and incorporated into the plan. It is also the responsibility of **Harry Sidhu**, or of his representatives, to insure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

As the geotechnical engineers for this Project, **GCI** strives to provide its services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This Report was prepared for the exclusive use of **Harry Sidhu** and his authorized agents.

It is recommended that **GCI** be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design specifications. If **GCI** is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations. The scope of current services for this Report did not include any environmental assessment or investigation for the presence or absence of wetlands, or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around the Site.

The statements contained in this Report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the conclusions of this Report may be invalidated, wholly or partially, by changes outside of **GCI**'s control, and should therefore be reviewed after one year.

11.0 ADDITIONAL SERVICES

This Report was based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check conformance with the recommendations of this Report. Maintaining **GCI** as the geotechnical engineering consultant from beginning to end of this Project will help provide continuity of services.

The recommended services include consultation as required during the final design stages of the Project; review of grading and/or building plans; observation and testing during Site preparation, grading, placement of engineered fill, and backfill of utility trenches; and consultation as required during construction.

12.0 CLOSURE

Geo-Cal, Inc. appreciates this opportunity to provide geotechnical engineering services. If there are any questions regarding the information contained in this Report, or if additional geotechnical engineering services are needed, please do not hesitate to contact this office.

Respectfully submitted,

Geo-Cal, Inc.

7 31 20

Henry Olivier, PG 5797
Vice President, Principal Geologist

REGISTERED PROFESSIONAL ENGINEER
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Project Engineer

FIGURES

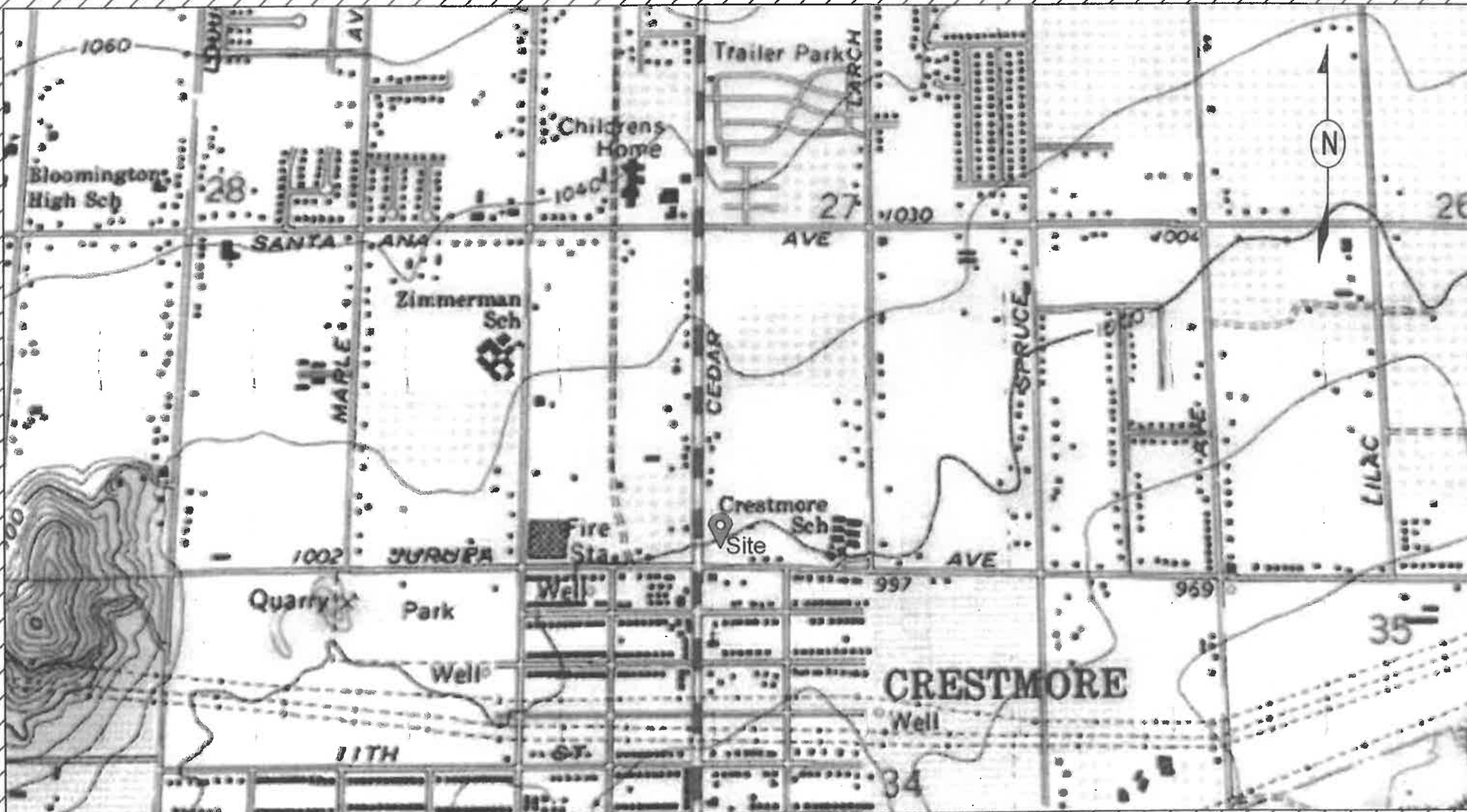

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Figure 1
Topographic Map

Proposed Commercial Development
11279 Cedar Avenue
Bloomington, CA

Legend:

Site Location

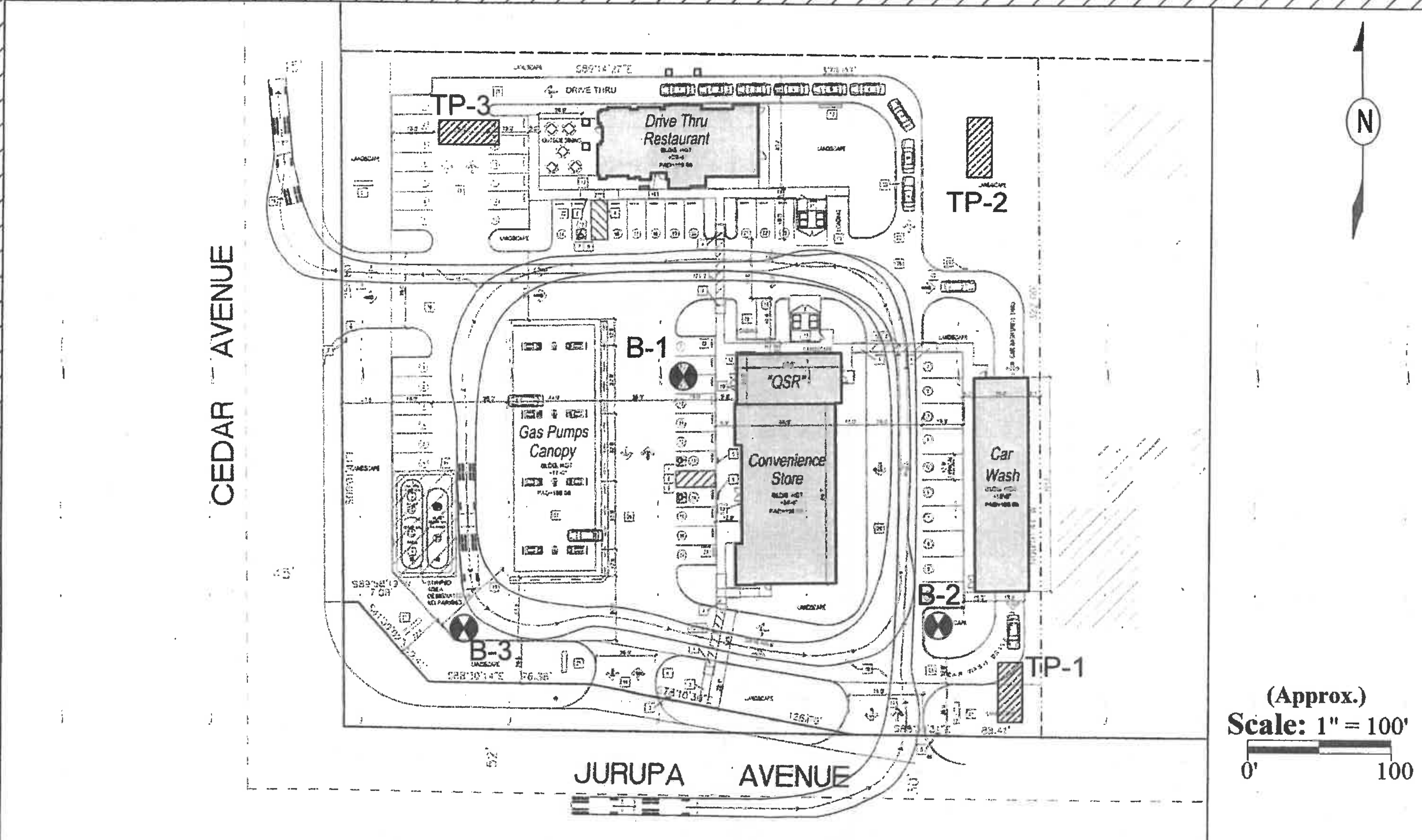

Figure 2
Site Plot Plan

Proposed Commercial Development
11279 Cedar Avenue
Bloomington, CA

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Legend:

TP-3 Test Pit

B-3 Boring Location

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Figure 3
Satellite Photo

Proposed Commercial Development
11279 Cedar Avenue
Bloomington, CA

Legend:
Site Perimeter
TP-3 Test Pit
B-3 Boring Location (Approx)

APPENDIX A:
EXPLORATORY LOGS

Geo-Cal, inc.
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4370 Hallmark Parkway, Suite 101
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LOG OF BORING 1

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date: 8-3-2018
Drilled By: 2R Drilling
Equipment: CME 75 Drill Rig
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 51.5'
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT. B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0	1A	B		0.5		MDC Cor		(SM) Silty Sand, fine, traces medium and coarse, with subangular to subrounded gravel to 2", mica, light gray
5	1-1	S	6 12 15	1.8		SA		(SP-SM) Sand, fine to medium, trace coarse, gray, medium dense, 5%fines
10	1-2	S	11 13 14	1.8				(SW) Gravelly Sand, fine to coarse, trace gravel, gray, medium dense
	1B (10'-15')	B		1.0				(SP) Sand, fine to medium, subrounded to rounded gravel to 2", brown
15	1-3	S	23 35 25	1.4				(SW) Gravelly Sand, fine to coarse, gravel to 2", gray, very dense
20	1-4	S	3 4 6	12.5				(SM) Silty Sand , fine, medium dense, mica, trace $CaCO_3$ strings, olive brown/orange
	1C (20'-25')	B		6.2		% fines		(SM) Silty Sand, fine, mica, brown, 27% fines
25	1-5	S	6 7 13	13.2	101	% fines tube		(ML) Sandy Silt, fine, mica,olive brown, medium dense 53% fines
30								

(Cont.)

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LOG OF BORING 1

Project:
Proposed Commercial Development
11279 Cedar Ave.
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Date: 8-3-2018
Drilled By: 2R Drilling
Equipment: CME 75 Drill Rig
Hole Size: 8"HSA
Logged By: Todd Wyland, RCE

Total Depth: 51.5'
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
30	1-6	S	8 10 16	11.1 4.9	106 108	tube tube		(SM) Silty Sand, fine, mica, olive, gray (SP) Sand, fine to medium, trace coarse, mica, gray, medium dense
35	1-7	S	10 13 13	18.9				(ML) Sandy Silt, fine, mica, olive brown, medium dense
40	1-8	S	15 29 27	3.4	112	tube		(SP-SM) Sand, fine to medium with silt, mica, light gray,dense
45	1-9	S	11 9 11	1.5 11.1 24.0				(SW) Sand, fine to coarse, trace subrounded gravel to 3/8", gray (SM) Silty Sand,fine, olive gray, mica (ML) Silt, oliver gray, medium dense
50	1-10	S	10 14 25	2.7 12.0				(SP) Sand, fine to medium, mica, gray (SM) Silty Sand, fine to medium, olive brown, dense

Total Depth: 51.5'
Shallow fill/Disturbed surface
No Groundwater
No Bedrock
No Refusal
Hole Backfilled with Drill Spoils

Geo-Cal, inc.
Environmental & Geotechnical Engineering
4370 Hallmark Parkway, Suite 101
San Bernardino, CA 92407
(909) 883-1143 FAX (909) 883-7557 email: info@geo-cal.com

LOG OF BORING 2

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date: 8-3-2018
Drilled By: 2R Drilling
Equipment: CME 75 Drill Rig
Hole Size: 8"HSA
Logged By: Todd Wyland, RCE

Total Depth: 16.5'
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0	2A 0-5'	B		0.8		MDC		(SM) Silty Sand, fine to medium with angular coarse and sub rounded to rounded gravel to 2", mica, gray
5	2-1	S	10 11 12	1.8	116	tube		(SW) Gravelly Sand, fine to coarse, subrounded gravel to 3/8", gray, medium dense
	2B 5'-10'	B		1.5				
10	2-2	S	4 6 6	4.9	103	tube % fines		(SM) Silty Sand, fine to medium, trace coarse rock fragments and gravel to 1/2", gray, medium dense, 23% fines
	2C 10'-15'	B		4.3				
15	2-3	S	4 6 8	4.6	106	tube		(SM) Silty Sand, fine, mica, gray brown, medium dense
20								Total Depth: 16.5' Shallow Fill / Disturbed Surface No Groundwater No Bedrock No Refusal Hole Backfilled with drill spoils
25								

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LOG OF BORING 3

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date: 8-3-2018
Drilled By: 2R Drilling
Equipment: CME 75 Drill Rig
Hole Size: 8"HSA
Logged By: Todd Wyland, RCE

Total Depth: 26.5'
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0	3A 0-5'	B		0.5		MDC		(SM) Silty Sand, fine, trace, subrounded gravel to 1", mica , gray
5	3-1	R	16 16 25	1.0	123.5			(SW) Gravelly Sand, fine to coarse, gravel to 1", gray, dense, 43% gravel
	3B 5'-10'	B		2.0				
10	3-2	R	38 50/5"				?	No Recovery, very dense
15	3-3	R	50/6"	9.9	Disturbed			(SM) Silty Sand, fine, traces medium and coarse, couple of 1" x 1.5" gravel, brown, very dense
20	3-4	R	21 20 20	14.2	113.6			(ML) Sandy Silt, fine, traces, coarse and gravel to 3/8", slightly plastic, medium dense
							?	
25	3-5	R	22 42 48	1.4	123.5			(SW) Gravelly Sand,fine to coarse, gravel and rock fragments to 2 1/4", 46% gravel,mica gray brown, very dense
26.5								

Total Depth: 26.5 ft
Shallow Fill /Disturbed Surface
No Groundwater
No Bedrock
No Refusal
Hole backfilled with drill spoils

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LOG OF TEST PIT 1

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date:	8-15-2018	Total Depth:	4 feet
Drilled By:	Morales Contracting	Groundwater Depth:	Not Encountered
Equipment:	Kubota Mini-Excavator		
Bucket Size:	18-Inches		
Logged By:	Todd Wyland, RCE		

T=6" SPT Tube

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count"/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0 – 1.5								(SM) Silty Sand, fine, traces medium and coarse, with organics (roots, dead weeds) and trash/debris (bricks), brown dry, loose, fill/disturbed native
3	TP 1-1	T		1.0	105			(SM) Silty Sand, fine-medium, mica, dry, light gray, Natural Alluvium
4	TP-1-2	T		0.3	112	SA		(SW-SM) Well Graded Sand, fine to coarse with silt, trace gravel to 1/2", 5% fines, gray brown

Infiltration Test IT-1

4' Total Depth
Fill/Disturbed Native to 1.5 ft
No Groundwater
No Bedrock
No Refusal
Slight Caving
Backfilled w/o Compaction

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San Bernardino, CA 92407
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LOG OF TEST PIT 2

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date: 8-15-2018
Drilled By: Morales Contracting
Equipment: Kubota Mini Excavator
Bucket Size: 18-Inches
Logged By: Todd Wyland, RCE

Total Depth: 6 feet
Groundwater Depth: Not Encountered

T=6" SPT Tube
C= Chunks of Clods
R=Ring Density Ave of 3 Hand Trimmed

** SA=Sieve Analysis

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0								**Top 7" organic(roots and dead weeds)**
								(SM) Silty Sand, fine to medium, mica, rootletts, dry, brown, loose, Disturbed Native
1								
	TP-2-1	T		0	101			**(SM) Silty Sand, fine, trace medium to coarse, mica,dry, brown, Natural Alluvium**
		C		1.0	100	R		
2								**1.5' thick lense of clast supoorted subangular to subrounded gravel and cobble up to 6", light brown**
3								
4	TP 2-2	T		0	105			**(SP-SM) Sand, fine with silt, trace medium, coarse and gravel to 1/2", mica, brown**
5								
6	TP 2-3	T		0.5	111	SA		**(SP) Poorly Graded sand, fine to coarse with gravel to 1/2", 4% fines, gray**
								Infiltration Test IT-2
								6' Total Depth **Disturbed to 1.5'** **No Refusal, No Groundwater, No Bedrock** **Slight Caring, Backfilled w/o Compacting**

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LOG OF TEST PIT 3

Project:
Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Date: 8-15-2018
Drilled By: Morales Contracting
Equipment: Kubota Mini Excavator
Bucket Size: 18-Inches
Logged By: Todd Wyland, RCE

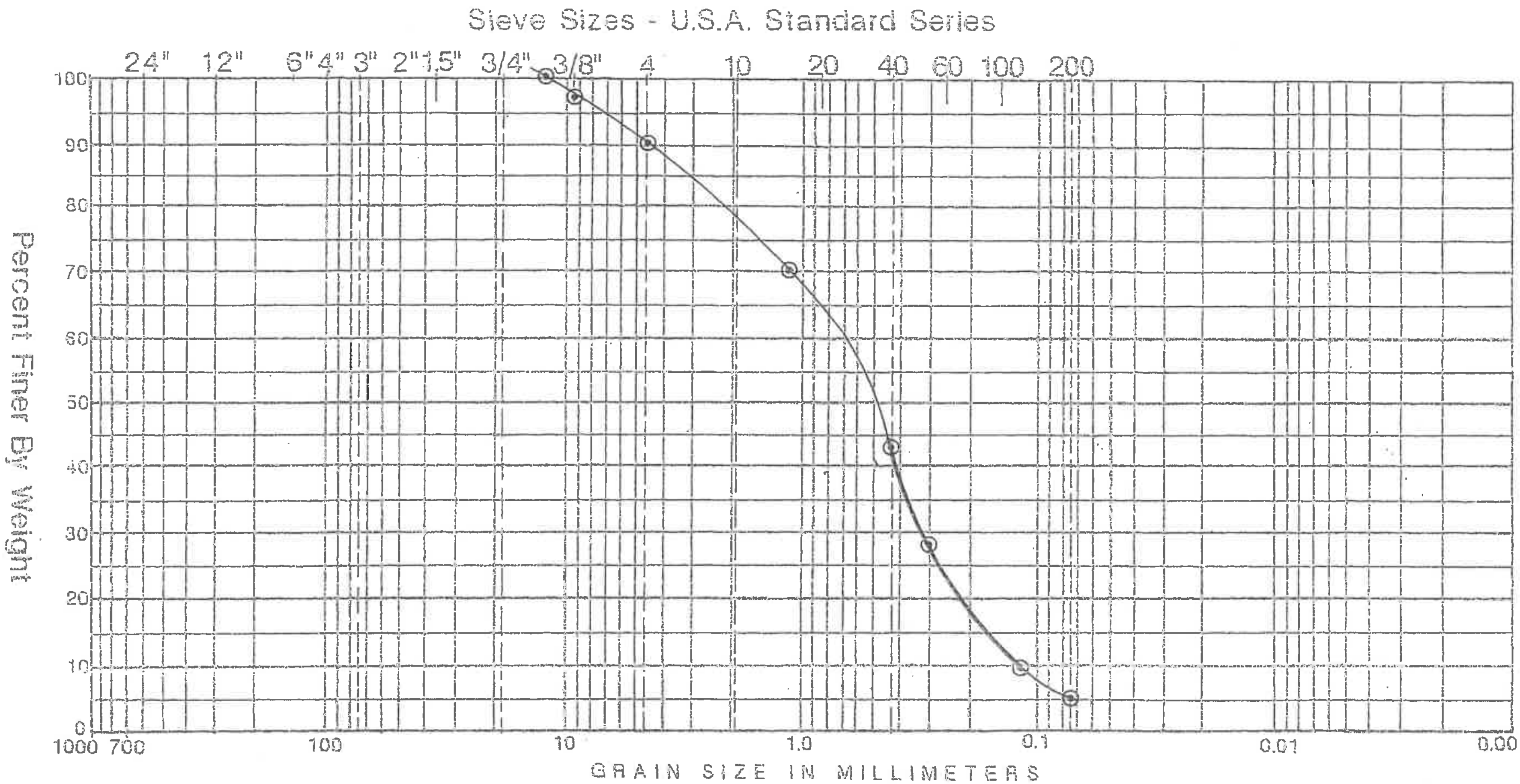
Total Depth: 5 feet
Groundwater Depth: Not Encountered

T=6" SPT Tube
C= Chunks of Clods
R=Ring Density Ave of 3 Hand Trimmed

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity
% fines= % passing No.200 sieve

Depth in Feet	Sample ID	Sample Type R=Ring S=SPT, B=Bulk	Blow Count*/6"	Moisture Content (%)	Dry Density (pcf)	Lab Tests **	Graphic	Description
0								(SM) Silty Sand, fine to medium, mica, dry , organics (roots/dead weeds), brown, disturbed
1	TP 3-1 1.5'	T		0	107			(SP-SM) Sand, fine with medium and silt, traces angular coarse and gravel to 3/8", dry mica, brown
2	TP 3-2 2'-2.5'	T		0	105			(SP-SM) Sand, fine to medium with silt, traces subangular to subrounded coarse and gravel to 3/8", mica, brown, trash (plastic bags), Fill
3	TP-3-3 2.5'-3'	T		0.5	105			(SP-SM) Sand, fine to medium with silt, traces subangular to subrounded coarse and gravel to 3/8", mica, gray brown Natural alluvium
4	TP 3-4 4'-4.5'	T		0	113			(SW) Sand, fine to coarse, with subangular to subrounded gravel to 3/8", mica, gray
5								5' Total Depth Fill to 2.5' No Groundwater No Bedrock No Refusal Moderate caving below 3' Backfilled w/o Compaction
6								

APPENDIX B:
LABORATORY GRAPHS



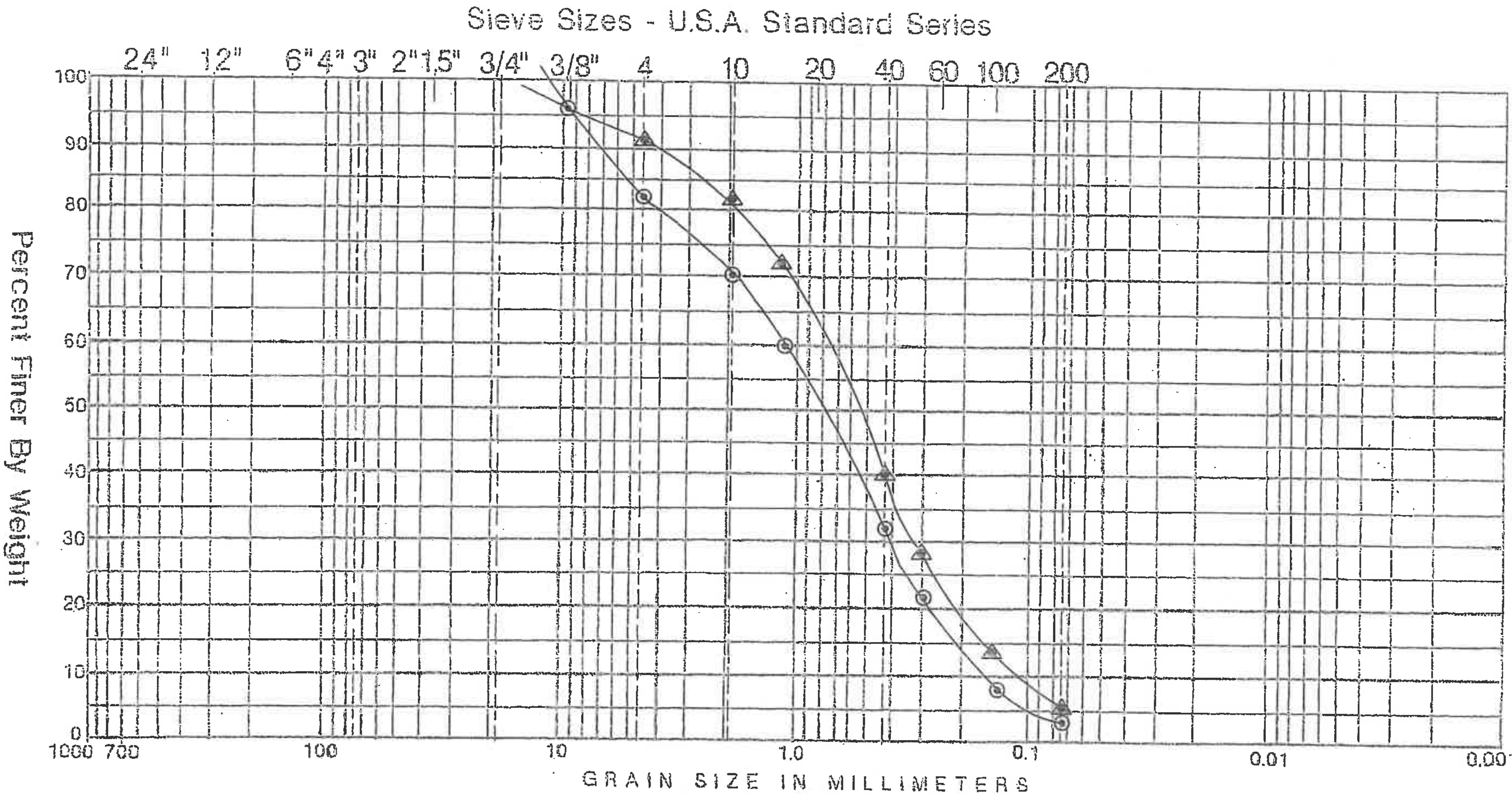
Cobbles & Boulders	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Boring	Depth	Classification
◉	1	5.5	(SP-SM) Poorly graded Sand, fine to coarse with silt and gravel to 1/2", 5% fines

Gradation Curves

Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

Geo-Cal, inc.
Environmental & Geotechnical Engineering



Cobbles & Boulders	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Boring	Depth	Classification
▲	TP-1	4'	(SW-SM) Well Graded Sand, fine to coarse, with silt, trace gravel to 1/2", 5% fines
⊙	TP-2	6'	(SP) Poorly Graded Sand, fine to coarse with gravel to 1/2",4% fines

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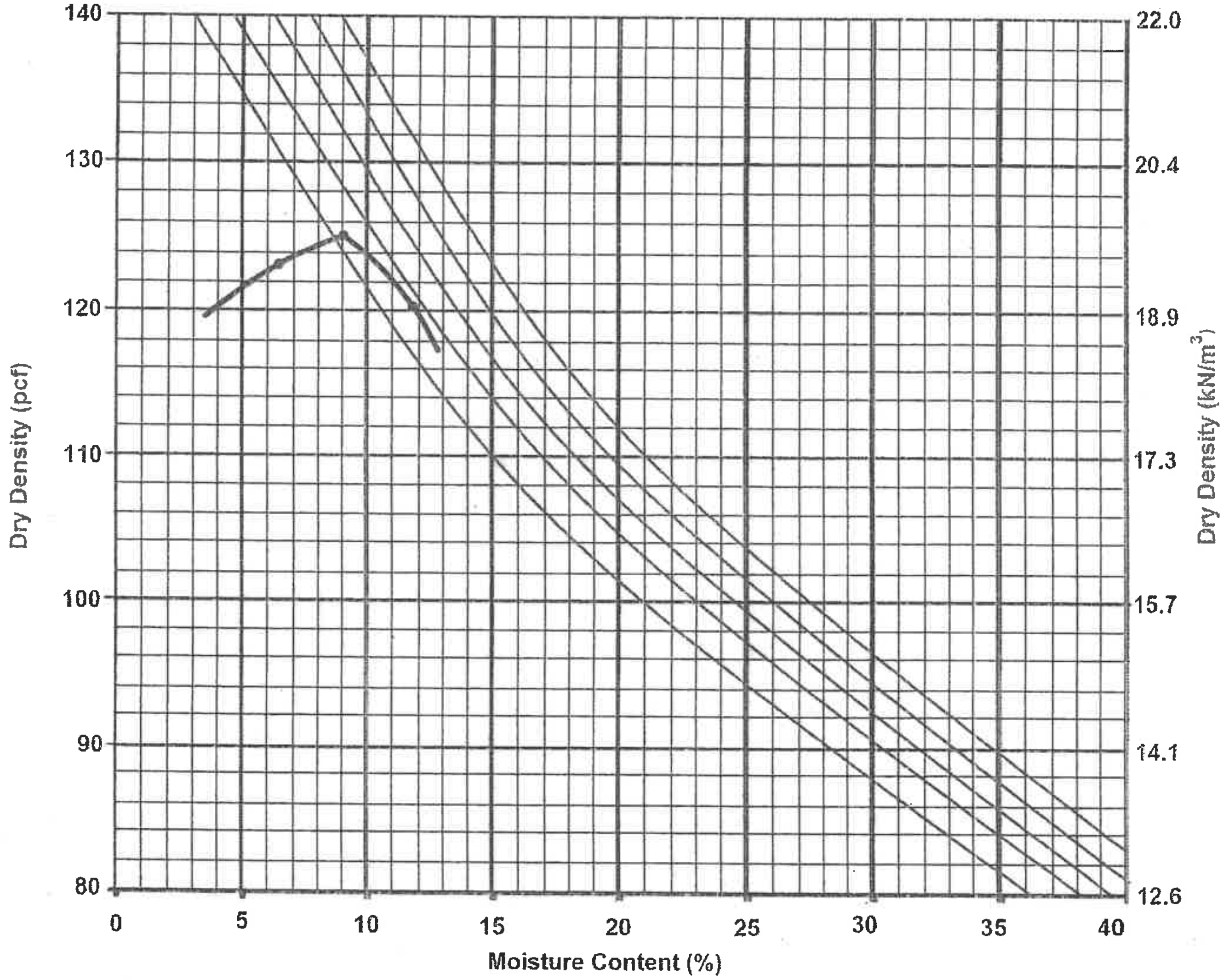
Gradation Curves

Proposed Commercial Development
11279 Cedar Ave.
Bloomington, CA 92316

MOISTURE DENSITY CURVE



ID Sample	Depth (ft)	Classification	γmax (pcf)	W_{opt} (%)
1A +2A+3A	0-5	(SM) Silty Sand, fine to medium with angular coarse and gravel to 2"	125	9.0

MOISTURE DENSITY CURVE (MDC) ASTM D 1557

Project: Proposed Commecial Development

Location: 11279 Cedar Ave, Bloomington, CA 92316

Geo-Cal, inc.
Environmental & Geotechnical Engineering

APPENDIX C:
HDR CORROSIVITY TEST RESULTS

TRANSMITTAL LETTER

DATE: August 10, 2018

ATTENTION: **Todd Wyland**

TO: Geo-Cal, Inc.
4370 Hallmark Parkway, #101
San Bernardino, CA 92407

SUBJECT: Laboratory Test Data
Bloomington
HDR Lab #18-0503LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Laboratory Services Manager

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

Table 1 - Laboratory Tests on Soil Samples

Geo-Cal, Inc.
Bloomington
HDR Lab #18-0503LAB
10-Aug-18

Sample ID			B-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	1,640,000
minimum		ohm-cm	11,600
pH			7.7
Electrical Conductivity		mS/cm	0.09
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	106
magnesium	Mg^{2+}	mg/kg	5.3
sodium	Na^{1+}	mg/kg	13
potassium	K^{1+}	mg/kg	20
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	247
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	7.1
phosphate	PO_4^{3-}	mg/kg	42
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	18
sulfide	S^{2-}	qual	na
Redox		mV	na

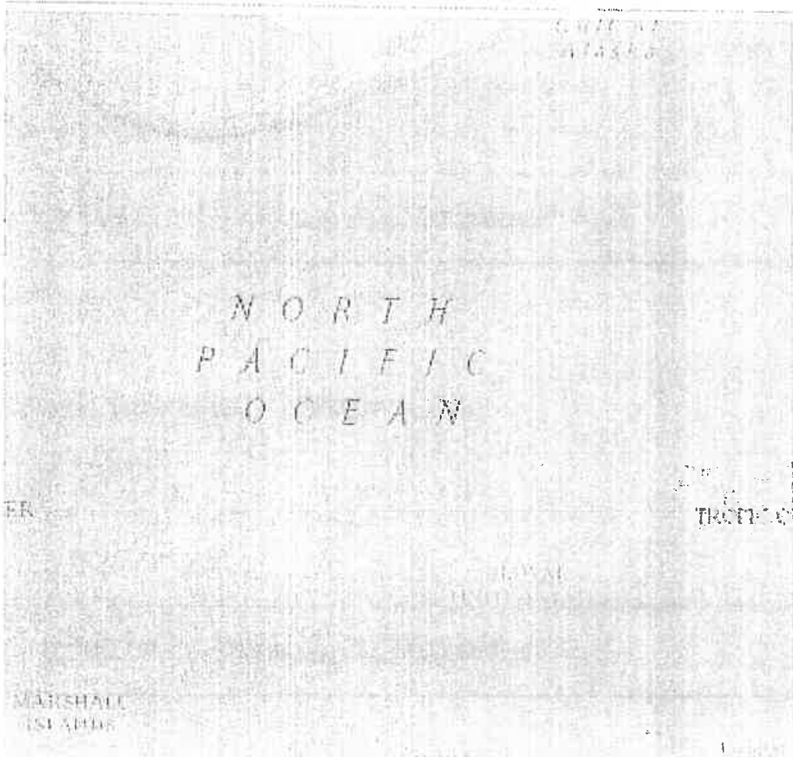
Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

APPENDIX D:
USGS SEISMIC DESIGN VALUES

Bloomington

Latitude = 34.049°N, Longitude = 117.396°W

Location



Reference Document

2015 NEHRP Provisions

Site Class

D (determined): Stiff Soil

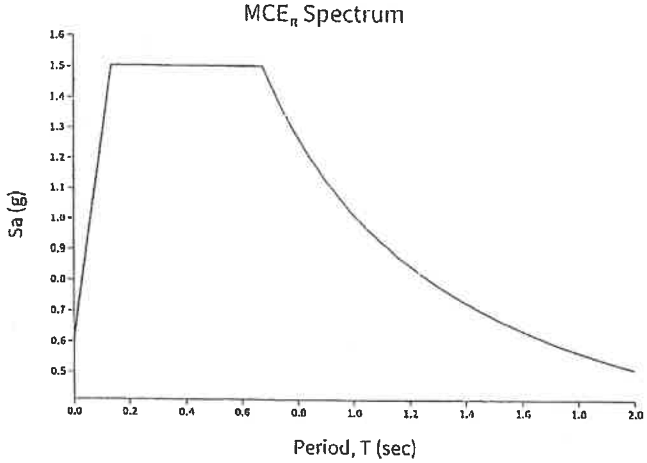
Risk Category

I or II or III

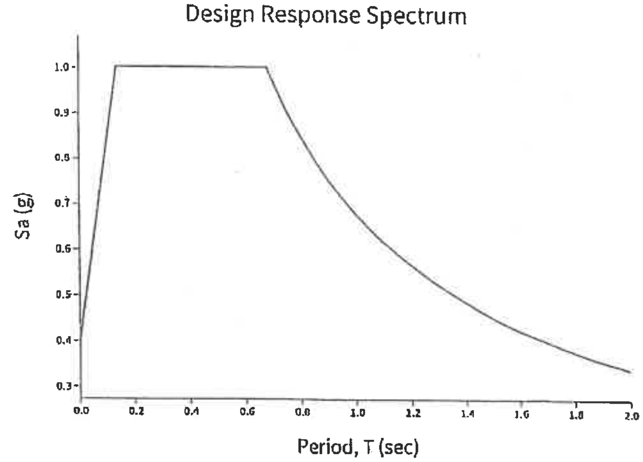
S_S =	1.503 g	S_{MS} =	1.503 g	S_{DS} =	1.002 g
S_1 =	0.600 g	S_{M1} =	1.020 g[1]	S_{D1} =	0.680 g[1]

[1] Since the Site Class is D and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

MCE$_R$ Spectrum



Design Response Spectrum



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

Risk-targeted Ground Motion (0.2 s)

$C_{RS}S_{SUH} = 0.931 \times 2.057 = 1.915$ g

Deterministic Ground Motion (0.2 s)

$S_{SD} = 1.503$ g

$S_S \equiv$ "Lesser of $C_{RS}S_{SUH}$ and S_{SD}" = 1.503 g

Risk-targeted Ground Motion (1.0 s)

$C_{R1}S_{1UH} = 0.905 \times 0.803 = 0.727$ g

Deterministic Ground Motion (1.0 s)

$S_{1D} = 0.600$ g

$S_1 \equiv$ "Lesser of $C_{R1}S_{1UH}$ and S_{1D}" = 0.600 g

Table 11.4-1: Site Coefficient F_a

Site Class	Spectral Reponse Acceleration Parameter at Short Period					
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 1.50$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.3	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.2	1.1	1.0	$\boxed{1.0}$
D (default)	1.6	1.4	1.2	1.2	1.2	1.2
E	2.4	1.7	1.3	1.2*	1.2*	1.2*
F	See Section 11.4.7					

For Site Class = D (determined) and $S_S = 1.503$ g, $F_a = 1.000$

Table 11.4-2: Site Coefficient F_v

	Spectral Response Acceleration Parameter at 1-Second Period					
Site Class	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.8	0.8	0.8	0.8	0.8	0.8
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.5	1.5	1.5	1.5	1.5	1.4
D (determined)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
D (default)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
E	4.2	3.3 [1]	2.8 [1]	2.4 [1]	2.2 [1]	2.0 [1]
F	See Section 11.4.7					

[1] For Site Class D or E and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

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

Note: Where Site Class B is selected, but site-specific velocity measurements are not made, the value of F_v shall be taken as 1.0 per Section 11.4.2.

For Site Class = D (determined) and S_1 = 0.600 g, F_v = 1.700

Site-adjusted MCE_R (0.2 s)

$$S_{MS} = F_a S_S = 1.000 \times 1.503 = 1.503 \text{ g}$$

Site-adjusted MCE_R (1.0 s)

$$S_{M1} = F_v S_1 = 1.700 \times 0.600 = 1.020 \text{ g}$$

Design Spectral Acceleration Parameters

Design Ground Motion (0.2 s)

$$S_{DS} = \tfrac{2}{3} S_{MS} = \tfrac{2}{3} \times 1.503 = 1.002 \text{ g}$$

Design Ground Motion (1.0 s)

$$S_{D1} = \tfrac{2}{3} S_{M1} = \tfrac{2}{3} \times 1.020 = 0.680 \text{ g}$$

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

Table 11.8-1: Site Coefficient for F_{PGA}

Site Class	Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration					
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA = 0.50	PGA ≥ 0.60
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.2	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.3	1.2	1.1	$\boxed{1.1}$
D (default)	1.6	1.4	1.3	1.2	1.2	1.2
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.7					

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

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of F_{pga} shall not be less than 1.2.

For Site Class = D (determined) and PGA = 0.639 g, F_{PGA} = 1.100

Mapped MCE_G

PGA = 0.639 g

Site-adjusted MCE_G

$PGA_M = F_{PGA}PGA = 1.100 \times 0.639 = 0.703$ g