GEOTECHNICAL ENGINEERING REPORT

PROPOSED FUELING STATION AND MARKET

CAJON BLVD, GLEN HELEN, CA

APN: 0349-182-11-0000

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1.0 INTRODUCTION

Geo-Cal, Inc. (**GCI**) has prepared this Geotechnical Engineering Report for a new gas station proposed to be located on the southwest side of Cajon Blvd opposite Park Ave in the Glen Helen area of San Bernardino County, California (**Figure 1**). The Site APN is 0349-182-11-0000. The Site coordinates are Latitude 34.219, Longitude -117.402.

1.1 Project Considerations

Based on information provided to this office, it is our understanding that the Project will include construction of a convenience store and a QSR with a drive thru, two fueling stations, one with 6 MPD's for truck fueling and a second with 6 MPD's for fueling regular vehicles, two steel canopies, three aboveground storage tanks (20,000-gallon capacity, each) 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. Drilled pier type foundations are anticipated for the fuel canopy and pole sign(s).

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. Because aboveground storage tanks are proposed, the 15 to 20 feet deep excavation normally assumed for underground storage tanks was not anticipated.

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 five test borings (35 ft max), 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. Percolation testing for BMP infiltration was conducted in two of the borings.

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, preliminary on-site pavement structural section design, and BMP infiltration.

2.0 SITE DESCRIPTION

The subject Site consists of a 1.4 acre vacant lot, approximately trapezoidal in shape, as shown on the Site Satellite Photo attached as **Figure 2**. At the time of this investigation, topography of the Site was near planar with a slight slope to the south east. The surface of the Site was disturbed and irregular with cobbles and boulders. The elevation near the center of the Site was about 2,040 feet. Vegetation included a slight growth of weeds across the Site and 3 trees along the south east property line. A minor amount of scattered trash and debris were seen at the Site.

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 five exploratory hollow-stem auger test borings to a maximum depth of 35 feet below the existing ground surface with a Mobil B-61 drill rig equipped with an automatic hammer for soil sampling. The approximate locations of the test borings are shown on **Figure 2**.

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 the borings to provide appropriate SPT data for geotechnical evaluations. 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 encountered were visually/manually classified (USCS) and evaluated by the project engineer. The soil samples were logged and placed in labeled sealed containers for transportation to the laboratories for testing and further evaluation.

The bore holes were backfilled with drill spoils, except for the percolation test borings, where the gravel packed pipe was left in place.

Logs of the exploratory borings 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 (ASTM D 2937). The results are included on the boring logs in **Appendix A**.

Sieve analyses were conducted on selected samples for classification purposes. A maximum dry density-optimum moisture content test (ASTM D 1557) was performed on a selected bulk sample to evaluate the compaction characteristics of the upper 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 indicate that the soil profile at the Site generally consists of alluvial wash deposits to the maximum depth of 35 feet attained with a disturbed surface. The soils encountered were generally classified as fine to coarse grained poorly graded Sand (SP) with variable gravel/rock fragments up to 1.25” with some poorly graded Sand with silt (SP-SM). **Based on observations and the drill rig response to drilling, the potential for cobbles and boulders exist throughout the Site.**

The SPT data at the 5-foot sample interval indicate that the soils encountered were generally in place in a “loose” state. At the 10-foot sample interval “medium dense” conditions were indicated becoming “dense” and “very dense” with depth.

Compressible soil conditions or soils prone to hydro-consolidation when inundated with water and subjected to surcharge loading were not encountered below the 5-foot sample interval.

All the materials encountered at the Site were granular non-plastic and non-expansive.

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

Refusal to further drilling was experienced (bouncing on a boulder) at 9 feet in PB-2.

Bedrock was not encountered.

No ground water or evidence of previous shallow groundwater (mottles) was encountered within any of the exploratory borings to the maximum depth of 35 feet attained.

For seismic design, the appropriate Site soil profile classification is D, "stiff soil", according to the California Building Code (CBC). The ASCE 7-16 seismic design values for the Site are included in **Appendix D.**

6.0 GEOLOGY AND GROUNDWATER

As shown on the attached Geologic Map (**Figure 3**), the site is underlain by Very young wash deposits (Q_{w2}) explained to be unconsolidated, mixed sand, gravel, pebble, cobble, and boulder deposits that form slightly elevated, low terraces within, or along active margins of, active washes (USGS Open-File Report 2006-1217).

No groundwater or evidence of previous shallow groundwater (mottles) was encountered within any of the exploratory borings to the maximum depth of 35 feet attained. Well data provided from Devore Water Company Well No.4, located about 300 feet southwest of the Site, indicated a depth to groundwater of about 150 feet bgs on November 9, 2021. Groundwater is not anticipated to rise within 65 feet of the ground surface.

7.0 LIQUEFACTION AND OTHER GEOLOGIC HAZARDS

Geologic hazards that may affect the proposed development include seismic shaking and other earthquake-related hazards. A Geologic Hazard Map for the area is attached as **Figure 4**.

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 landslides or slope failure, rockfalls, tsunamis, and seiches.

Due to the inland location of the Site, hazards from tsunamis are not of concern. No water storage reservoirs or facilities are located near the Site; therefore, hazards from seiches or storage facility failure are not present.

Because there are no slopes at or near the Site and because there are no slopes proposed, there are no slope stability related hazards.

The Site is not located within a mapped liquefaction hazard zone. Well data provided from Devore Water Company indicated a depth to groundwater of about 150 feet bgs. Therefore, the potential for liquefaction is considered to be low.

Because the Site is located near major active faults and underlain by very young wash deposits, **the potential for seismic settlement was evaluated**.

The differential seismic settlement potential of the improved Site would be greatly minimized by the recommended removal and recompaction of at least the upper five feet of existing soils across essentially the entire Site.

Based on inspection of the SPT data, below about 15 feet the soils were indicated to be sufficiently dense to preclude significant seismic settlements.

Therefore, a thickness of about 5 to 10 feet of medium dense competent natural wash deposits would remain beneath the compacted fill with a limited potential for seismic settlements. Because the amount of settlement tends to be proportional to the thickness, **it appears reasonable to assume that differential seismic settlements up to one inch across 30 feet may occur over the lifetime of the project and the proposed structures should be designed accordingly.**

Because settlement of sand due to foundation loading occurs almost immediately, with the majority occurring during construction, it is our opinion that the estimated static settlements do not need to be combined with the seismic.

8.0 SITE INFILTRATION

Two boring percolation tests were performed in order to provide infiltration rate recommendations for storm water BMP design. The percolation test borings were drilled with 8-inch diameter hollow stem augers to anticipated BMP depths of 5 and 9 feet bgs. The holes were fitted with 3-inch diameter perforated pipe, gravel packed to the surface and filled with water to presoak.

Based on the measured water drop over two 25-minute time intervals, the sandy soil criteria was met and the testing proceeded with water drop measurements at 10-minute intervals for an additional hour.

Both percolation test borings indicated relatively fast infiltration test rates (PB-1 at 5.5 ft was 13.0 in/hr and PB-2 at 9 ft was 12.2 in/hr).

By applying a factor of safety of 3, the design infiltration rates are 4.34 in/hr and 4.07 in/hr for PB-1 and PB-2, respectively.

The percolation test data and calculated results are included in **Appendix E**.

9.0 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 during design and construction.

9.1 Initial Site Preparation

Because of the disturbed surface conditions observed at the Site and the loose conditions encountered at the 5-ft sample interval of our borings**, a minimum mandatory removal and recompaction of the upper 5 feet of existing soils should be performed across the entire Site** with possible exceptions for shallow infiltration areas. The minimum mandatory removal should help to identify any buried structures and areas of deeper fill or disturbance associated with past land use. By virtue of the minimum mandatory removal and recompaction of the upper 5 feet of existing soils, a continuous compacted fill surface across the Site will provide uniform support for the proposed improvements and excavations.

Based on the considerable amount of oversize material (cobble over 6-inches and boulders) observed at the surface, it follows that significant quantities of oversize material can be encountered in the Site excavations, and, as such, should be considered throughout planning, design, and construction.

9.2 Foundations and Settlement

If the Site is prepared and graded as recommended, conventional spread foundations may be utilized in conjunction with a compacted fill matt 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 observed and approved by the geotechnical consultant. However, excavation difficulties, such as caving, should be anticipated due to the cohesionless nature of the natural sand and gravel wash deposits that likely contain larger clasts (cobbles and boulders).

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 1/2 inch. Differential settlement may be assumed to be fifty percent of the total settlement.

Based on the recommended minimum removal and recompaction of at least the upper five feet of existing soils across essentially the entire Site, as well as the complete removal and recompaction of any deeper loose soils encountered and because the SPT data of the materials tested below about 15 feet are sufficiently dense to preclude significant seismic settlements, **differential seismic settlements are anticipated to be within acceptable tolerable limits (less than 1 inch across 30 feet)**.

9.3 Infiltration

Based upon the materials and conditions encountered at the Site and the results of the two boring percolation tests, it is the opinion of **GCI** that infiltration BMPs are geotechnically feasible for the Site with a recommended design infiltration rate 4.0 in/hr.

10.0 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.

10.1 Site Preparation and Grading

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, such as septic tanks or seepage pits, 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 a lean 2 sack concrete slurry mix to within 5 feet of proposed final grade or proposed footing elevations. The final 5 feet should consist of compacted fill as described below.

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 existing soils is recommended across the entire Site with exceptions for shallow infiltration areas.

Prior to any fill placement, the geotechnical consultant should be notified to observe and approve the open bottom of the removal excavation.

Once approved, the bottom should be **scarified** (ripped) 6 inches, brought to near optimum moisture content, 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.

Based on the considerable amount of oversize material (cobble over 6-inches and boulders) observed at the surface, it follows that significant quantities of oversize material can be encountered in the Site excavations, and, as such, should be considered during planning, design, and construction.

Fill materials should be mixed and moisture treated to near optimum moisture content 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. Observation and compaction testing shall be performed by the 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 may be performed using "self-compacting" pea gravel subject to approval by the geotechnical consultant.

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 the removal of oversize cobbles (6" max) and boulders**, and Site clearing operations should also be considered when estimating earthwork quantities.

10.2 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 of the existing compacted fill 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 in compacted fill should be shored or sloped back from the base of the excavation to at least a 1.5 horizontal to 1 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.

10.3 Utility Trenches

Standard construction techniques should be sufficient for utility trench excavations made in the compacted fill associated with the recommended minimum mandatory removal and recompaction of the top 5 feet of existing soil.

Deeper trenches, made in the underlying natural wash deposits which were found to be **cohesionless (non-cemented) sands and gravels (with possible cobbles and boulders) that tend to cave, run, or flow and, as such, should be considered with means of mitigation prior to excavation.**

It is recommended that utility trench backfill be mixed and moisture conditioned to near optimum moisture content, and be uniformly compacted to at least **90%**relative compaction (ASTM D1557).

In AC 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 compaction tested by the geotechnical consultant to verify conformance with these recommendations.

10.4 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 existing 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 undisturbed natural wash deposits observed and approved by the geotechnical consultant. Cohesionless (non-cemented) sands with the tendency to cave or flow were encountered and, as such, the need for mitigation measures should be anticipated for deep foundation excavations. Excavation/drilling difficulties associated with possible cobbles and boulders should also be considered.

Excavations for foundations should be cleaned of all loose soils and debris prior to placement of concrete.

All foundation excavations shall be observed and approved in writing by the geotechnical consultant prior to steel placement.

10.5 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 of 36 inches of properly compacted and tested fill.

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.

Resistance to lateral loading will be provided by passive earth pressure and friction acting along the foundation base. For foundations bearing against 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 footings designed and constructed as recommended, we would anticipate a maximum static settlement on the order of 1/2 inch. Differential settlement can be assumed to be approximately half the total settlement.

10.6 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.

10.7 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 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 35pcf 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 materials and conditions have been determined and are applicable only to properly drained level backfill with no additional surcharge loading.

Because the cohesionless sands (with gravel, cobble and possible boulders) encountered tend to cave, the need for shoring should be considered for the UST excavation. Shoring may be designed assuming no cohesion (C=0 psf) and a friction angle of 33 degrees to model the shear strength of the natural wash deposits.

Surcharge may be treated as additional height of backfill by assuming 1 additional foot for each 125 psf of areal surcharge.

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

10.8 Expansive Soil

Because all the materials encountered at the Site were 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.

10.9 Preliminary AC Pavement Sections

For preliminary planning purposes, the following asphalt concrete (AC) structural section designs were calculated based on an assumed R-value of 50 and assumed Traffic Indexes (T.I.'s) of 4 for automobile areas and 6.5 for the truck areas:

Auto Areas: 0.25' (3") AC over 0.33' (4") AB

Truck Areas: 0.33' (4") AC over 0.50' (6") AB

The aggregate base (AB) should have an R-value of at least 78. The AB and the top 6 inches of soil subgrade should be compacted to at least 95% relative compaction (ASTM D 1557).

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

The actual pavement sections should be determined during construction and based on R-value testing of the actual subgrade soil.

10.10 Soil Corrosivity

A selected sample of near-surface soil was delivered to HDR for a suite of Caltrans soil corrosivity tests including soluble sulfates and chlorides, and resistivity.

The **soluble sulfate** results (5.1 ppm) indicate a "**negligible**" anticipated exposure to sulfate attack of concrete of which no special design or construction measures, such as special cement types or water to cement ratios, are needed.

The results of the **soluble chlorides** (8.5 ppm) are categorized as "**not corrosive**" to ferrous materials.

The **minimum resistivity** results (18,800 ohm-cm) are categorized as "**not corrosive**" to normal grade steel.

A **pH** of 7.8 "**not corrosive**" was determined for the soil tested.

The soil corrosivity test results are provided in **Appendix C** and 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 can be conducted during construction on the actual soils to be in contact with the item or material of concern, especially if fill is imported.

10.11 Infiltration

Based on the boring percolation infiltration test rates and using a Factor of Safety of 3, a design infiltration rate of 4.0 in/hr is recommended.

The final BMP design should be reviewed by the geotechnical consultant.

The geotechnical consultant should be notified for observation of the open BMP excavation in order to verify soil conditions and provide additional recommendations, if needed.

Foundations should be set back at least 8 feet from the BMP limits.

Equipment should be kept away from shallow infiltration BMP areas during grading.

11.0 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 **Henry Olivier**, 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 **Henry Olivier**, 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 **Henry Olivier** 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.

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.

13.0 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,

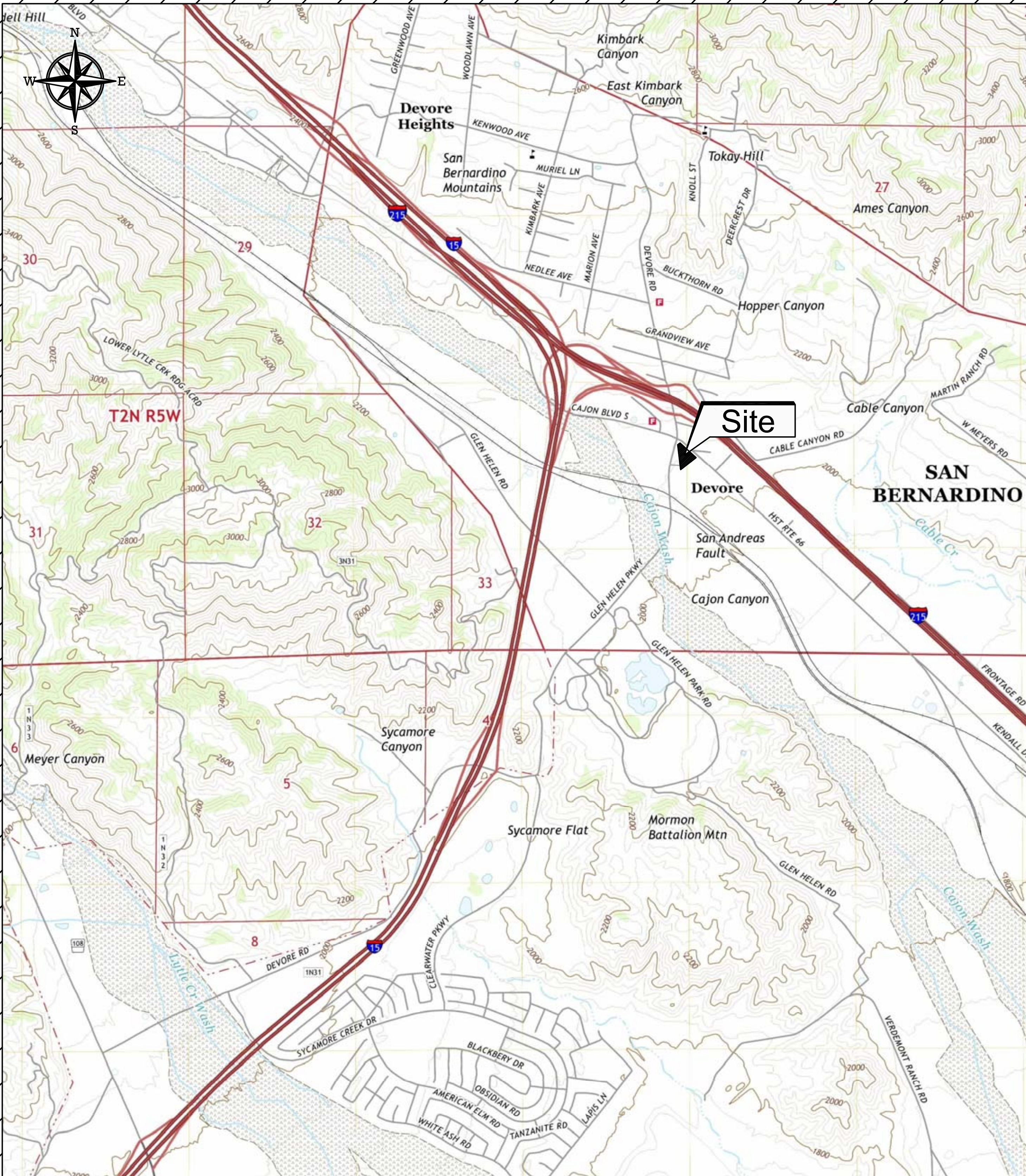
Geo-Cal, Inc.

REGISTERED PROFESSIONAL ENGINEER
TODD R. WYLAND
No. 60618
Exp. 12-31-22
CIVIL
STATE OF CALIFORNIA

1-30-22

Todd R. Wyland, RCE 60618
Project Engineer

FIGURES



USGS 7.5 minute Topographic Map Devore Quadrangle

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Environmental & Geotechnical Engineering
4370 Hallmark Prkwy. Ste #101
San Bernardino CA 92407

Figure 1
Site Location Map

APN:0349-182-11
Glen Helen, CA

LEGEND:

Site

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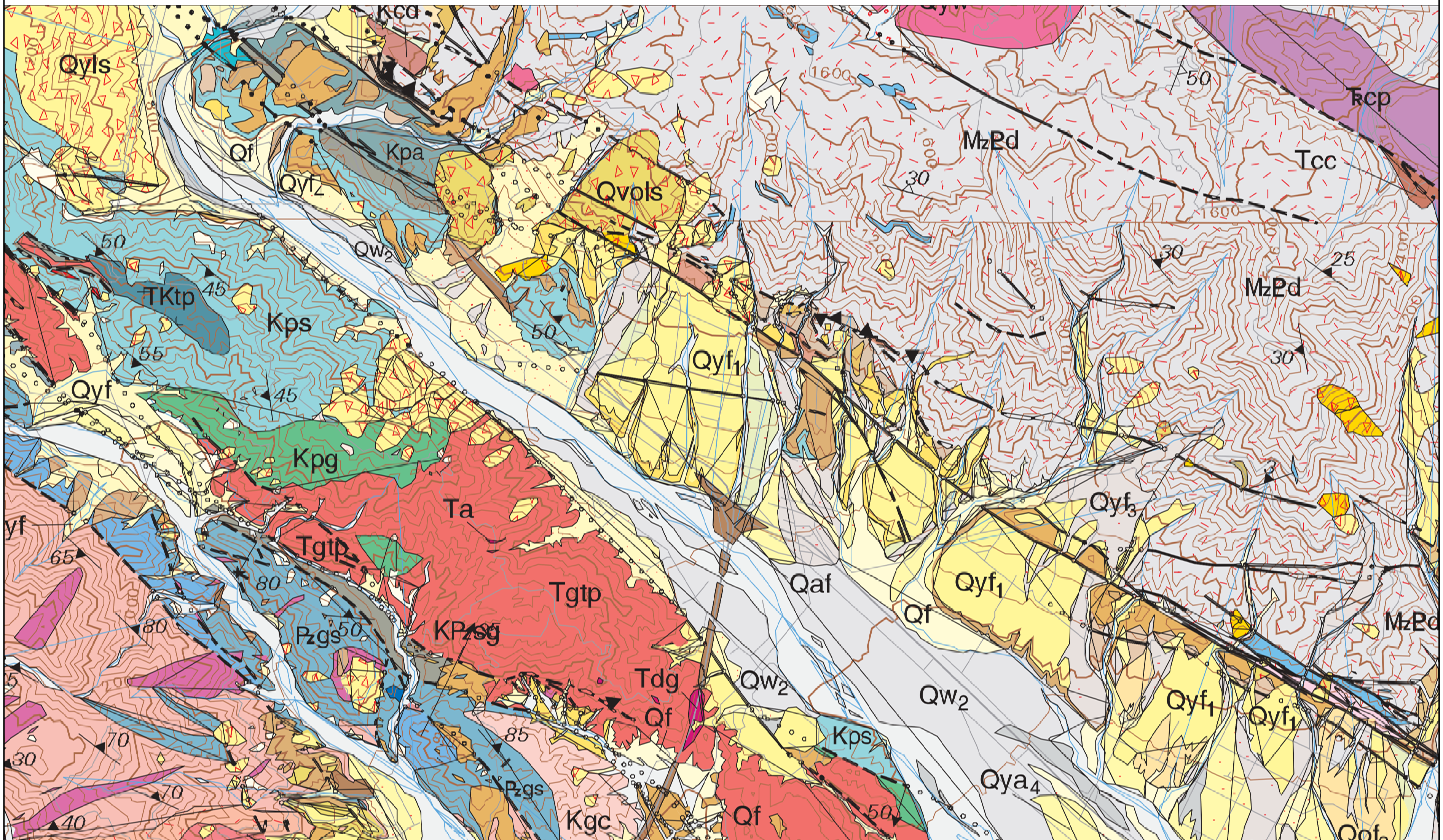
San Bernardino CA 92407

Figure 2
Site Plan Showing Boring Locations

APN:0349-182-11
Glen Helen, CA

Legend: (Locations are approx.)

PB-1 Percolation Boring Location
B-3 Exploratory Location



Reference: USGS Open-File Report 2006-1217

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Figure 3
Geologic Map

APN:0349-182-11
Glen Helen, CA

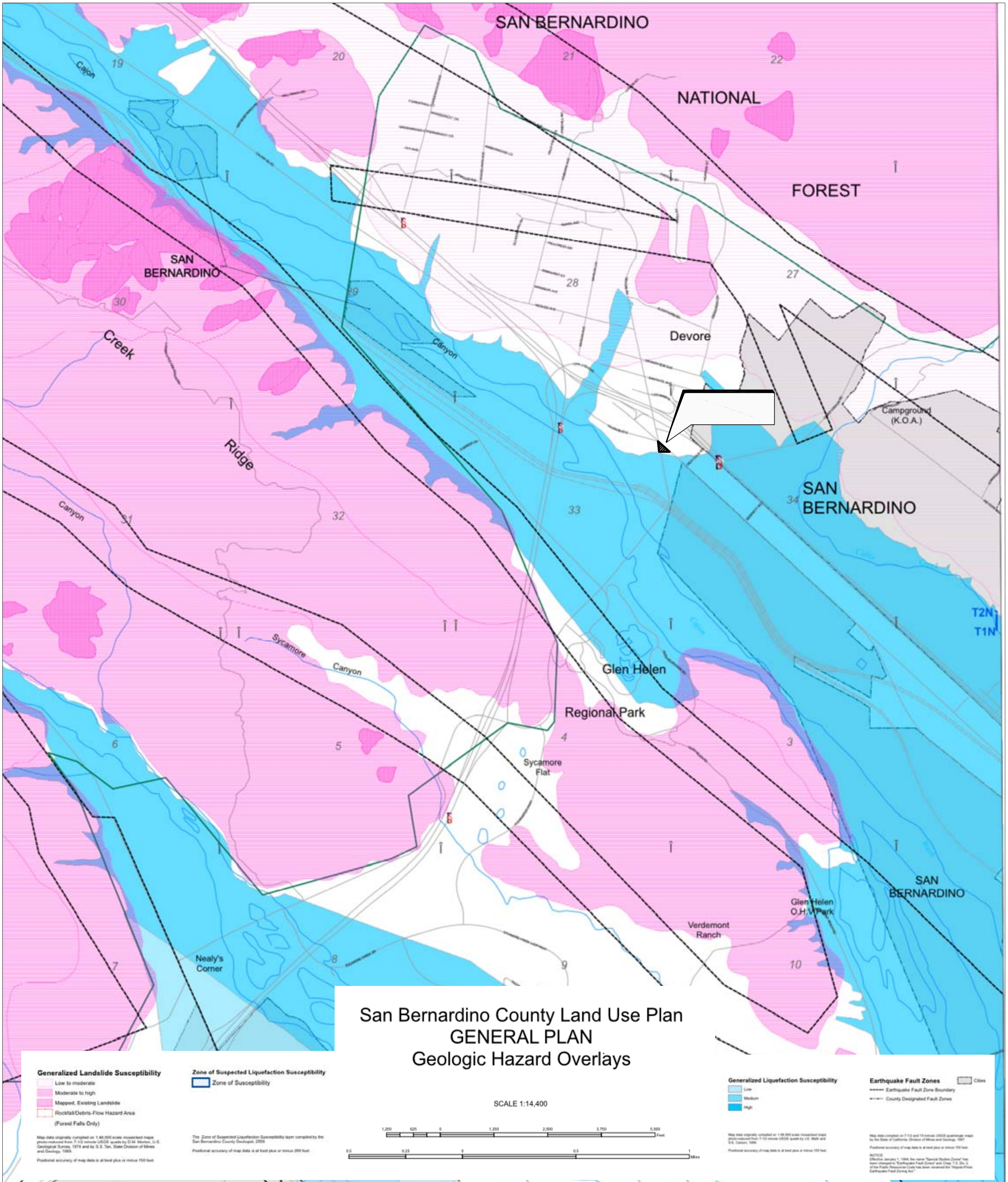

Figure 3
Area Geologic Hazard Map

APN:0349-182-11
Glen Helen, CA

LEGEND:
Site

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San Bernardino CA 92407

APPENDIX A

EXPLORATORY LOGS
(6 PAGES)

Geo-Cal, inc.
Environmental & Geotechnical Engineering
4370 Hallmark Parkway, Suite 101
San Bernardino, CA 92407
(909) 880-1146 FAX (909) 880-1557 email: info@geo-cal.com

LOG OF BORING B-1

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 16.5 ft
Groundwater Depth: Not Encountered

*Automatic Hammer
140 lbs 30-Inch Drop

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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								Disturbed Native: (SP-SM)Sand, fine to medium, trace coarse, with Silt and gravel to 1", brown
	1A (0'-5')	B		1.6				
								Very young wash deposits-Qw2: (SP) Sand, fine to medium, trace coarse with gravel to 1.5", gray brown, medium dense.
5	1-1	S	3 7 19	1.5				
	1B (5'-10')			1.3				
10	1-2	S	8 11 8	2.2				Same as Above
15	1-3	S	18 25 16	1.4				(SP) Sand, fine to coarse with gravel/rock fragments to 1.25', gray, dense
								End of Boring Total Depth 16.5'
20								No Refusal No Bedrock No Groundwater Disturbed native to about 3ft.
								Borehole backfilled with drill spoils
25								
30								

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LOG OF BORING B-2

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 35.25 ft
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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								Disturbed Native: (SP)Sand, fine to coarse, with gravel to 1/2", brown.
	2A (0'-5')	B		1.6				
5	2-1	S	5 4 4	1.8				Very young wash deposits-Qw2: (SP-SM) Sand, fine to medium with coarse, silt and gravel to 1.25', brown, loose
10	2-2	S	9 15 12	2.2				Same as Above, medium dense
	2B (10'-15')	B		2.1				
15	2-3	S	11 14 14	2.3				(SP) Sand, fine to coarse with gravel to 3/4", gray, dense.
20	2-4	S	26 29 32	2.4				(SP) Sand, fine to coarse with gravel to 1", gray, very dense.
25	2-5	S	38 50/6"	3.0				(SP) Sand, fine to coarse with gravel to 1", gray, very dense.
30								

Cont. Next Page-------->

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San Bernardino, CA 92407
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LOG OF BORING 2

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 35.25 ft
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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	2-6	S	50/6"	2.4				(SW) Sand, fine to coarse, angular, gravel to 1/2", gray, very dense
35	2-7	S	50/3"	2.2				Cobble or boulder? Recovered as white rock chips about 1.25" in diameter, very dense
40								End of Boring Total Depth 35.25' No Refusal No Bedrock No Groundwater Disturbed native to about 3'.
45								
50								
55								Borehole: backfilled with drill spoils
60								

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San Bernardino, CA 92407
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LOG OF BORING B-3

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 16.5 ft
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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								Disturbed Native:
	3A (0'-5')	B		1.9		COR SA MDC		(SP) Sand, fine to coarse with gravel to 3/4", brown, 3% fines.
								Very young wash deposits-Qw2:
5	3-1	S	3 3 5	2.9				(SP-SM) Sand, fine to medium, traces coarse and gravel/rock fragments to 1.25", brown, loose
10	3-2	S	12 7 18	2.6				(SP) Sand, fine to medium, with coarse and gravel /rock fragments to 1.25", gray, medium dense
15	3-3	S	22 23 24	1.4				(SP) Sand, fine to coarse with granitic rock fragments to 1.25", gray, dense
20								
25								End of Boring Total Depth 16.5' No Refusal No Bedrock No Groundwater Hole backfilled with drill spoils
30								

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San Bernardino, CA 92407
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LOG OF BORING PB-1

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 5.5 ft
Groundwater Depth: Not Encountered

*Automatic Hammer
140 lbs 30-Inch Drop

**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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								Very young wash deposits-Qw2: (SP) Sand, fine to coarse with gravel to 3/4", brown.
	PB1A (0'-5')	B		0.6				
5	PB1-1	S	20 50/1"	2.3		SA		(SP-SM) Sand with Silt, fine to coarse with gravel to 3/8", brown, medium dense, 9% fines.
10								End of Boring Total Depth 5.5' No Refusal No Bedrock No Groundwater No Fill 3"-dig perc pipe packed with 3/4" gravel to surface
15								
20								
25								
30								

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San Bernardino, CA 92407
(909) 880-1146 FAX (909) 880-1557 email: info@geo-cal.com

LOG OF BORING PB-2

Project:
Proposed Gas Station
APN:0349-182-11-0000
Cajon Blvd, Glen Helen, CA

Date: 10-15-2021
Drilled By: Cal-Pac
Equipment: Mobil B-61
Hole Size: 8" HSA
Logged By: Todd Wyland, RCE

Total Depth: 9 ft
Groundwater Depth: Not Encountered

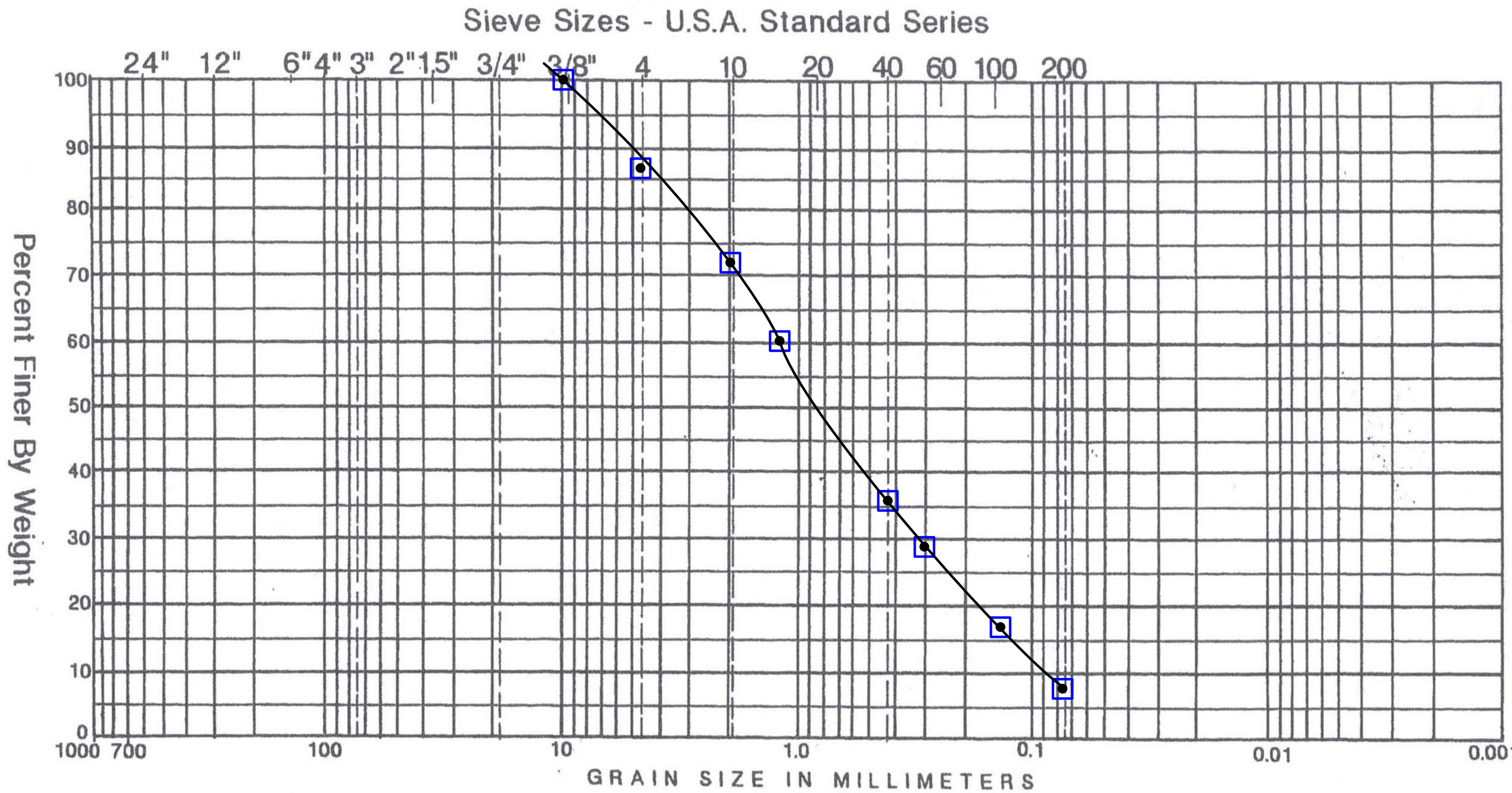
*Automatic Hammer 140 lbs 30-Inch Drop

** SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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								Very young wash deposits-Qw2: (SP) Sand, fine to medium, trace coarse, gravel to 1.5", brown
	PB2A	B		1.8				
5	PB2-1	S	7 3 4					(SP) Sand, fine to coarse with gravel to 3/8", graybrown, loose, 2% fines
	PB2B (5'-9')	B		2.3		SA		Boulder? at 9 ft.
10								End of Boring Total Depth 9' Refusal/ Bouncing on Rock at 9' No Bedrock No Groundwater Disturbed Surface 3"-dia. Perc Pipe Packed with 3/4" gravel to surface
15								
20								
25								
30								

APPENDIX B

LABORATORY GRAPHS
(4 PAGES)



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

Symbol	Boring	Depth	Classification
▣	PB-1	5 ft.	(SP-SM) Poorly graded Sand with Silt, fine to coarse with gravel to 3/8", 9% fines.

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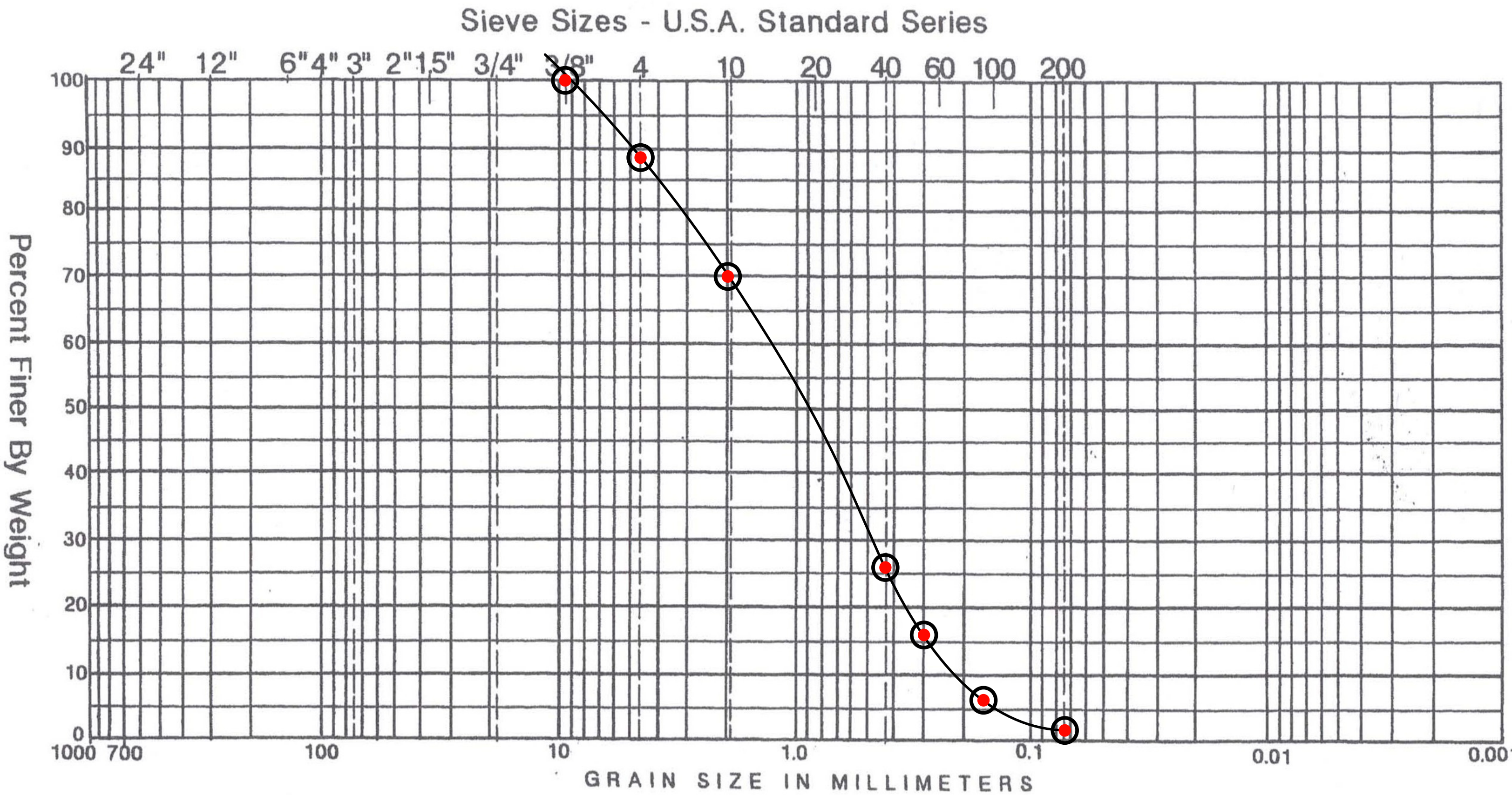
Environmental & Geotechnical Engineering

Gradation Curves

APN: 0349-182-11-0000
Glen Helen, CA



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

Symbol	Boring	Depth	Classification
⦿	PB-2	5-9" ft.	(SP) Poorly graded Sand, fine to coarse with gravel to 3/8", 2% fines.

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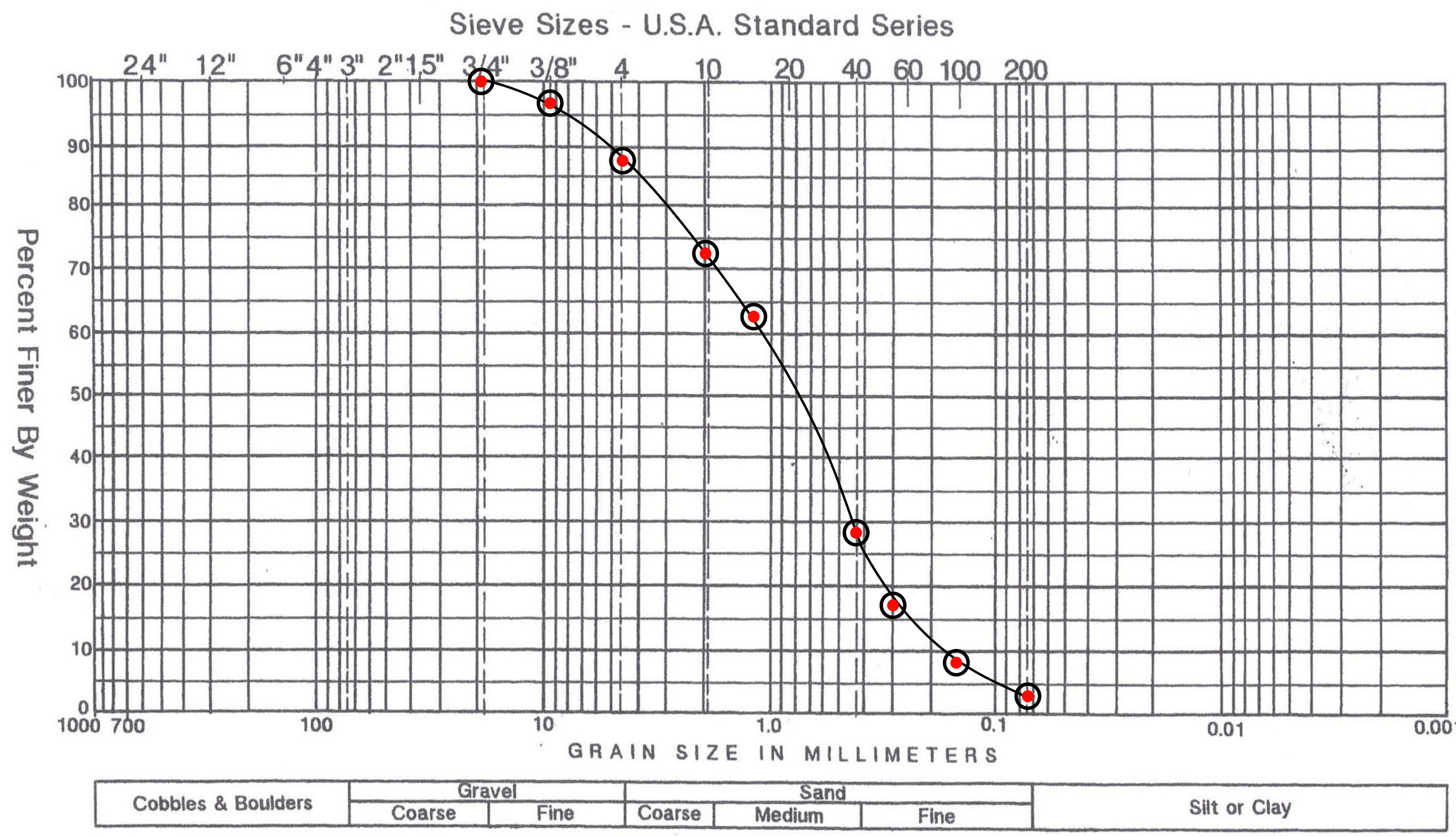
Environmental & Geotechnical Engineering

Gradation Curves

APN: 0349-182-11-0000
Glen Helen, CA



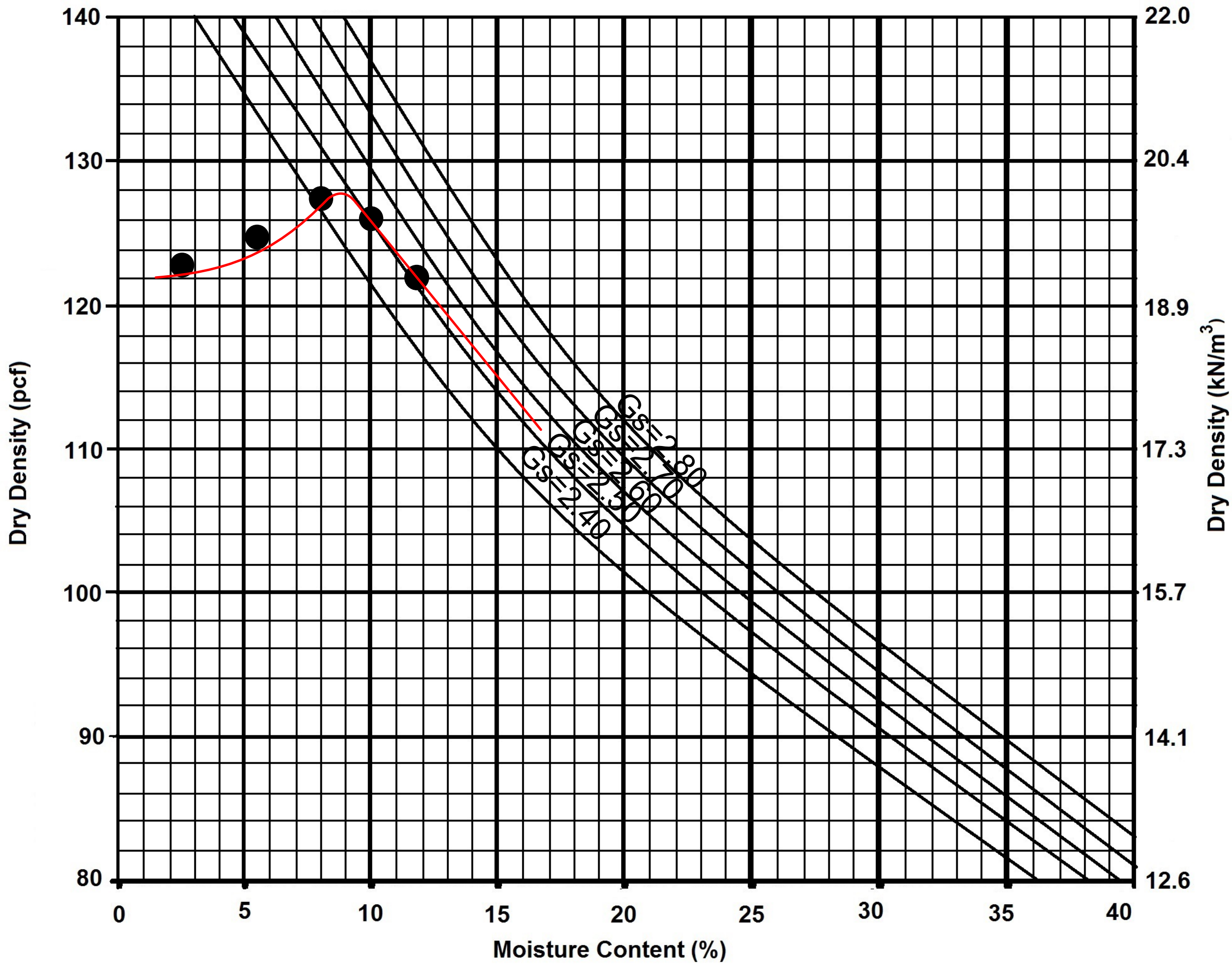
Symbol	Boring	Depth	Classification
●	B-3	0'-5'	(SP) POORLY GRADED SAND, FINE TO COARSE WITH GRAVEL TO 3/4", 3% FINES

Geo-Cal, inc.
Environmental & Geotechnical Engineering

Gradation Curves

APN: 0349-182-11-0000
Glen Helen, CA

MOISTURE DENSITY CURVE



Boring	Depth (ft)	Classification	γ max (pcf)	w_{opt} (%)
3	0'-5'	(SP) Sand, fine to coarse with gravel to 3/4"	128	9.0

MOISTURE DENSITY CURVE (MDC) ASTM D 1557

Project: Proposed Gas Station

Location: Cajon Blvd, Glen Helen, CA

Geo-Cal, inc.
Environmental & Geotechnical Engineering

APPENDIX C

HDR CORROSIVITY TEST RESULTS
(1 PAGE)

TRANSMITTAL LETTER

DATE: October 25, 2021

ATTENTION: **Todd Wyland**

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

SUBJECT: Laboratory Test Data

HDR Lab #21-1018LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

Table 1 - Laboratory Tests on Soil Samples

Geo-Cal, Inc.

HDR Lab #21-1018LAB
25-Oct-21

Sample ID			
			B-3 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	72,000
minimum		ohm-cm	18,800
pH			7.8
Electrical Conductivity		mS/cm	0.04
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	28
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	5.5
potassium	K^{1+}	mg/kg	6.8
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	149
fluoride	F^{1-}	mg/kg	0.9
chloride	Cl^{1-}	mg/kg	8.5
sulfate	SO_4^{2-}	mg/kg	5.1
nitrate	NO_3^{1-}	mg/kg	4.6
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Minimum resistivity and pH per CTM 643, Chloride per CTM 422, Sulfate 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

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

APPENDIX D

USGS SEISMIC DESIGN VALUES
(1 PAGE)

Cajon Blvd, Glen Helen

Latitude, Longitude: 34.219425, -117.401645

Date	11/20/2021, 12:47:22 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.369	MCE_R ground motion. (for 0.2 second period)
S_1	0.999	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.369	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.579	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	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	1.014	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.116	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	3.149	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.511	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.369	Factored deterministic acceleration value. (0.2 second)
S1RT	1.302	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.478	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.999	Factored deterministic acceleration value. (1.0 second)
PGAd	1.014	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.897	Mapped value of the risk coefficient at short periods
C_{R1}	0.881	Mapped value of the risk coefficient at a period of 1 s

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APPENDIX E

PERCOLATION TEST DATA
(2 PAGES)

Geo-Cal, inc.
Environmental & Geotechnical Engineering

Percolation Test Data Sheet

Project:	Glen Helen		Project No:	Henry Olivier		Date:	10-15-21
Test Hole No:		PB-1	Tested By:	Todd Wyland, RCE			
Depth of Test Hole, D_T:		66″	USCS Soil Classification:	(SP-SM) 9% fines			
Test Hole Dimensions (inches)				Length	Width		
Diameter (if round)=		8″	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	12:25	12:50	25	0	Empty	66	Yes
2	12:52	1:17	25	0	Empty	66	Yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	1:20	1:30	10	0	34.75	34.75	0.29
2	1:30	1:40	10	34.75	66.0	31.25	0.32
3	1:43	1:53	10	0	22.0	22.0	0.45
4	1:53	2:03	10	22.0	39.5	17.5	0.57
5	2:03	2:13	10	39.5	52.0	12.5	0.81
6	2:13	2:23	10	52.0	63.25	11.25	0.89
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

$$I_t = \frac{\triangle H \cdot 60 \cdot r}{\Delta t\,(r+2H_{avg})} = 13.0\ \text{in/hr}$$

FS=3

$I_{Design} = 4.34$ in/hr

>0.5 in/hr OK

Geo-Cal, inc.

Environmental & Geotechnical Engineering

Percolation Test Data Sheet

Project:	Glen Helen	Project No:	Henry Olivier	Date:	10-15-21
Test Hole No:	PB-2	Tested By:	Todd Wyland, RCE		
Depth of Test Hole, D_T:	108″	USCS Soil Classification:	(SP) 2% fines		
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round)=	8″	Sides (if rectangular)=			

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	1:15	1:40	25	0	Empty	108	Yes
2	1:43	2:08	25	0	Empty	108	Yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	2:10	2:20	10	0	58.0	58.0	0.17'7"
2	2:20	2:30	10	580	93.5	35.5	0.28
3	2:33	2:43	10	0	28.25	28.25	0.35
4	2:43	2:53	10	28.25	53.75	25.5	0.39
5	2:53	3:03	10	53.75	77.00	23.25	0.43
6	3:03	3:13	10	77.0	99.25	22.25	0.45
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

FS=3

$$I_t = \frac{\Delta H \cdot 60 \cdot r}{\Delta t\,(r + 2H_{avg})} = 12.2\ \text{in/hr}$$

$I_{Design} = 4.07$ in/hr

>0.5 in/hr OK