GEOTECHNICAL ENGINEERING REPORT

PROPOSED GAS STATION

WAGON TRAIN ROAD

PHELAN, CALIFORNIA

Prepared for

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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 Wagon Train Road in the Phelan area of San Bernardino County, California (**Figures 1 and 2**). The Site APN is 0351-171-55-0-000.

The Site consists of an existing compacted fill pad. Although requested, no information or documentation (compaction report) was provided for the existing fill.

1.1 Project Considerations

A proposed Site Plan (CUP-1), prepared by AGC Design Concepts, was provided for our use and was utilized for **Figure 2**. Based on the proposed Site Plan, it is our understanding that the Project will consist of a 4,900 sf ARCO AM/PM convenience store gas station with a car wash and Tesla charging cabinets.

The anticipated construction includes 9 multi-product fuel dispensers and steel canopy, fuel underground storage tanks (USTs) and associated piping, traffic access and parking pavements, walkways, landscaping, and signage.

An ALTA/NSPS Land Title Survey map was also provided for our use and was utilized for **Figure 3**. **A significant consideration of this project is the presence of the two private sewer easements shown on the Alta Map where existing septic seepage pits exist.** Although requested, documentation (percolation reports) or design information (depths, etc.) for the existing seepage pits have not yet been provided. The existing seepage pits are to be abandoned and no on-site wastewater disposal systems are proposed.

The sewer easements from the Alta Map were approximately transferred onto the proposed Site Plan and, as shown on **Figure 2**, they overlap with the north eastern portion of the proposed fuel canopy.

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. An excavation depth of 15 to 20 feet has been assumed for the USTs.

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 (36.5 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 1.42 acre compacted fill pad located on the southwest side of Wagon Train Road in the Phelan area of San Bernardino County, California. A Site Satellite Photo is included as **Figure 4**.

At the time of this investigation, topography of the Site was near planar with a slight southward slope. The elevation near the center of the Site was about 3,060 feet. Earthen berm was present along the southwest and southeast property lines atop descending 2(h) to 1(v) fill slopes with a maximum height of about 20 feet. A concrete down slope drainage device was present west of the southern corner of the Site as shown on **Figures 3 and 4**. Ascending fill slopes about 5 feet high were present along the northwest property line leading up to a McDonald's parking Lot.

Vegetation included a slight growth of weeds across the Site and 3 trees along the northwest property line. A couple of small piles of debris and a minor amount of scattered trash were seen near the west corner of the Site. Two steel storage containers were present in the northern portion of the Site. Large boulders and berm of loose soil and debris had been placed along Wagon Train Road to prevent vehicles from entering the Site.

Risers associated with the existing seepage pits were observed within the area of the two private sewage easements.

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 36.5 feet below the existing ground surface with a Mobil B-61 drill rig equipped with an automatic hammer for soil sampling. Percolation testing was conducted in two of the borings. The approximate locations of the exploratory and percolation 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. California ring samplers were typically utilized at the 5-ft level 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 encountered 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 the laboratories for testing and further evaluation.

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

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 and field dry densities of all relatively undisturbed ring samples (ASTM D 2937 and ASTM D 2216). The results are included on the exploratory logs in **Appendix A**.

Sieve analyses 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 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 about 20 feet of compacted fill over natural young alluvial fan deposits to the maximum depth of 36.5 feet attained.

The existing compacted fills were generally classified as fine to medium grained Silty Sand (SM), some with traces of coarse and gravel to ½ inch. The underlying natural alluvium was classified as poorly graded Sand (SP), fine to coarse grained with gravel and rock chips to ¾ inch.

The SPT and density data indicate that the existing fills are in place in a “medium dense” state and the underlying natural alluvium is in a “dense” state.

Compressible soil conditions or soils prone to hydro-consolidation when inundated with water and subjected to surcharge loading were not encountered.

The materials encountered at the Site were generally 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 to further drilling was not experienced.

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 36.5 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 GEOLOGIC SETTING AND GROUNDWATER

The Site is situated at an elevation of approximately 3060 feet Above Mean Sea Level (AMSL). It is located in the Cajon Pass which is a mountain pass between the San Bernardino Mountains to the east and the San Gabriel Mountains to the West.

A series of Cenozoic deposits underly the regional area of the Site, that in turn overly unconformably older basement rocks. Geologic formations mapped in the area include the Punchbowl Formation (Miocene age) of the Cajon Valley, described as non-marine cobbly to pebbly sandstone. Overlying the Punchbowl Formation in the Pliocene age, Crowder Formation which has been described as a non-marine arkosic sandstone and conglomerate. Foster (1980) divided the Crowder Formation into five units and provided detailed information on source areas and Paleo-current direction from the northeast. Overlying the Crowder Formation is the Pleistocene age, Harold Formation and Shoemaker Gravel. These are non-marine, fine to coarse grained sediments. Overlying the Harold Formation and Shoemaker Gravel are Pleistocene age well dissected alluvial fan deposits (Bortugo and Splitter, 1986).

Based on the current investigation, the site is underlain by Young Alluvial Fan Deposits (Qyf).

The Site is located approximately 5 miles northeast of the San Andreas Fault. It is not located within a mapped Alquist Priolo (AP) Earthquake Fault Zone.

Groundwater monitoring conducted for the Circle K Station, located approximately ½ mile northwest of Site, groundwater occurred at depth ranging between 45.51 and 57.98 feet in 2001.

No ground water or evidence of previous shallow groundwater (mottles) was encountered within any of the exploratory borings to the maximum depth of 36.5 feet attained.

7.0 LIQUEFACTION AND OTHER 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 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. Based on the results of our slope stability calculations, slope stability hazards are considered minimal.

The Site is not located within a mapped liquefaction hazard zone and inspection of the raw SPT blow count data indicate that the materials tested at the Site are sufficiently dense to preclude significant seismic settlement and liquefaction.

8.0 SLOPE STABILITY ANALYSES

To verify the gross stability of the existing adjacent compacted fill slopes, slope stability calculations were performed for both Spencer's and Bishop's Simplified methods of analysis using the SLIDE computer program. A simple 25 ft high 2(h) to 1(v) compacted fill slope over natural was analyzed for both static and seismic conditions at the location of Section A-A shown on **Figure 2**.

To model the shear strength of the existing medium dense Silty Sand (SM) compacted fill, a cohesion value of 250 psf and a friction angle of 33 degrees were used. For the underlying natural Sand (SP) with gravel alluvial deposits, a cohesion value of 1 psf and friction angle of 35 degrees were used.

The seismic calculations were performed pseudo-statically by applying a lateral seismic coefficient (k) of 0.25, which is larger than the minimum 0.15 due to the proximity to the San Andreas Fault.

The results of the slope stability calculations indicated acceptable 2.08 static and 1.25 seismic factors of safety against gross failure (**Appendix E**).

9.0 SITE INFILTRATION

Two boring percolation tests were 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 10 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 failing infiltration test rates (PB-1 at 10 ft was 0.12 in/hr and PB-2 at 5 ft was 0.17 in/hr). The percolation test data and calculated results are included in **Appendix F**.

10.0 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.

10.1 Initial Site Preparation

The existing compacted fill was found to be consistently medium dense Silty Sand (SM) and, unless disturbed, will not need to be removed and recompacted.

The existing seepage pits will need to 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 recommended.

10.2 Foundations and Settlement

If the Site is prepared as recommended, conventional spread foundations may be utilized in conjunction with the existing compacted fill to support the proposed building structures. Because the existing fills encountered were relatively uniform, and at least 15 feet deep, overexcavation and recompaction of the building pad areas will not be necessary. Drilled pier foundations for the proposed fuel canopy and pole sign(s) may be safely supported by the existing compacted fill encountered to depths greater than 15 feet. The proposed UST's are located in an area where the fill depth was greater than 20 feet deep.

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

If the site is properly prepared and the 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 density and SPT data, the existing fills and underlying natural alluvial fan deposits tested at the Site are sufficiently dense to preclude significant seismic settlement and liquefaction.

10.3 Infiltration

Based upon the materials and conditions of the compacted fill encountered in the exploratory borings, which were relatively impermeable medium dense silty sands, the fill being over 15 feet deep, and the failing infiltration rates of the subject percolation tests, it is the opinion of **GCI** that infiltration BMPs are not geotechnically feasible for the Site.

11.0 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.

11.1 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 the **existing septic system/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.

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

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.

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 and subsidence need not be considered due to the presence and condition of the existing compacted fill tested. Site clearing operations should also be considered when estimating earthwork quantities.

11.2 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.

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 one horizontal to one vertical (1H: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.

11.3 Utility Trenches

Standard construction techniques should be sufficient for utility trench excavations.

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.

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.

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

11.4 Foundation Preparation

Foundations for the proposed building structures may be safely supported by the existing compacted fill pad without the need to over excavate and recompact. Any disturbed footing base grade soil will need to be properly recompacted as recommended.

Foundations for the proposed canopy, pole signs, and UST's may also be safely supported by the existing compacted fill, which was determined to be over 15 feet deep at our exploratory boring locations. Natural cohesionless sand and gravel alluvial deposits that might contain larger clasts were encountered beneath the fill.

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

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

11.5 Foundation Design

The proposed building structures may be safely supported by conventional shallow foundations, either continuous wall footings and/or individual spread footings bearing in the existing compacted fill pad.

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 1,500** 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 3,500 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 settlement on the order of 1/2 inch. Differential settlement can be assumed to be approximately half the total settlement.

11.6 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 150 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.

11.7 Lateral Earth Pressures 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 retained materials and conditions have been determined and are applicable only to properly drained level backfill with no additional surcharge loading.

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

11.8 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.

11.9 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 25 and assumed Traffic Indexes (T.I.'s) of 4 and 6.5 for the parking stalls and drive areas, respectively:

Parking Stalls: 0.25' (3") AC over 0.50' (6") AB

Drive Areas: 0.33' (4") AC over 1.00' (12") 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.

11.10 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 (18 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** (41 ppm) are categorized as "**not corrosive**" to ferrous materials.

The **minimum resistivity** results (1,720 ohm-cm) are categorized as "**not corrosive**" to normal grade steel.

A **pH** of 8.1 "**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 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.

11.11 Infiltration

Because our percolation testing conducted on the existing fill pad indicated failing infiltration test rates for the fill and because the fill is over 15 feet deep, **infiltration BMP devices are not recommended for the Site**.

12.0 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 **Jerry Bajwa**, 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 **Jerry Bajwa**, 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 **Jerry Bajwa** 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.

13.0 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.

14.0 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.

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FIGURES

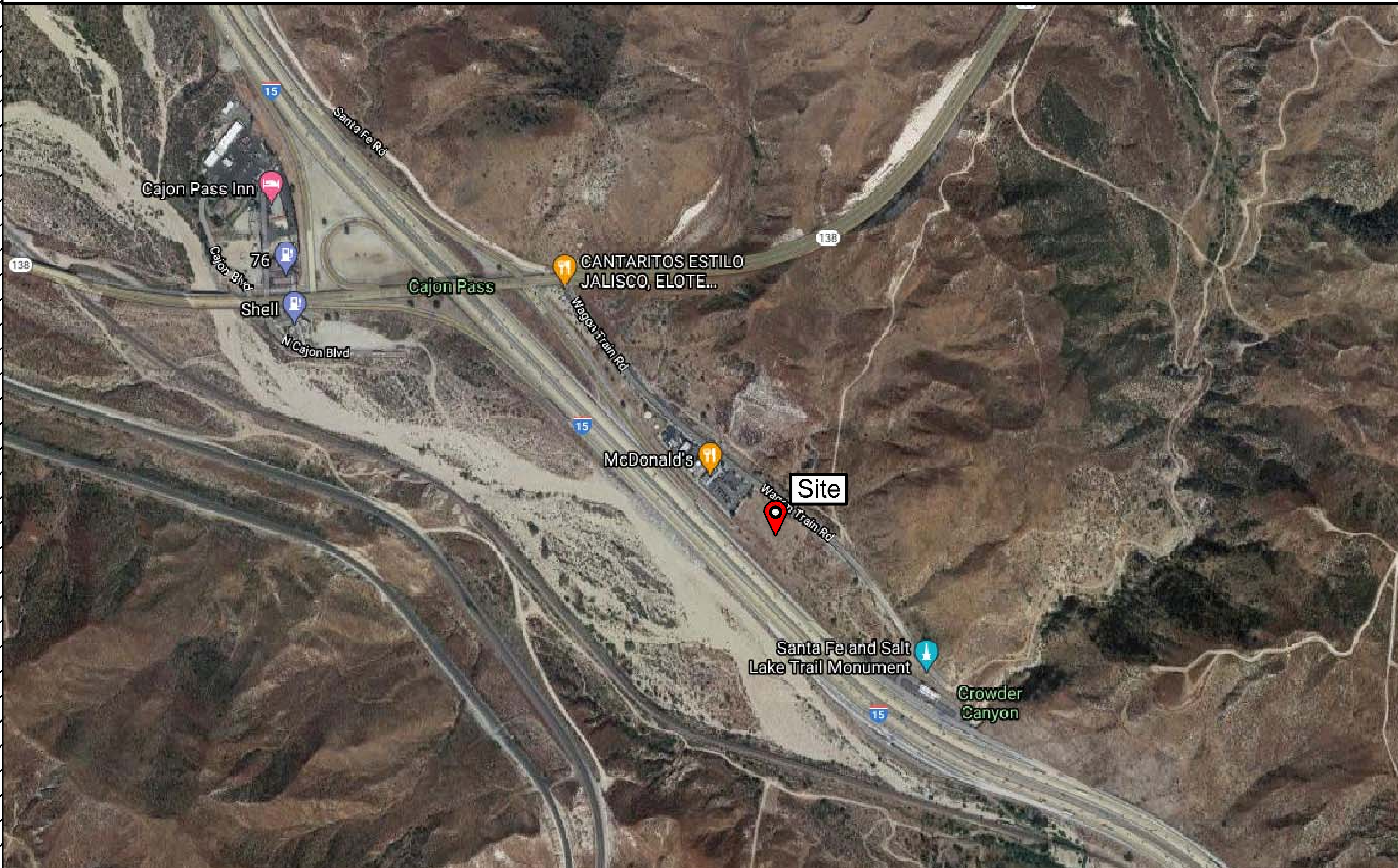

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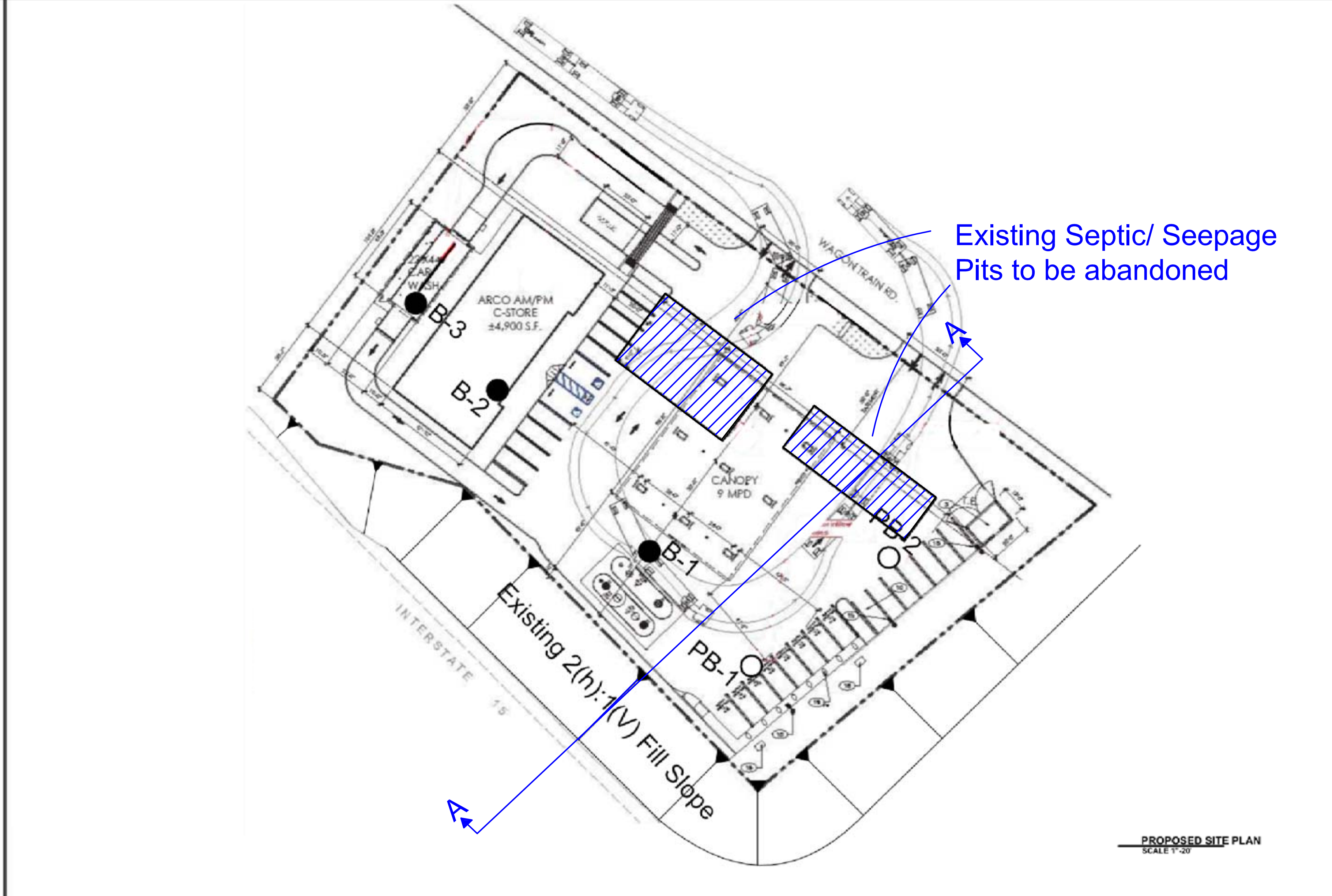
Figure 1
Site Location

New Gas Station
Wagon Train Road, Phelan, CA

Legend:

Site Location



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San Bernardino CA 92407

Figure 2
Site Plot Plan

New Gas Station
Wagon Train Road, Phelan, CA

Legend: (all locations are approx.)

B-3 Exploratory Boring

PB-2 Percolation Test Boring

A—A Slope Stability Cross-Section

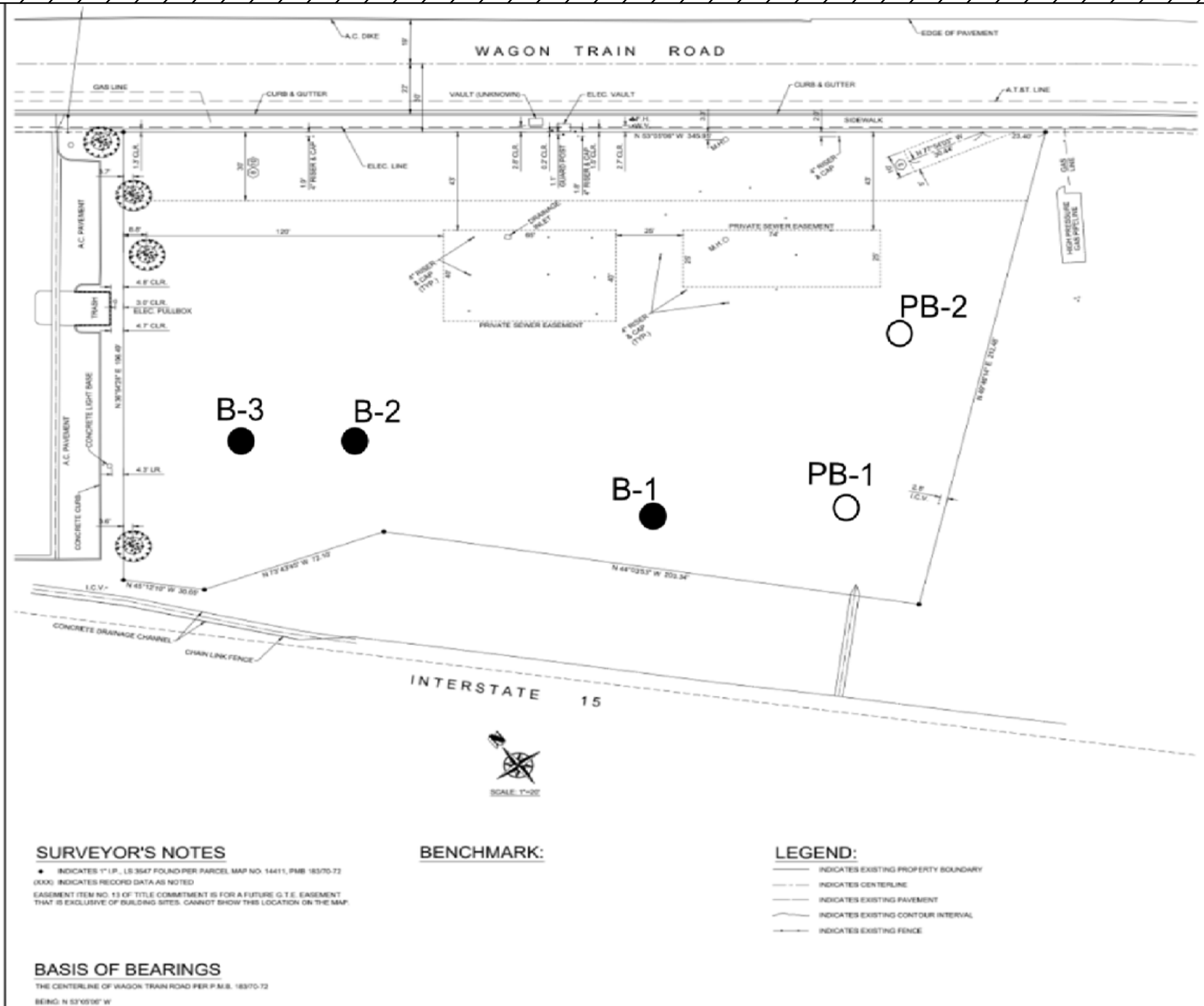

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Figure 3
Alta Map

New Gas Station
Wagon Train Road, Phelan, CA

Legend: (all locations are approx.)

B-3 ● Exploratory Boring

PB-2 ○ Percolation Test Boring

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San Bernardino CA 92407

Figure 4
Site Satellite Photo

New Gas Station
Wagon Train Road, Phelan, CA

Legend:
B-3 Exploratory Boring
PB-2 Percolation Test Boring

APPENDIX A

EXPLORATORY LOGS

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:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
Logged By: Todd Wyland, RCE

Total Depth: 36.5 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	1A (0'-5')	B		2.7		MDC CDR		(SM) Silty Sand, fine to coarse, trace gravel to 3/8", brown, medium dense
5	1-1	R	9 12 15	5.1 5.3	125.7			(SM) Silty Sand, fine to coarse, trace gravel to 3/8", brown, medium dense
	1B (5'-10')	B						
10	1-2	R	9 13 12	7.6 8.8	124.4			(SM) Silty Sand, fine to coarse, trace gravel to 3/8", brown, medium dense
	1C (10'-15')	B						
15	1-3	S	6 7 9	11.0				(SM) Silty Sand, fine to coarse, trace gravel to 3/8", gray, medium dense
20	1-4	S	5 7 8	11.1				(SM) Silty Sand, fine to coarse, trace gravel to 3/8", gray, medium dense
25	1-5	S	14 19 22	3.6		SA		Natural: (Qyt) young alluvial (SP) Sand fine to coarse with gravel to 3/4", gray brown, dense, 3% fines

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

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LOG OF BORING B-1

Project:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
Logged By: Todd Wyland, RCE

Total Depth: 36.5 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
30	1-6	S	18 21 18	3.2				(SP) Sand, fine to coarse with gravel and rock fragments to 3/4", gray brown, dense
35	1-7	S	25 26 17	2.1				(SP) Sand, fine to coarse with more gravel and rock fragments to 3/4", gray brown, dense
40								End of Boring Total Depth 36.5' No Refusal, No Groundwater, No Bedrock, <u>Compacted Fill</u> <u>Confirmed to 22'</u>
45								
50								Natural: (Qyt) young alluvial fan deposits were encountered at 25 sample depth and below. Hole backfilled with drill spoils
55								

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LOG OF BORING B-2

Project:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
Logged By: Todd Wyland, RCE

Total Depth: 21.5 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								Compacted Fill:
	2A (0'-5')	B		3.8		MDC		(SM) Silty Sand, fine to medium, trace coarse, gray brown, medium dense.
5	2-1	R	12 19 17	5.9	123.5			(SM) Silty Sand, fine to medium, trace coarse, gray brown, medium dense.
	2B (5'-10')	B		8.2				
10	2-2	S	5 6 11	6.7				(SM) Silty Sand, fine to medium, trace coarse, gray brown, medium dense.
15	2-3	S	4 6 8	10.7 10.7				(SM) Silty Sand, fine to medium, gray brown, medium dense, trace gravel to 1/2".
	2C (10'-15')	B						
20	2-4	S	6 8 9	10.6				Trace Organics (roots) at about 20' End of Boring
25								Total Depth 21.5' No Refusal No Groundwater No Bedrock Compacted Fill to 21.5', Hole Backfilled with Drill Spoils

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San Bernardino, CA 92407
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LOG OF BORING B-3

Project:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
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								Compacted Fill:
	3A (0'-5')	B		4.0				(SM) Silty Sand,fine to medium, trace coarse, gray brown, medium dense
5	3-1	R	11 13 14	8.0	122.9			(SM) Silty Sand,fine to medium, trace coarse, gray brown, medium dense
	3B (5'-10')	B		8.3				
10	3-2	S	5 7 10	9.1				(SM) Silty Sand,fine to medium, trace coarse, gray brown, medium dense
	3C (10'-15')	B		10.5				
15	3-3	S	5 7 9	11.5				End of Boring
20								Total Depth 16.5' No Refusal No Groundwater No Bedrock Compacted Fill to 16.5', Hole Backfilled with Drill Spoils
25								

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

LOG OF BORING PB-1

Project:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
Logged By: Todd Wyland, RCE

Total Depth: 11.5 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								
	PB-1A (0'-5')	B		3.9				(SM) Silty Sand, fine to medium with coarse and gravel to 1/2", gray brown, medium dense.
5	PB1-1	S	6 6 7	8.1 9.6				
	PB-1B (5'-10')	B						(SM) Silty Sand, fine to medium with coarse and gravel to 1/2", gray brown, medium dense.
10	PB1-2	S	5 6 8	8.8		SA		(SM) Silty Sand, fine to coarse with gravel to 1", gray brown, medium dense, 24% fines.
15								End of Boring Total Depth 11.5' No Refusal No Groundwater No Bedrock Compacted Fill to 11.5' 3"-Diameter perc pipe packed with gravel to the surface
20								
25								

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LOG OF BORING PB-2

Project:
APN: 0351-171-55-0-0000
Wagon Train Rd. Phelan, CA

Date: 10-15-21
Drilled By: Cal Pac Drilling
Equipment: Mobil B-61
Hole Size: 8-Inch Hollow Stem Auger
Logged By: Todd Wyland, RCE

Total Depth: 6.5 ft
Groundwater Depth: Not Encountered

*Automatic Hammer 140 lbs 30-Inch Drop

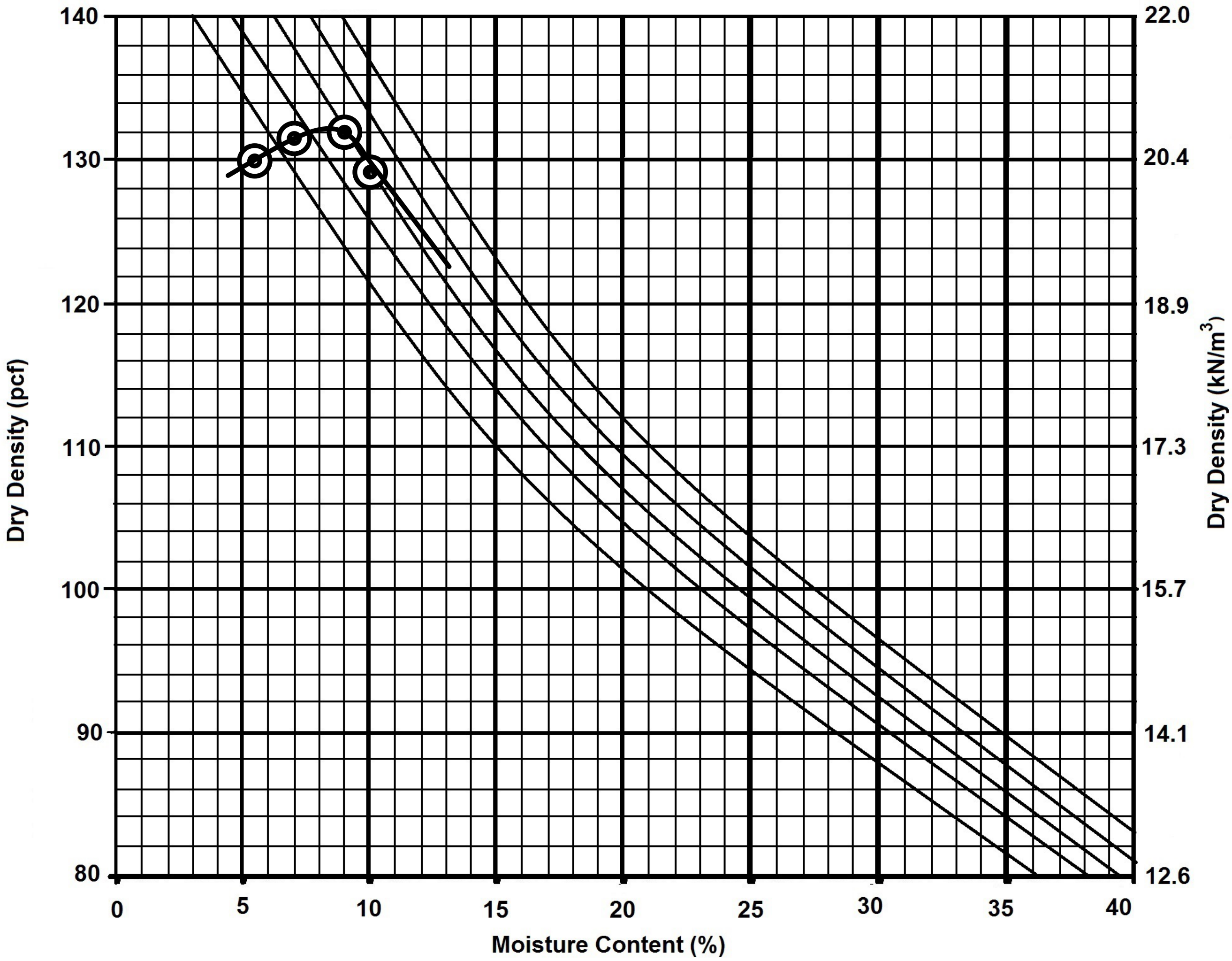
**
SA=Sieve Analysis
MDC=(ASTM D 1557)
COR= Caltrans Corossivity

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	PB-2A (0'-5')	B		4.2				(SM) Silty Sand, fine to medium , trace coarse and gravel to 1/2", brown.
5	PB2-1	S	5 9 12	7.9		SA		(SM) Silty Sand, fine to coarse, gray brown, medium dense, 25% fines.
								End of Boring Total Depth 6.5' No Refusal No Groundwater No Bedrock Compacted Fill to 6.5' 3"-Diameter perc pipe packed with gravel to the surface
10								
15								
20								
25								

APPENDIX B

LABORATORY GRAPHS

MOISTURE DENSITY CURVE



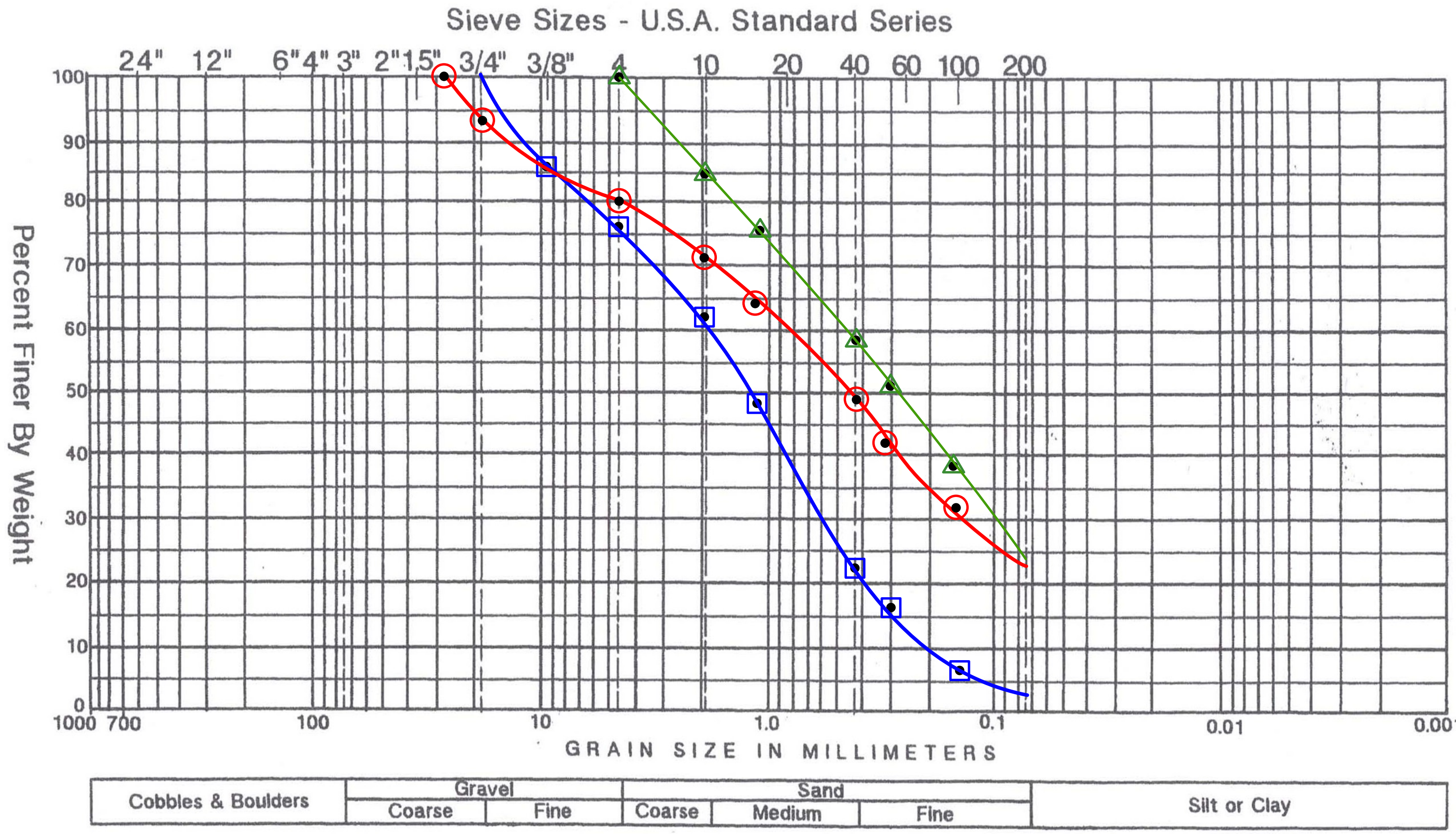
Boring	Depth (ft)	Classification	γ_{max} (pcf)	w_{opt} (%)
B-1&2	0-5	(SM) Silty Sand, fine to coarse,	132	9.0
		trace gravel to 3/8", brown,		
		compacted fill		

MOISTURE DENSITY CURVE (MDC) ASTM D 1557

Project: Wagon Train Road

Location: APN: 0351-171-55-0000, Phelan, CA

Geo-Cal, inc.
Environmental & Geotechnical Engineering



Symbol	Boring	Depth	Classification
⊙	PB-1	11 ft.	(SM) Silty Sand, fine to coarse with gravel to 1" 24% fines, fill
△	PB-2	6 ft.	(SM) Silty Sand, fine to coarse, 25% fines, fill
⊡	1	26 ft.	(SP) Poorly Graded Sand, fine to coarse with gravel to 3/4%"
			3% fines, Natural Alluvial

Geo-Cal, inc.
Environmental & Geotechnical Engineering

Gradation Curves

APN: 0351-171-55-0000

Wagon Train Road, Phelan, CA

APPENDIX C

HDR CORROSIVITY TEST RESULTS

HDR

Table 1 - Laboratory Tests on Soil Samples

Geo-Cal, Inc.

HDR Lab #21-1017LAB
25-Oct-21

Sample ID			
			B-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	3,760
minimum		ohm-cm	1,720
pH			8.1
Electrical Conductivity		mS/cm	0.15
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	21
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	174
potassium	K^{1+}	mg/kg	5.7
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	66
bicarbonate	HCO_3^{1-}	mg/kg	113
fluoride	F^{1-}	mg/kg	5.0
chloride	Cl^{1-}	mg/kg	41
sulfate	SO_4^{2-}	mg/kg	18
nitrate	NO_3^{1-}	mg/kg	50
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

APPENDIX D

USGS SEISMIC DESIGN VALUES

Wagon Train Road

Latitude, Longitude: 34.308277, -117.46943

Date	10/23/2021, 2:18:36 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.233	MCE_R ground motion. (for 0.2 second period)
S_1	0.946	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.233	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.489	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	0.961	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.057	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	3.094	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.426	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.233	Factored deterministic acceleration value. (0.2 second)
S1RT	1.277	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.442	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.946	Factored deterministic acceleration value. (1.0 second)
PGAd	0.961	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.903	Mapped value of the risk coefficient at short periods
C_{R1}	0.885	Mapped value of the risk coefficient at a period of 1 s

APPENDIX E

SLIDE SLOPE STABILITY RESULTS

Section A-A'
(Looking NW)

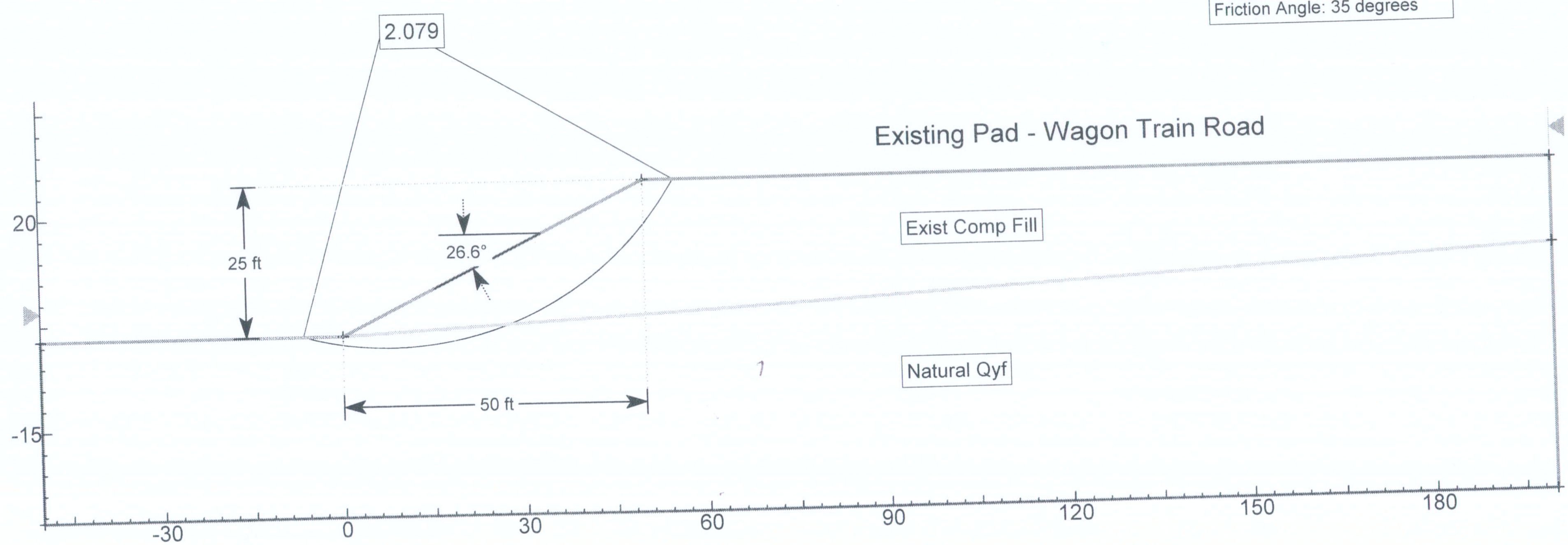
Method spencer

Factor of Safety: 2.079
Center: 8.135, 52.231
Radius: 54.302
Left Slip Surface Endpoint: -6.720, 0.000
Right Slip Surface Endpoint: 55.116, 25.000

Material Properties

Material: Exist Comp Fill
Strength Type: Mohr-Coulomb
Unit Weight: 124 lb/ft3
Cohesion: 250 psf
Friction Angle: 33 degrees

Material: Natural Qyf
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 1 psf
Friction Angle: 35 degrees



Scale 1:312.

Section A-A'
(Looking NW)

Method bishop simplified

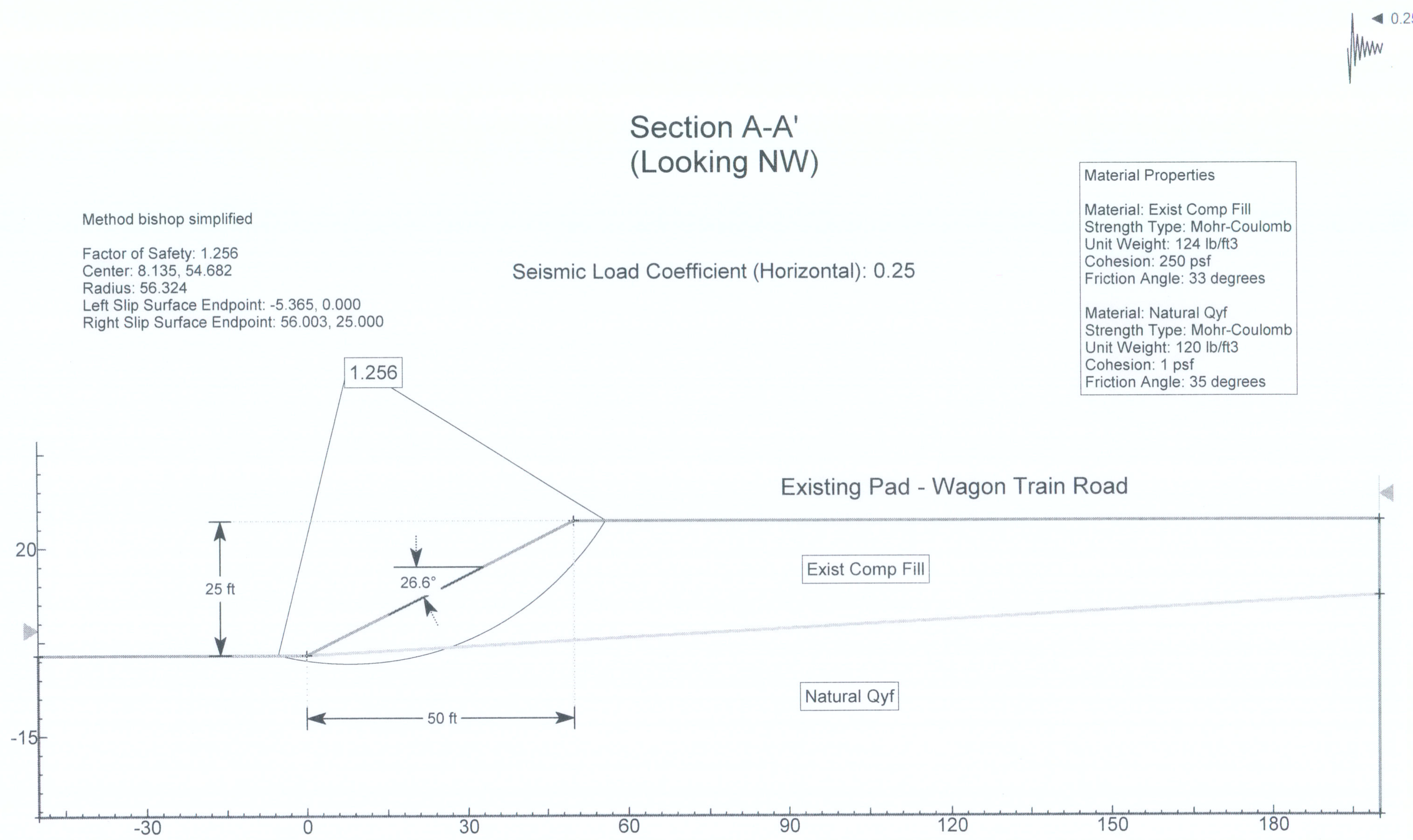
Factor of Safety: 1.256
Center: 8.135, 54.682
Radius: 56.324
Left Slip Surface Endpoint: -5.365, 0.000
Right Slip Surface Endpoint: 56.003, 25.000

Seismic Load Coefficient (Horizontal): 0.25

Material Properties

Material: Exist Comp Fill
Strength Type: Mohr-Coulomb
Unit Weight: 124 lb/ft3
Cohesion: 250 psf
Friction Angle: 33 degrees

Material: Natural Qyf
Strength Type: Mohr-Coulomb
Unit Weight: 120 lb/ft3
Cohesion: 1 psf
Friction Angle: 35 degrees



Scale 1:312.7

APPENDIX F

PERCOLATION TEST DATA

Geo-Cal, inc.
Environmental & Geotechnical Engineering

Percolation Test Data Sheet

Project:	Wagon Train Rd		Project No:	Jerry Bajwa		Date:	10-15-21
Test Hole No:		PB-1	Tested By:	Todd Wyland, RCE			
Depth of Test Hole, D_T:		120″	USCS Soil Classification:		(SM) Compacted Fill		
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	8:10	8:35	25	0	10.5	10.5	Yes
2	8:35	9:00	25	10.5	17	6.5	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	9:00	9:10	10	17.0	18.25	1.25	8
2	9:10	9:20	10	18.25	19.50	1.25	8
3	9:20	9:30	10	19.50	20.50	1.0	10
4	9:30	9:40	10	20.50	21.50	1.0	10
5	9:40	9:50	10	21.50	22.50	1.0	10
6	9:50	10:00	10	22.50	23.50	1.0	10
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

FS=3

$$I_t = \frac{\Delta H \cdot 60 \cdot r}{\Delta t\,(r+2H_{avg})} = 0.12\ \text{in/hr} \qquad I_{Design} = 0.04\ \text{in/hr}$$

<0.5 in/hr NOT OK

Geo-Cal, inc.
Environmental & Geotechnical Engineering

Percolation Test Data Sheet

Project:	Wagon Train Rd		Project No:	Jerry Bajwa	Date:		10-15-21
Test Hole No:		PB-2	Tested By:	Todd Wyland, RCE			
Depth of Test Hole, D_T:		54″	USCS Soil Classification:		(SM) Compacted Fill		
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	8:25	8:50	25	0	10.0	10.0	Yes
2	8:50	9:15	25	10.0	17.0	7.0	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	9:15	9:25	10	17.0	18.0	1.0	10
2	9:25	9:35	10	18.0	18.75	0.75	15
3	9:35	9:45	10	18.75	9.50	0.75	15
4	9:45	9:55	10	19.50	20.0	0.5	20
5	9:55	10:05	10	20.0	20.50	0.5	20
6	10:05	10:15	10	20.50	21.0	0.5	20
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

FS=3

$$I_t = \frac{\Delta H \cdot 60 \cdot r}{\Delta t\,(r+2H_{avg})} = 0.17\ in/hr \qquad I_{Design} = 0.06\ in/hr$$

<0.5 in/hr NOT OK