Appendix 5

Geotechnical Engineering Report

October 17, 2022
(Revised November 9, 2022)

Project No. 22504

Mr. Michael Ramirez
Beyond Foot Mart
4300 Edison Avenue
Chino, CA 91710

Subject: Limited Geotechnical Engineering Report
Proposed Commercial Development
24831 Clinton Keith Road, Wildomar, California

Dear Mr. Ramirez:

In accordance with your request and authorization, we have completed a preliminary geotechnical study for the design and construction of the subject structures. We are presenting, herein, our findings and recommendations.

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project. This report was prepared in accordance with the requirements of the 2019 California Building Code and the City of Wildomar requirements.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned at (657) 888-4608 or info@ntsgeo.com.

Respectfully submitted,
NTS GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
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Principal Engineer

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INTRODUCTION

This report presents results of the limited geotechnical study conducted on the subject site for the proposed commercial development located at 24831 Clinton Keith Road, in the City of Wildomar, County of Riverside, California. The general location of the subject site is indicated on Plate 1 – Location Map.

SITE DESCRIPTION

The subject property comprises approximately 4.35 acres and was nearly rectangular in shape. The subject property is located at 24831 Clinton Keith Road in the City of Wildomar, California.

The subject property is bound by existing self-storage facility on the south and west, by Jana Lane on the east and Clinton Keith Road on the north.

PURPOSE AND SCOPE OF STUDY

The scope of work performed for this study was designed to evaluate the surface and subsurface conditions of the subject site with respect to geotechnical characteristics, including potential geologic hazards that may affect the development of the site, and to provide geotechnical recommendations and criteria for use in the design and construction of the proposed development. The scope of work included the following:

- Review of locally and readily available published and unpublished soils and geologic reports and data for the site and surrounding areas (see References section), Google Earth photographs, flood hazard maps, well data, etc. to ascertain earth material, geologic, and hydrologic conditions of the area.
- Utilize subsurface data performed by others for analysis.
- Utilize previous geotechnical laboratory test results performed by others.
- Define the general geology of the subject site and evaluate potential geologic hazards which would influence the proposed site development.
- Determine seismic classification of the site to meet the requirements of the 2019 California Building Code (CBC).
- Engineering analysis of previous field and laboratory data to provide a basis for geotechnical conclusions and recommendations regarding site grading and foundation, floor slab, retaining wall, etc. design parameters.

- Preparation of this report to present the preliminary geotechnical and geologic conclusions and recommendations for the proposed site development.

This report presents our preliminary conclusions and/or recommendations regarding:

- The geologic setting of the site.
- Potential geologic hazards (including landslides, seismicity, faulting, liquefaction potential, etc.)
- General subsurface earth conditions.
- Presence and effect of expansive and compressible earth materials.
- Groundwater conditions within the depth of our subsurface study.
- Excavation characteristics of the on-site earth materials.
- Characteristics and compaction requirements of proposed fill and backfill materials.
- Recommendations and guide specifications for earthwork.
- Seismic design coefficients for structural design purposes.
- Types and depths of foundations.
- Allowable bearing pressure and lateral resistance for foundations.
- Temporary and permanent cut and fill slope recommendations.
- Slope maintenance and protection recommendations.

The scope of work performed for this report did <u>not</u> include any testing of earth materials or groundwater for environmental purposes, an environmental assessment of the property, or opinions relating to the possibility of surface or subsurface contamination by hazardous or toxic substances.

This study was prepared for the exclusive use of **Beyond Food Mart** and their consultants for specific application to proposed structures in accordance with generally accepted standards of the geotechnical professions and generally accepted geotechnical (soil and foundation) engineering and practices at the time this report was prepared. Other warranties, implied or expressed, are not made. Although reasonable effort has been made to obtain information regarding geotechnical and subsurface conditions of the site, limitations exist with respect to knowledge of unknown regional or localized off-site conditions which may have an impact at the site. The conclusions and recommendations presented in this report are valid as of the date of this report. However, changes in conditions of a property can occur with passage of time, whether they are due to natural processes or to works of man on this and/or adjacent properties.

If conditions are observed or information becomes available during the design and construction process which are not reflected in this report, NTS, as Geotechnical Consultant of record for the project, should be notified so that supplemental evaluations can be performed and conclusions and recommendations presented in this report can be verified or modified in writing, as necessary. Changes in applicable or appropriate standards of care in the geotechnical professions occur, whether they result from legislation or the broadening of knowledge and experience. Accordingly, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes outside the influence of the project Geotechnical Consultant which occur in the future.

PROPOSED DEVELOPMENT

Based upon information presented to this firm by the client, it is our understanding that the proposed project will consist of construction of new gas station, car wash, fuel pump station, two retail buildings and an office/warehouse building at the southern side of the property. All structures all planned to be 1-story structures and constructed at-grade.

The above project description and assumptions were used as the basis for the field exploration, laboratory testing program, the engineering analysis, and the conclusions and recommendations presented in this report. NTS should be notified if structures, foundation loads, grading, and/or details other than those represented herein are proposed for final development of the site so a review can be performed, a supplemental evaluation made, and revised recommendations submitted, if required.

FIELD EXPLORATION

The field study performed for this report included utilizing existing data from previous study performed by Others.

The subsurface exploration performed by others consisted of excavating ten (10) hollow stem auger borings to a maximum depth of 23 feet below the existing grade. The approximate locations of the exploratory excavations are shown on Plate 2 – Geotechnical Map. Logs of the borings are presented in Appendix A – Field Exploration.

The result of the subsurface soil conditions are presented on Plate 3 – Geotechnical Section.

GEOTECHNICAL LABORATORY TESTING

Laboratory testing was performed on bulk and undisturbed samples collected during the previous subsurface exploration performed by Others. Testing was performed on soil samples and included the following tests:

- Moisture and density
- Maximum density and optimum moisture content
- Sieve analysis; and
- Corrosivity.

Laboratory test results from the previous investigations are presented in Appendix B of this report.

It is recommended that samples be obtained at the completion of rough grading and remolded direct shear, consolidation and expansion index be performed to confirm the foundation recommendations provided in this report.

GEOLOGIC FINDINGS

Regional Geologic Setting

According to the geologic maps, we note that the subject property underlain by Monzogranite to granodiorite bedrock (Kpvg) that consists of pale gray, massive, medium-grained monzogranite rock.

Subsurface Materials

Majority of the materials encountered during the subsurface investigation performed by others consist of alluvium ranging from about 2 to 16 feet in thickness underlain by monzogranite bedrock to the to total depth of the exploration.

In general, the alluvium consists light brown to red brown, slightly moist to damp, medium dense to dense, silty sands.

The bedrock consists of pale brown to pale gray, fine- to- medium grained, dense to very dense Monzogranite.

Groundwater

Groundwater was not encountered in the exploratory excavations to the maximum depth explored of approximately 23 feet below existing ground surface at the time the field study was performed for this report.

No groundwater data was found during a literature search pertaining to the subject property. There are no known shallow groundwater bearing soil or rock formations beneath the subject property. No evidence of onsite springs was found during the field study. Based on anticipated lot grading and the inferred groundwater depths, groundwater should not be a factor for project design or long-term performance.

Surface water was not observed on the subject site at the time the field study was performed for this report.

Based on results of our subsurface exploration and experience, variations in the continuity and nature of surface and subsurface conditions should be anticipated. Due to uncertainty involved in the nature and depositional characteristics of earth materials at the site, care should be exercised in extrapolating or interpolating subsurface conditions between and beyond the exploratory excavation locations.

Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions and may change over time as a consequence of seasonal and meteorological fluctuations, or activities by humans at this site and nearby sites. However, based on the above findings, groundwater is unlikely to impact the proposed development.

GEOLOGIC HAZARDS

Faulting and Seismicity

The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on the reviewed geologic maps crossing the site, however, the site is located in the seismically active region of Southern California. The nearest known active fault is the Elsinore fault system, which is located approximately 1.2 mile from the site, and capable of generating a maximum earthquake magnitude of 7.9. The site is also located approximately 1,885 feet east of the County's Unnamed fault in the Elsinore fault zone.

Liquefaction and Seismic Settlement

Liquefaction occurs when the pore pressures generated within a soil mass approach the effective overburden pressure. Liquefaction of soils may be caused by cyclic loading such as that imposed by ground shaking during earthquakes. The increase in pore pressure results in a loss of strength, and the soil then can undergo both horizontal and vertical movements, depending on the site conditions. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion.

Based on our review of the County of Riverside Map My County website, approximately the southern ¾ of the site is located within a moderate liquefaction hazard zone. Based on lack of shallow groundwater and the presence of shallow bedrock and our liquefaction analysis as presented within Appendix D of this report, it is our professional opinion that the potential for liquefaction to occur is low.

Landslides

The subject site is not located within an earthquake-induced landslide zone. Field reconnaissance did not disclose the presence of older, existing landslides on or near the subject property. In addition, due to the relatively gentle sloping of the site, the potential for landslides to impact the proposed development is considered low.

Flooding

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in administering the National Flood Insurance Program. Based on our review of the FEMA flood map, the site is located in an area identified as Area of Minimal Flood Hazard (Zone X).

Tsunami and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. The site is not located on any State of California – County of Riverside Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible because the site is located several miles inland from the Pacific Ocean shore, at an elevation exceeding the maximum height of potential tsunami inundation.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

GEOTECHNICAL ENGINEERING FINDINGS

Expansive Soil

Based on our evaluation and experience with similar material types, the soil encountered near the ground surface at the site exhibit a very low expansion potential.

Soil Corrosion

The potential for the on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on representative soil samples to evaluate pH, minimum resistivity, and soluble chloride and sulfate contents. The results of our corrosivity testing is presented within Appendix B of this report. General recommendations to address the corrosion potential of the on-site soils are provided below. Imported fill materials, if used, should be tested to evaluate whether their corrosion potential is more severe than those assumed.

<u>Structural Concrete</u>

Laboratory tests indicate that the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” or “S0” exposure in accordance with ACI 318, Table 19.3.1.1. Therefore, restriction on the type of cement, water to cement ratio, and compressive strength is not required.

<u>Ferrous Metal</u>

The results of the laboratory chemical tests performed on a sample of soil collected within the site indicate that the on-site soils are mildly corrosive to ferrous metals. Consequently, metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.) may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. Additional provisions will be required to address high chloride contents of the soil per the 2019 CBC to protect the concrete reinforcement. The laboratory testing program performed for this project does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary).

The above discussion is provided for general guidance in regards to the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements are beyond our purview. If detailed testing is required, a corrosion engineer should be consulted to perform the testing and develop appropriate mitigation measures.

Preliminary Infiltration Testing

Two (2) infiltration tests were performed others previously in general conformance with the County of Riverside requirements. The borings were excavated to a depth of 5 and 10 feet below the existing grade using a hollow-stem-auger drill rig. The result of our infiltration testing is summarized in the table below, which includes a factor of safety of 3.

Preliminary Infiltration Rates Summary

Boring No.	Depth Below Existing Grade (feet)	Factored Infiltration Rate (inches/hour)
I-1	5	0.18
I-2	5	0.23

Based on our infiltration testing and due to the presence of very dense bedrock underlying the site, infiltration within the site soils is deemed not feasible from a geotechnical standpoint. Alternate methods of disposing of stormwater should be considered by the project civil engineer.

Excavation Characteristics

The native soil materials underlying the site can be excavated with conventional grading equipment (i.e., backhoes, excavators, or loaders). However, the bedrock materials may be difficult to excavate and will heavy duty equipment or jack hammers.

GEOTECHNICAL ENGINEERING CONLUSIONS AND RECOMMENDATIONS

Conclusions

The conclusions and recommendations presented in this report are based on information provided to this firm, the results of the field and laboratory data obtained from ten (10) exploratory excavations located on the subject property, experience gained from work conducted by this firm on projects within the general vicinity of the subject site, the project description and assumptions presented in the ‘Proposed Development’ section of this report, engineering analyses, and professional judgement.

Based on a review of the field and laboratory data and the engineering analysis, the proposed development is feasible from a geotechnical standpoint. The subject property can be developed without adverse impact onto or from adjoining properties providing the recommendations contained within this report are adhered to during project design and construction.

The field observations indicate that the upper 3 feet of the site soils are considered loose and compressible and are not considered suitable for the support of structural fills, foundations, slab-on-grade floor slabs, hardscape, and/or pavement without removal and replacement as compacted fill. On this basis, it is recommended that the upper 3 feet of the site soils be removed and replaced as engineered fill in order to densify the material and to reduce the potential for additional settlement to occur. Additionally, new foundations should not span a cut/fill transition. Wherever a cut/fill transition occurs, the cut portion of the pad should be excavated to at least 3 feet below the bottom of footing and the excavated material be placed as engineered fill to create a uniform blanket of engineered fill and minimize differential movement.

The actual conditions of the near-surface supporting material across the site may vary. The nature and extent of variations of the surface and subsurface conditions between the exploratory excavations may not become evident until construction. If variations of the material become evident during construction of the proposed development, NTS should be notified so that the project Geotechnical Consultant can reevaluate the characteristics of the material and the conclusions and recommendations of this report, and, if needed, revise the conclusions and recommendations presented herein.

Preliminary recommendations for site grading, foundations, slab support, and spa design are presented in the subsequent paragraphs.

Site Preparation

Site preparation should begin with the removal of utility lines, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be selectively removed and disposed offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed at a legal dump site away from the project areas.

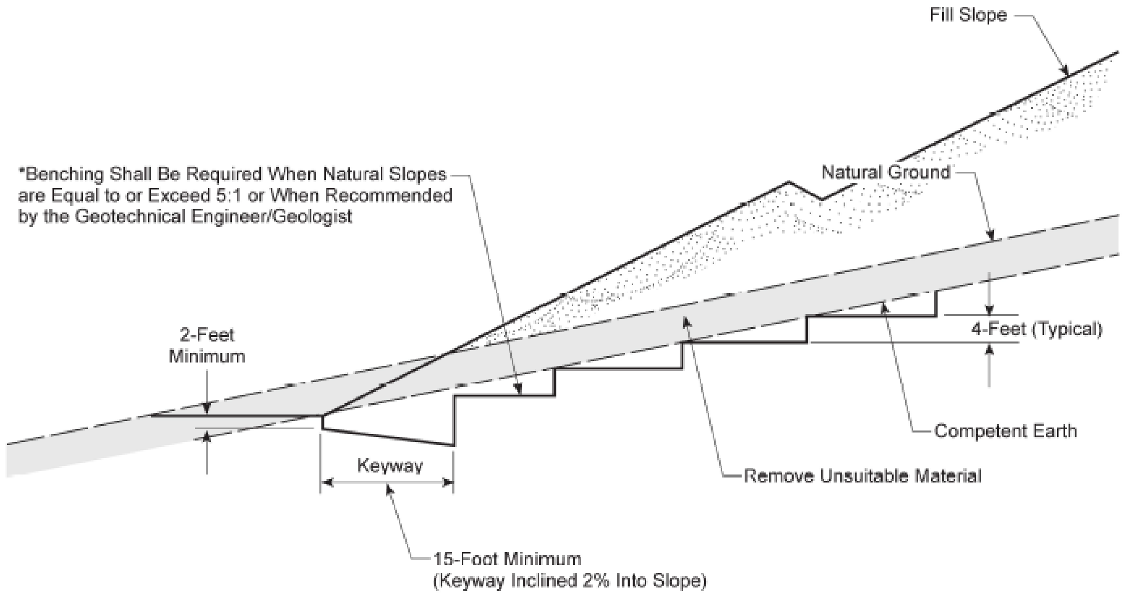
Grading

It should be noted that the recommendations provided herein are based on our subsurface exploration and knowledge of the on-site geology. Actual removals may vary in configuration and volume based on observations of geologic materials and conditions encountered during grading. The bottom of all corrective grading removals should be observed by a representative of NTS to verify the suitability of in-place soil prior to performing scarification and recompaction.

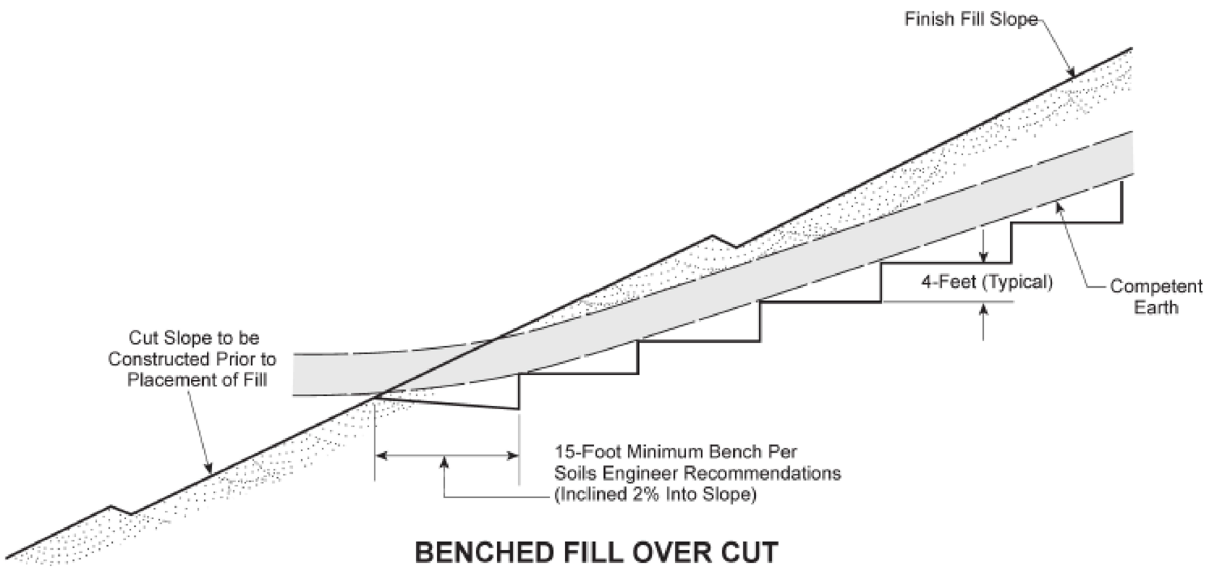
The need for corrective grading, i.e., removal of existing soil materials from areas to receive fill, where exposed at design grade in cut and fill areas

Fill Support Benches:

Benches should be excavated through any non-engineered fill, topsoil, or loose soil wherever the toe of a fill slope is located at a natural ground surface having a gradient of 5 horizontal to 1 vertical, or steeper, or in flatter areas where recommended by the geotechnical consultant. Horizontal benches should also be excavated at the daylight line between cut and fill portions of all graded slopes. The bottom of these keys and initial benches should be 15 feet in minimum width, and any non-engineered fill, topsoil, or native soil should be completely removed to expose undisturbed, in-place rock unit materials. Further benching should be performed uphill from these keys or initial benches simultaneously with fill placement to remove surficial soil materials and provide additional level surfaces for fill support where the natural ground surface is 5 horizontal to 1 vertical, or steeper. Benching detail is provided in the figure below.



BENCHED FILL OVER NATURAL



BENCHED FILL OVER CUT

Warehouse Building Pad:

The proposed warehouse building pad should be supported on engineered fill, a minimum of 4 feet below the footing or 6 feet from existing grade, whichever is deeper.

Other Buildings Pads:

All the other building pads planned onsite should also be supported on engineered fill, a minimum of 3 feet below the footing or 5 feet from existing grade, whichever is deeper.

Pavement:

New pavement should be supported on engineered fill, a minimum of 2 feet below the proposed pavement section (i.e., below the aggregate base section).

Further subexcavation may be necessary depending on the conditions of the underlying soils. The actual depth of removal should be determined at the time of grading by the project geotechnical engineer. The determination will be based on soil conditions exposed within the excavations. At minimum, any undocumented fill, topsoil or other unsuitable materials should be removed and replaced as properly compacted fill.

In-place density tests may be taken in the removal bottom areas where appropriate to provide data to help support and document the engineer's decision.

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site, pre-job meeting with the developer, the contractor and the geotechnical engineer should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

Materials for Fill

On-site soils with an organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Soil material to be used as fill should not contain contaminated materials, rocks, or lumps over 6 inches in largest dimension, and not more than 40 percent larger than ¾ inch. Utility trench backfill material should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite.

Any imported fill material should consist of granular soil having a “low” expansion potential (that is, expansion index of 50 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Materials to be used as fill should be evaluated by a representative of NTS prior to importing or filling.

Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed excavation bottom by NTS. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of at least 8 inches and watered or dried, as needed, to achieve generally consistent moisture contents near optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Compacted fill should be placed in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted to a relative compaction of 90 percent as evaluated by ASTM D1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

Personnel from NTS should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

Temporary Excavations

Temporary excavations for the demolishing, earthwork, footing and utility trench are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 5 feet high will generally be stable; however, sloughing of cohesionless sandy materials encountered at the site should be expected.

Where the space is available, temporary, unsurcharged excavation sides over 5feet in height should be sloped no steeper than an inclination of 1H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within the imaginary zone of 1:1 as measured from the bottom of the excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. NTS should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces. Where space for sloped excavations is not available, temporary shoring may be utilized.

Personnel from NTS should observe the excavation so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

Slope Construction

Slopes should not be steeper than 2(h):1(v). Should steeper inclinations of slopes be required for the project, additional analysis may be warranted. Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than 5(h):1(v), and the depth of fill exceeds 5 feet, the existing slopes should be benched into competent bearing materials to provide a series of level benches to seat the fill and to remove potential loose soil or fill soil. The benches should be in accordance with the recommendations provided in the Grading section of this report.

In addition, a shear key should be constructed across the toe of fill slopes. The shear key should be a minimum of 15 feet wide and should penetrate a minimum of 2 feet beneath the toe of the slope into firm and competent bedrock.

Slope Protection

Inasmuch as the native materials are susceptible to erosion by wind and running water, it is our recommendation that the slopes at the project be planted as soon as possible after completion. The use of succulent ground covers, such as iceplant or sedum, is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the water system and to prevent over watering.

Measures should be provided to prevent surface water from flowing over slope faces.

Rodent infestation can also be a serious issue with respect to slope stability. Rodent tunneling and burrowing alters the strength of the soil and can allow water to infiltrate the soil, resulting in ultimate slope failure. Rodent burrows can also provide direct access for surface water to the slope face, causing surficial slope "blowouts". Although a maintenance issue, we recommend that measures be taken to prevent rodent infestation in slopes.

Slope Structural Setback

As per section 1808.7.2 of the 2019 CBC, which references Figure 1808.7.1 of the 2019 CBC, the distance between the face of the footing from the face of descending slopes should be at least the smaller of H/3 and 40 feet, where H is the height of the slope. Footings should be deepened as necessary to meet this requirement.

The distance between the face of the structure and the toe of ascending slopes should be at least the smaller of H/2 and 15 feet. The building should be setback from ascending slopes to meet this requirement.

Seismic Design

Our recommendations for seismic design parameters have been developed in accordance with 2019 CBC and ASCE 7-16 (ASCE, 2016) standards. The applicable site class is C based on the results of our field investigation. The table presents the seismic design parameters for the site that are obtained from USGS Design Ground Motions website and are based on the ASCE 7-16 and 2019 California Building Code.

2019 CBC and ASCE 7-16 Seismic Design Parameters

Seismic Item	Design Values[(a)]	2016 ASCE 7-16 or 2019 CBC Reference
Site Class based on soil profile (ASCE 7-16 Table 20.3-1)	C	ASCE 7-16 Table 20.3-1
Short Period Spectral Acceleration S_s	1.598	CBC Figures 1613.2.1 (1-8)
1-sec. Period Spectral Acceleration S_1	0.598	CBC Figures 1613.2.1 (1-8)
Site Coefficient F_a (2019 CBC Table 1613.2.3(1))	1.2	CBC Table 1613.2.3 (1)
Site Coefficient F_v (2019 CBC Table 1613.2.3(2))	1.402	CBC Table 1613.2.3 (2)
Short Period MCE* Spectral Acceleration S_{MS} $S_{MS} = F_a S_s$	1.917	CBC Equation 16-36
1-sec. Period MCE Spectral Acceleration S_{M1} $S_{M1} = F_v S_1$	0.838	CBC Equation 16-37
Short Period Design Spectral Acceleration S_{DS} $S_{DS} = 2/3S_{Ms}$	1.278	CBC Equation 16-38
1-sec. Period Design Spectral Acceleration S_{D1} $S_{D1} = 2/3S_{M1}$	0.559	CBC Equation 16-39
MCE[(b)] Peak Ground Acceleration (PGA)	0.699	ASCE 7-16 Figures 22-9 to 22-13
Site Coefficient F_{PGA} (ASCE 7-16 Table 11.8-1)	1.2	ASCE 7-16 Table 11.8-1
Modified MCE[(b)] Peak Ground Acceleration (PGA_M)	0.839	ASCE 7-16 Equation 11.8-1
Seismic Design Category	D	ASCE 7-16 Tables 11.6.1 and 11.6.2

(a) Design Values Obtained from USGS Earthquake Hazards Program website that are based on the ASCE-7-16 and 2019 CBC and site coordinates of N33.596774° and W117.226484°.

(b) MCE: Maximum Considered Earthquake.

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2019 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

Foundation Design and Construction

The proposed improvements may be safely founded on shallow foundations, either individual spread footings and/or continuous wall footings, bearing on competent engineered fill.

The footings should be designed in accordance with the following recommendations:

Foundation Design Parameters

Bearing Material	▪ Competent engineered fill ▪ See Grading section
Minimum Footing Size	▪ Width: 18 inches ▪ Embedment: A minimum of 24 inches
Allowable Bearing Capacity	▪ An allowable bearing capacity of 2,500 psf for the minimum footing size given above. ▪ The above value may be increased by 1/3 for temporary loads such as wind or earthquake.
Settlement	▪ Total static settlement is estimated to be 1 inch with differential settlement estimated to be approximately ½ inch over a span of 30 feet.
Allowable Lateral Passive Resistance	▪ 300 pcf (equivalent fluid pressure)
Allowable Coefficient of Friction	▪ 0.35

Slab-On-Grade Design and Construction

The slab-on-grade should be designed and constructed with the minimum recommendations presented below, however, final design of the slab should be determined by the project structural engineer.

Minimum Thickness: The minimum slab thickness should be 5 inches.

Minimum Slab Reinforcement: Minimum slab reinforcement shall not be less than No. 4 bars placed at 18 inches on center. Welded wire mesh is not recommended. Care should be taken to position the reinforcement bars in the center of the slab.

Slab Subgrade:

- The upper 24 inches of the slab subgrade should be moisture conditioned to near optimum moisture content and compacted to a minimum relative compaction of compacted to 90 percent relative compaction in accordance with the latest version of ASTM D1557 prior to placement of vapor retarder.

- A moisture vapor retarder should be placed direct blow the slab in accordance with the “Moisture Vapor Retarder” section below.

Moisture Vapor Retarder

A vapor retarder, such as a 10-mil-thick moisture vapor retarder that meets the requirements of ASTM E1745 Class C (Stego Wrap or equivalent) should be placed directly over the prepared soil subgrade to provide protection against vapor transmission through concrete floor slabs thatare anticipated to receive carpet, tile or other moisture sensitive coverings. The use of moisture vapor retarder should be determined by the project architect. At minimum, the vapor retarder should be installed as follows:

- Per the manufacture’s specifications as well as with the applicable recognized installation procedures such as ASTM E1643;
- Joints between the sheets and the openings for utility piping should be lapped and taped. If the barrier is not continuously placed across footings/ribs, the barrier should at minimum be lapped into the side of the footing/rib trenches down to the bottom of the trench; and,
- Punctures in the vapor retarder should be repaired prior to concrete placement.

It should be noted that the moisture retarder is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry in the building construction in Southern California. It is not intended to provide a “waterproof” or “vapor proof” barrier or reduce vapor transmission from sources above the retarder (i.e., concrete). The evaluation of water vapor from any source and its effect on any aspect of the proposed building space above the slab (i.e., floor covering applicability, mold growth, etc.) is beyond our purview and the scope of this report.

Pole Foundations

It is expected that the canopy structures and light poles will be supported on pole foundations. As a minimum, the pole foundations should be at least 18 inches in diameter and at least 4 feet deep; however, the actual dimensions should be determined by the project structural engineer based on the following design parameters.

Bearing Materials: The pole foundations may bear into competent bearing soils approved by a representative from NTS.

Bearing Values: End-bearing capacity may be combined to determine the allowable bearing capacities of the pole foundations. An allowable bearing pressure of 3,000 pounds per square foot (psf) may be used for pole foundations at least 18 inches in diameter and embedded a minimum of 4 feet below the lowest adjacent grade.

Lateral Load Design: Lateral loads may be resisted by passive resistance within the adjacent earth materials. For passive resistance, an allowable passive earth pressure of 300 pounds per foot of pile diameter per foot of depth into competent bearing material may be used; however, passive resistance should be disregarded within the upper foot due to possible disturbance during drilling. The passive resistance value may be applied over an area equivalent to two pile diameters.

Retaining Walls Design

The following design parameters may be utilized by the structural engineer to design the proposed retaining walls at the subject project. We understand that the retaining walls could retain over 6 feet of soil.

Foundation Recommendations

Retaining walls foundation may be sized based on the recommendations presented in "Foundation Design and Construction" section of this report.

Lateral Earth Pressure

The values presented below assume that the supported grade is level and that surcharge loads are not applied. In addition, the recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the retaining walls and that external hydrostatic pressure will not develop. Where adequate drainage is not provided behind the walls, further evaluation should be conducted by the project geotechnical engineer and the lateral earth pressures will need to be adjusted accordingly.

Unrestrained Wall:	40 pcf for level backfill
Unrestrained Wall	60 pcf for 2:1 backfill

The unrestrained values are applicable only when the walls are designed and constructed as cantilevered walls allowing sufficient wall movement to mobilize "active" pressure conditions. This wall movement should be less than 0.01 H (H = height of wall) for the unrestrained values to be applicable.

Vertical surcharge loads within 1:1 project from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressure acting on the wall.

Seismic Earth Pressure

Given the general seismicity and the fact that the basement walls are retaining more than 6 feet of earth, it is recommended that the walls also be designed for seismic earth pressure. The seismic earth pressure distribution may be considered to be a triangle with the maximum pressure at the bottom. The total seismic earth pressure may be represented by an equivalent fluid pressure (EFP) of 25 pcf.

Drainage

The backdrain system should consist of 4-inch perforated pipe surrounded by at least one cubic foot of ¾” – 1.5” open graded gravel wrapped in Mirafi 140N fabric or equivalent. The perforated pipe should consist of SDR-35 or Schedule 40 PVC pipe or approved equivalent laid on at least 2 inch of crushed rock with the perforations laid down. The back drain gradient should not be less than 1 percent. The perforated pipe should outlet into area drains or other suitable outlet points at runs 200 feet or less, if practical. If the back drains cannot be outleted by gravity flow, a sump pump system will need to be designed and constructed. Redundant back-up pumps or components are recommended. Design of the system is outside of the purview of NTS.

Waterproofing

The back side of the retaining walls should be waterproofed prior to placement of subdrains or backfill. Waterproofing is outside of our purview and should be designed by a waterproofing consultant.

Wall Backfill

Backfill behind the wall may consist of onsite soil granular fill material approved by NTS. If select backfill is used, then all select backfill within 2 feet of final grade should consist of free-draining granular material (i.e., SE 30 sand or crushed rock). Crushed rock, if used, should be completely wrapped in filter fabric (Mirafi 140N or equivalent) to minimize the potential for migration of fines into the rock. The select backfill should be moisture conditioned to near optimum moisture content and compacted to achieve at least 90 percent relative compaction in accordance with ASTM D1557. The upper two feet of backfill should consist of fine-grained native soils, moisture conditioned to 4 percent above optimum moisture content and compacted to 90 percent relative compaction in order to cap the select backfill zone.

The select backfill should extend horizontally a minimum of H/2 behind the wall, where H is the retained height.

Utility Trench Backfill Considerations

New utility line pipeline trenches should be backfilled with select bedding materials beneath and around the pipes (pipe zone) and compacted soil above the pipe bedding. Recommendations for the types of the materials to be used and the proper placement of these materials are provided in the following sections.

Pipe Zone (Bedding and Shading)

The pipe bedding and shading materials should extend from at least 6 inches below the pipes to at least 12 inches above the crown of the pipes. Pipe bedding and shading should consist of either clean sand with a sand equivalent (SE) of at least 30, or crushed rock. If crushed rock is used, it should consist of ¾-inch crushed rock that conforms to Table 200-1.2.1 (A) of the 2018 "Greenbook." Pipe bedding and shading should also meet the minimum requirements of the City of Los Angeles. If the requirements of the City are more stringent, they should take precedence over the geotechnical recommendations. Sufficient laboratory testing should be performed to verify the bedding and shading meets the minimum requirements of the Greenbook and City of Wildomar grading codes.

Granular pipe bedding and shading material should be properly placed in thicknesses not exceeding 3 feet, and then sufficiently flooded or jetted in place. Crushed rock, if used, should be capped with filter fabric (Mirafi 160N, or equivalent; Mirafi 140N filter fabric is suitable if available) to prevent the migration of fines into the rock.

Trench Backfill

All existing soil material within the limits of the site are considered suitable for use as trench backfill above the pipe bedding and shading zone if care is taken to remove all significant organic and other decomposable debris, moisture condition the soil materials as necessary, and separate and selectively place and/or stockpile any inert materials larger than 6 inches in maximum diameter.

Imported soils are not anticipated for backfill since the on-site soils are suitable. However, if imported soils are used, the soils should consist of clean, granular materials with physical and chemical characteristics similar to or better than those described herein for on-site soils. Any imported soils to be used as backfill should be evaluated and approved by NTS prior to placement.

Soils to be used as trench backfill should be moistened, dried, or blended as necessary to achieve near optimum moisture content, placed in lifts which, prior to compaction shall not exceed the thickness specified in Section 306-12.3 of the 2018 "Greenbook" for various types of equipment, and mechanically compacted/densified to at least 90 percent relative compaction as determined by ASTM Test Method D 1557. Jetting is not permitted in this trench zone.

No rock or broken concrete greater than 6 inches in maximum diameter should be utilized in the trench backfills.

Asphalt Concrete Pavement Design

In accordance with Chapter 600 of the Caltrans Highway Design Manual, we have performed pavement structural design utilizing assumed traffic indices (TI) of 4.5, 5.5 and 7 and assumed R-value of 30. Based on our analysis, we have developed the pavement structural sections presented in the following table. We note that the assumed TI's should be reviewed by a traffic engineer to confirm their applicability to the project. The assumed R-value should be confirmed by testing at the completion of rough grading.

Asphalt Concrete Pavement Structural Sections

Location	Traffic Index	Asphalt Concrete (in.)	Aggregate Base (in.)*
Parking Stalls	4.5	3.0	4.0
Driveways	5.5	4.0	5.0
Fire lane	7.0	4.0	10.0

The planned pavement structural sections should consist of the following:

- Aggregate Base materials (AB) consisted of either Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB).
- Asphalt Concrete (AC) material of a type meeting the minimum City of Wildomar standards.
- The subgrade soils should be moisture conditioned to 2 percent above optimum moisture content to a depth of at least 18 inches and compacted to 90 percent relative compaction.
- The AB and AC should be compacted to at least 95 percent relative compaction.

Exterior Flatwork/Hardscape Design Considerations

For exterior flatwork and hardscape planned as part of the proposed development, the following design may be considered by the project civil engineer. These recommendations may be considered as minimal design based on the soils conditions encountered during our investigation. Final design of the proposed flatwork and hardscape area should be provided by the project civil engineer. Based on the conditions encountered, we recommend that the subgrade for the subject concrete flatwork and hardscape be moisture conditioned to near optimum to a depth of 18 inches below finish subgrade elevation and compacted to 90 percent relative compaction. A Type II/V cement may be used from a geotechnical perspective. Our flatwork and hardscape design considerations are presented in the table below.

Concrete Flatwork Table

Description	Subgrade Preparation (1)	Minimum Concrete Thickness	Cut-Off Barrier Or Edge Thickness	Joint Spacing (Maximum)	Concrete(3)
Concrete Sidewalks and Walkways (4)	1) 2 percent above optimum to 18"(1), 2) 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	4 inches	Not Required	5 feet	Type II/V

(1) The moisture content of the subgrade must be verified by the geotechnical consultant prior to sand/rock placement.
(2) Reinforcement to be placed at or above the mid-point of the slab (i.e., a minimum of 2.0 to 2.5 inches above the prepared subgrade).
(3) The site has negligible levels of sulfates as defined by the CBC. Concrete mix design is outside the geotechnical engineer's purview.
(4) Where flatwork is adjacent a stucco surface, a ¼" to ½" foam separation/expansion joint should be used.
(5) If dowels are placed in cored holes, the core holes shall be placed at alternating in-plane angles (i.e., not cored straight into slab).

Planters and Trees

Where new trees or large shrubs are to be located in close proximity to new concrete flatwork, rigid moisture/root barriers should be placed around the perimeter of the flatwork to at least 12 inches in depth in order to offer protection to the adjacent flatwork against potential root and moisture damage. Existing mature trees near flatwork areas should also incorporate a rigid moisture/root barrier placed at least 2 feet in depth below the top of the flatwork.

Drainage Control

The control of surface water is essential to the satisfactory performance of the site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 2 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked.

Review, Observation, and Testing

The recommendations presented in this report are contingent upon review of final plans and specifications for the project by NTS. NTS Geotechnical, Inc. should review and verify in writing the compliance of the final grading plan and the final foundation plans with the recommendations presented in this report.

It is recommended that NTS be retained to provide Geotechnical Consulting services during the earthwork operations and foundation installation process. This is to observe compliance with the design concepts, specifications and recommendations and to allow for design changes in the event that subsurface conditions differ from those anticipated during our subsurface investigation.

It is the responsibility of the owner and their representative to bring any deviations or unexpected conditions observed during construction to the attention of NTS Geotechnical, in order for supplemental recommendations can be made with a minimum delay to the project. Construction should be observed and/or testing at the following stages by NTS Geotechnical, Inc.:

- Continuous observation during the excavation and recompaction for the structures.
- During preparation of subgrade for slab-on-grade.
- During installation of foundations.
- Installation of drainage for new retaining walls.
- Backfill of retaining walls.
- Grading for new parking lots.
- Testing of aggregate base for the new parking lot.
- Testing of asphalt concrete for the new parking lot.
- Backfill of utility trenches.
- When unusual conditions are encountered.

If any of these inspections to verify site geotechnical conditions are not performed by NTS Geotechnical, liability for the safety and stability of the project is limited only to the actual portions of the project that is observed and approved by NTS Geotechnical.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgments. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and site construction will be identical to those observed, sampled, and interpreted during our study, or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during construction and grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report. Since our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project.

It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

This report has not been prepared for the use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

REFERENCES

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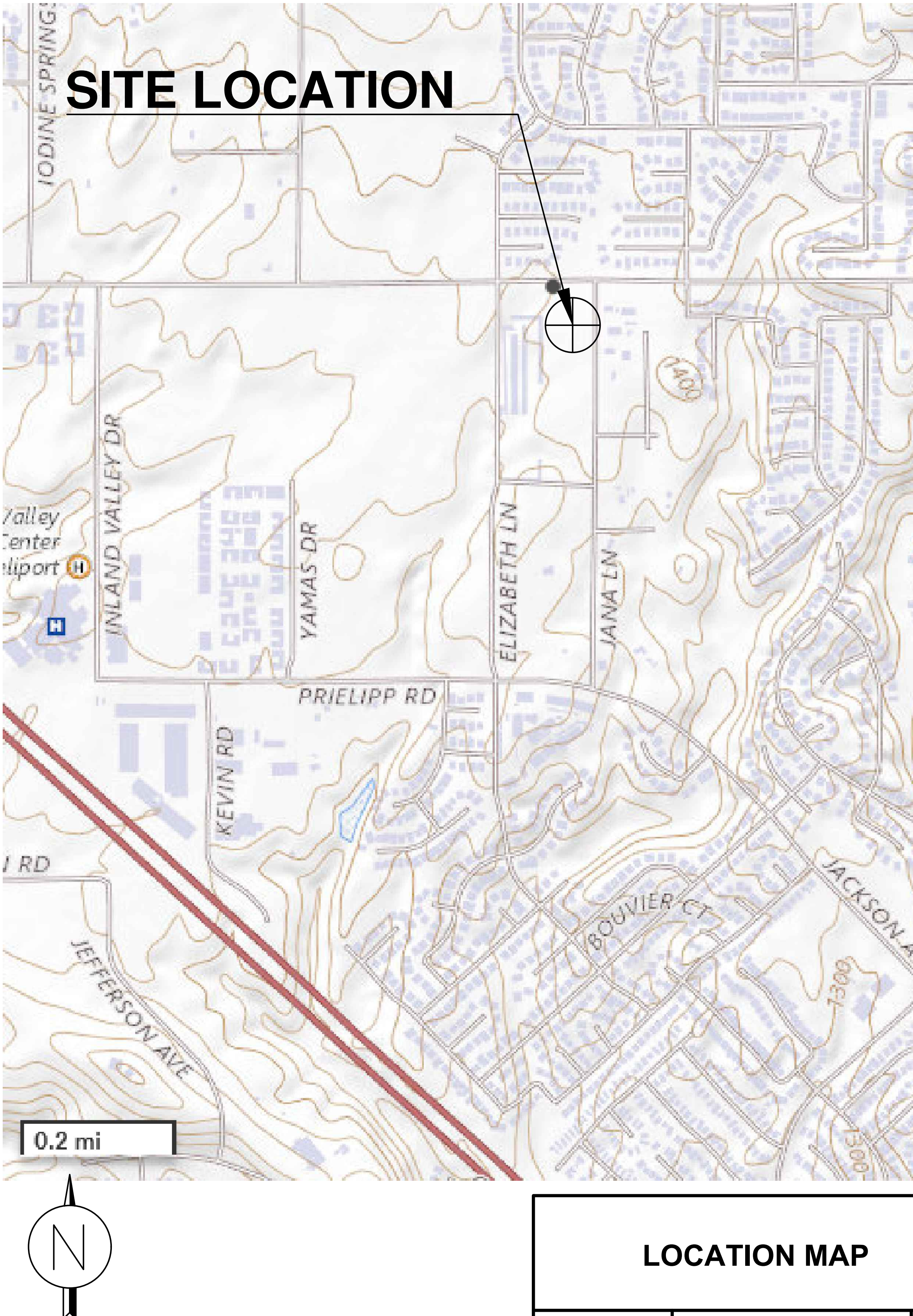
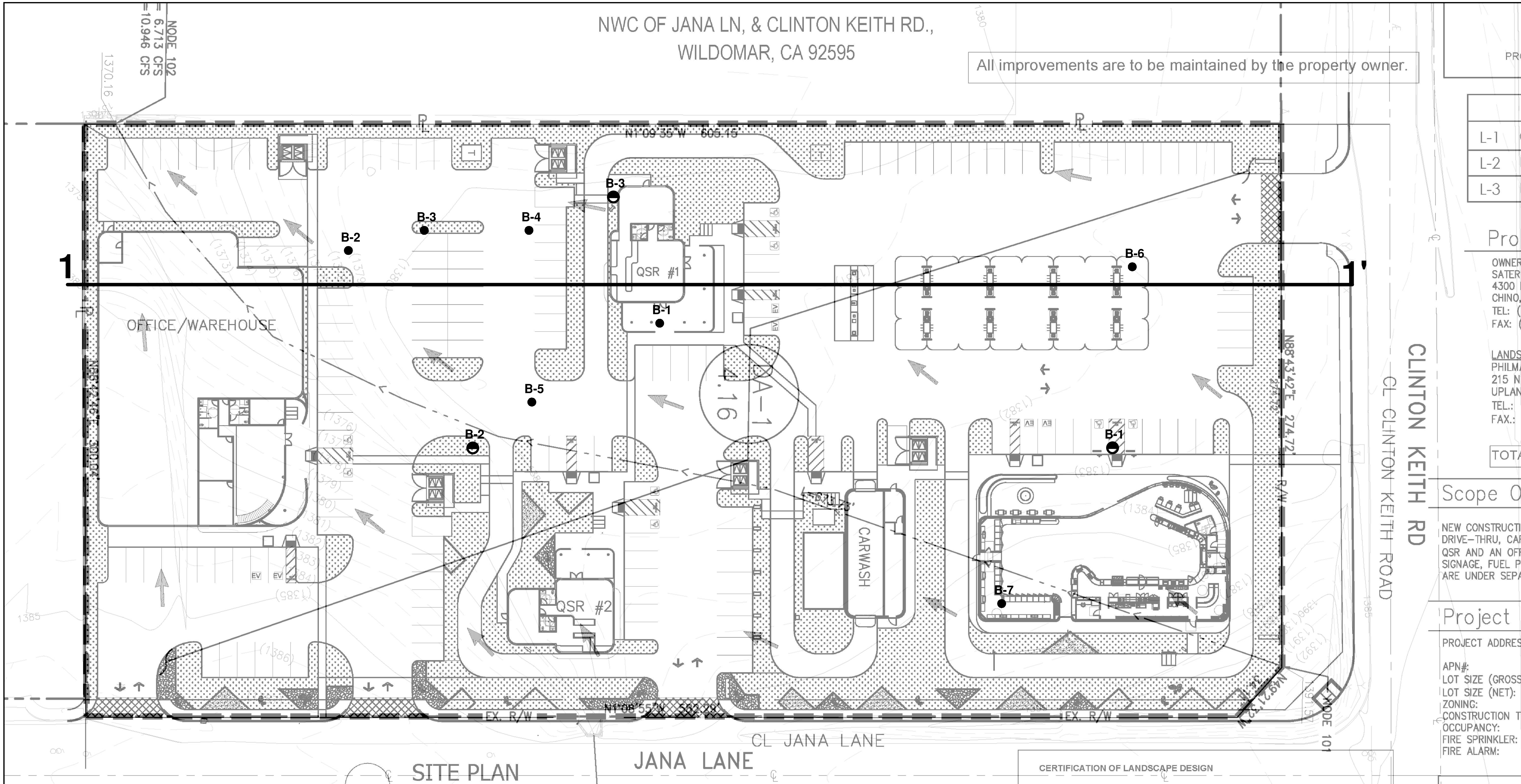
United States Geological Survey (USGS), 2008, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (update) (v4.2.0), Retrieved May 14, 2020, from: https://earthquake.usgs.gov/hazards/interactive/



LOCATION MAP

NTS GEOTECHNICAL	Date: October 17, 2022	Plate 1
	Project No.: 22504	



NTS GEOTECHNICAL

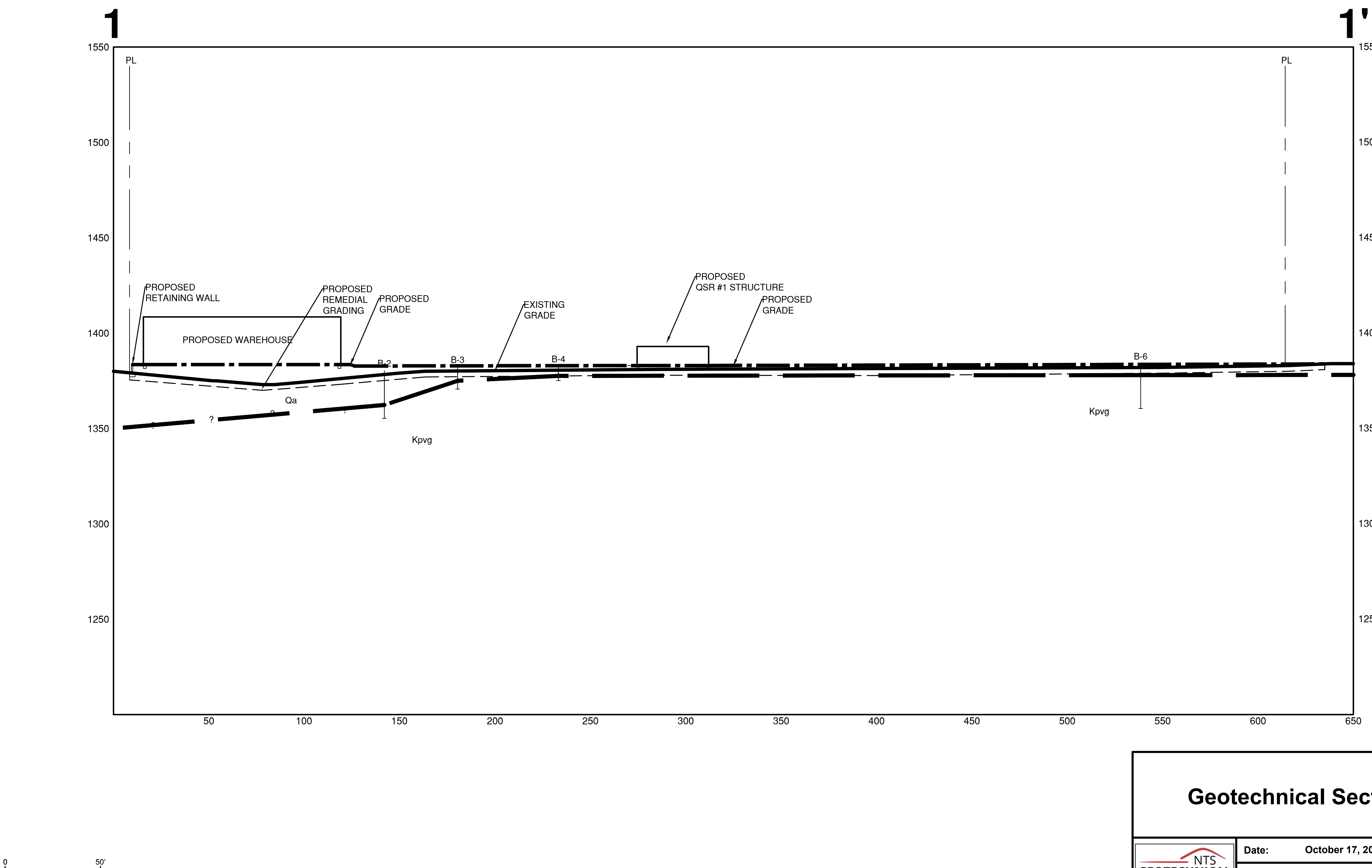
GEOTECHNICAL LEGEND

B-1 ●	APPROXIMATE LOCATION OF BORINGS BY GEO-CAL
B-1 ◒	APPROXIMATE LOCATION OF BORINGS BY SOIL EXPLORATION
1 —— 1'	GEOTECHNICAL SECTION

0 50'

GEOTECHNICAL MAP

Date: 10/17/2022	Project No.: 22504	Plate: 2



DRAWING: c:\users\info\nts geotechnical\projects - general\2022\22504 - swc jana ln & clinton keith rd, wildomar\reports\plates\plate 3 - geotechnical section.dwg PLOTTED: 10/17/2022 9:26 PM BY: Nadim Sunna

Geotechnical Section

NTS GEOTECHNICAL

Date:	October 17, 2022	Plate 3
Project No.:	22504	

APPENDIX A

Field Exploration

GEOTECHNICAL BORING LOGS

Drill Hole No. B-1

Date: 2/25/21 **Project No.** 20237-01
Drilling Company: Larry Harklerode **Type of Rig:** B-53
Hole Diameter: 8" **Drive Weight:** 140 lbs. **Drop:** 30" **Elevation:** Existing Ground

DEPTH (feet)	TYPE OF TEST	SAMPLE TEST	BLOWS PER 6 INCH	DRY DENSITY (%)	MOISTURE (%)	SOIL CLASSIFICATION USCS	GEOTECHNICAL DESCRIPTION LOGGED BY: GL SAMPLED BY: GL
1	Alluvium					SM	**SILTY SAND:** Light brown, fine to medium grained, slightly moist, dense
2							
3							
4		X	11/18/18		9.6		
5	Bedrock					Kpvg	**MONZOGRANITE:** Pale brown, fine to medium grained, medium dense
6		X	25/21/35		7.4		Pale gray, very dense
7							
8							
9							
10							
11		X	17/43/50/ 4"				Very dense
12							
13							
14							
15							
16		X	35/50/4"				
17							
18							
19							
20							
21		X	38/50/4"				Very dense
22							
23							
24							
25		X	48/50/3"				Very dense

TOTAL DEPTH = 25'
NO GROUNDWATER
NO CAVING
BORING BACKFILLED

GEOTECHNICAL BORING LOGS

Drill Hole No. B-2

Date: 2/25/21
Drilling Company: Larry Harklerode
Hole Diameter: 8" **Drive Weight:** 140 lbs. **Drop:** 30"

Project No. 20237-01
Type of Rig: B-53
Elevation: Existing Ground

DEPTH (feet)	TYPE OF TEST	SAMPLE TEST	BLOWS PER 6 INCH	DRY DENSITY (%)	MOISTURE (%)	SOIL CLASSIFICATION USCS	GEOTECHNICAL DESCRIPTION LOGGED BY: GL SAMPLED BY: GL
1	Alluvium					SM	**SILTY SAND:** Light brown, fine to medium grained, slightly moist, dense
2	Bedrock					Kpvg	**MONZOGRANITE:** Pale gray, fine to medium grained, very dense
3		X	50/4"		6.8		
4							
5							
6		X	50/1"		2.8		Very dense
7							
8							
9							
10							
11		X	50/3.5"				
12							
13							
14							
15							
16		X	50/2"				Very dense
17							
18							
19							
20							
21		X	50/3"				
22							
23							
24							
25		X	50/3"				Very dense

TOTAL DEPTH = 25'
NO GROUNDWATER
NO CAVING
BORING BACKFILLED

GEOTECHNICAL BORING LOGS

Drill Hole No. B-3

Date: 2/25/21 **Project No.** 20237-01
Drilling Company: Larry Harklerode **Type of Rig:** B-53
Hole Diameter: 8" **Drive Weight:** 140 lbs. **Drop:** 30" **Elevation:** Existing Ground

DEPTH (feet)	TYPE OF TEST	SAMPLE TEST	BLOWS PER 6 INCH	DRY DENSITY (%)	MOISTURE (%)	SOIL CLASSIFICATION USCS	GEOTECHNICAL DESCRIPTION LOGGED BY: GL SAMPLED BY: GL
1	Alluvium					SM	**SILTY SAND:** Light brown, fine to medium grained, dry, very dense
2							
3			30/50/4"	95.4	12.1		Slightly moist, very dense
4							
5							
6	Bedrock		35/50/2.5"		6.3	Kpvg	**MONZOGRANITE:** yellow, fine to medium grained, very dense
7							
8							
9							
10							
11			28/25/32				Pale gray, fine to medium grained, very dense
12							
13							
14							
15			35/50/4"				Very dense
16							TOTAL DEPTH = 15' NO GROUNDWATER NO CAVING BORING BACKFILLED
17							
18							
19							
20							
21							
22							
23							
24							
25							

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LOG OF BORING B-1

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date: **6-23-21.**
Drilled By: **Cal-Pac Drilling**
Equipment: **Mobil B-61**
Hole Size: **6″ HSA**
Logged By: **Todd Wyland, RCE**

Total Depth: **10.5 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		4.9	MDC SA			**(SM) Silty Sand, fine to medium, traces coarse and subrounded gravel to 3/4″, light red brown**
5	1-1	R	37 50/5″	4.3	128			**(SM) Silty Sand, fine to medium, traces coarse and subrounded gravel to 3/4″, light red brown, very dense**
								<u>Weathered Bedrock</u> recovered as (SW-SM) Sand, fine to coarse with silt, gray.
10	1-2	S	50/6″	4.6				**Very dense**
								End of Boring
15								**Total Depth 10.5 ft.** **No Groundwater** **No Fill.** **No Refusal** **Boring Backfilled w/ drill spoils** **Weathered Bedrock at Approximately 7 ft.**
20								
25								
30								

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LOG OF BORING B-2

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date: **6-23-21.**
Drilled By: **Cal-Pac Drilling**
Equipment: **Mobil B-61**
Hole Size: **6″ HSA**
Logged By: **Todd Wyland, RCE**

Total Depth: **23 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								Fill
	2A (0′-5′)	B		4.9		SA MDC		**(SM) Silty Sand, fine to medium, trace coarse 3/8" and light brown red.**
5	2-1	R	7 17 24	8.7	120			**(SM) Silty Sand, fine to medium, light brown dense.**
10	2-2	S	5 11 16	11.1				**(SM) Silty Sand, fine, trace medium, mica, dark to down, medium dense.**
15	2-3	S	8 9 16	13.2				**(SM) Silty Sand, fine, trace medium, mica, dark to down, medium dense.**
								Weathered Bedrock
20		S	-					**Lost Sampler**
	2-4	S	50/0"					**No Recovery, very dense**
								End of Boring
25								**Refusal @ 23′**
								Fill to 1 ft (estimated) **No Groundwater** **Weathered Bedrock estimated at 16 ft** **Boring Backfilled w/Drill Spoils**
30								

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LOG OF BORING B-3
(PB-1)

(Page 1 of 1)

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date: 6-23-21.
Drilled By: Cal-Pac Drilling
Equipment: Mobil B-61
Hole Size: 8″ HSA
Logged By: Todd Wyland, RCE

Total Depth: 9.5 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	3A (0′-5′)	B		4.9				(SM) Silty Sand, fine to medium, traces coarse and gravel 1/2", brown
5								Weathered Bedrock
	3-1	R	9 15 18	5.1 5.5	114 118	SA SA		(SW) Well Graded Sand, fine to coarse, trace gravel to 3/8", gray, medium dense End of Boring
10								Total Depth 9.5 ft. 3″-Diameter perc pipe packed with gravel to the surface No Fill No groundwater No Refusal
15								
20								
25								
30								

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LOG OF BORING B-4
(PB-2)

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date:	6-23-21.	Total Depth:	5.5 ft
Drilled By:	Cal-Pac Drilling	Groundwater Depth:	Not Encountered
Equipment:	Mobil B-61		
Hole Size:	6″ HSA		
Logged By:	Todd Wyland, RCE		

*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	4A (0′-5′)	B		4.8				(SM) Silty Sand, fine to medium, brown
								Weathered Bedrock
5	4-1	R	20 50/3″	5.8	129.5	SA		(SW-SM) Well graded Sand, fine to coarse with silt, very dense
								at 5.5 ft white and less weathered
								End of Boring
								Total Depth 5.5 ft.
10								No Fill, No Groundwater, No Refusal 3″-Diameter perc pipe packed with gravel to the surface Weathered Bedrock at 3ft.(estimated) Persistent Bedrock at 5.5 ft.
15								
20								
25								
30								

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LOG OF BORING B-5
(PB-3)

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date:	6-23-21.	Total Depth:	6.5 ft
Drilled By:	Cal-Pac Drilling	Groundwater Depth:	Not Encountered
Equipment:	Mobil B-61		
Hole Size:	6″ HSA		
Logged By:	Todd Wyland, RCE		

*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								(SM) Silty Sand, fine to medium, brown
	5A (0′-4′)	B		5.5		COR		
5	5-1	R	16 25 31	4.8	134	SA		Weathered Bedrock (SW-SM) Well Graded Sand, fine to coarse with silt and gravel to 3/8″, dark gray, dense End of Boring
10								Total Depth 6.5 ft. 3″-Diameter perc pipe packed with gravel to the surface No Fill No Groundwater No Refusal Weathered Bedrock at 4 ft. (estimated)
15								
20								
25								
30								

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-6

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date:	6-23-21.	Total Depth:	21.5 ft
Drilled By:	Cal-Pac Drilling	Groundwater Depth:	Not Encountered
Equipment:	Mobil B-61		
Hole Size:	6″ HSA		
Logged By:	Todd Wyland, RCE		

*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	6-A (0'-5')	B		5.6				(SM) Silty Sand, fine to medium trace, gravel to 3/4', brown
5	6-1	R	8 18 24	4.1				Weathered Bedrock (SM) Silty Sand, fine to medium, mica, dark gray, dense
10	6-2	S	11 16 18	6.8	126			(SM) Silty Sand, fine to medium, mica, dark gray, dense
15	6-3	S	24 50/6"	4.8				(SM) Silty Sand, fine to medium, mica, dark gray, very dense
20	6-4	S	9 20 38	3.4				(SP-SM) Sand, fine to medium, with silt, gray, very dense End of Boring
25								Total Depth 21.5 ft. No Fill No Groundwater No Refusal Weathered Bedrock at 4 ft(Estimated) Boring Backfilled w/ drill spoils
30								

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LOG OF BORING B-7

Project:
24831 Clinton Keith Rd.
Wildomar, CA

Date:	6-23-21.	Total Depth:	16 ft
Drilled By:	Cal-Pac Drilling	Groundwater Depth:	Not Encountered
Equipment:	Mobil B-61		
Hole Size:	6″ HSA		
Logged By:	Todd Wyland, RCE		

*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								
	7A (0′-5′)							(SM) Silty Sand, fine to medium, light red brown, medium dense
5	7-1	S	5 9 25	4.2				Weathered Bedrock reducing to (SW) Sand, fine to coarse, gray, dense
10	7-2	S	32 50/3″					(SW) Sand, fine to coarse, gray, very dense
15	7-3	S	26 50/6″					(SW) Sand, fine to coarse, gray, very dense
								End of Boring
20								Total Depth 16 ft. No Groundwater No Fill Weathered Bedrock @ 6′ No Refusal Boring Backfilled with Drill Spoils
25								
30								

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Exploratory Trench No. T-1

Project:
New Development of A Beyond Market w/Gas Station & Drive-Thru, Plus a Carwash

Location:
24831 Clinton Keith Rd.
Wildomar, CA

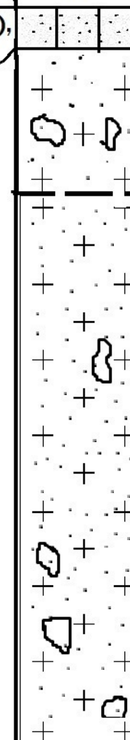
Excavation Method: Mini Excavator w/ 12" bucket

Logged By: Henry Olivier, P.G

Date: 6/23/2021

Total Depth: 9.5 Ft.

Groundwater: Not Encountered

Depth (Ft.)	Description	Graphic
	Top Soil: Silty Sand (SM), fine grained, relatively loose, brown, gravelley, random cobbles (6"-diameter), dry, with rootlets.	
	Weathered Rock (DG), possibly weathered Granodiorite, medium to coarse grain size, dense, relatively friable, gray, dry.	
	Same, but more dense, less weathered, less friable, with caliche stingers	
5'		
	Digging refusal at 9.5 ft bgs using mini excavator	
10'		
15'		

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Exploratory Trench No. T-2

Project: New Development of A Beyond Market w/Gas Station & Drive-Thru, Plus a Carwash	**Excavation Method:** Mini Excavator w/ 12" bucket	**Date:** 6/23/2021
	Logged By: Henry Olivier, P.G	**Total Depth:** 4 Ft.
Location: 24831 Clinton Keith Rd. Wildomar, CA		**Groundwater:** Not Encountered

Depth (Ft.)	Description	Graphic
	Top Soil: Silty Sand (SM) with sparse cobbles (≥ 6"), loose to medium dense, light brown, dry, w/ rootlets	
	Weathered Rock (DG), possibly weathered Granodiorite, medium to coarse grain, dense, dry	
	Refusal at 4 ft.	
5'		
10'		
15'		

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Exploratory Trench No. T-3

Project: New Development of A Beyond Market w/Gas Station & Drive-Thru, Plus a Carwash	Excavation Method: Mini Excavator w/ 12" bucket	Date: 6/23/2021
Location: 24831 Clinton Keith Rd. Wildomar, CA	Logged By: Henry Olivier, P.G	Total Depth: 4 Ft.
		Groundwater: Not Encountered

Depth (Ft.)	Description	Graphic
	Top Soil: Silty Sand (SM) with sparse cobbles (≥ 6"), loose to medium dense, light brown, dry, w/ rootlets	
	Weathered Rock (DG), possibly weathered Granodiorite, medium to coarse grain, dense, dry	
	Refusal at 4 ft.	
5'		
10'		
15'		

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Exploratory Trench No. T-4

Project:
New Development of A Beyond Market w/Gas Station & Drive-Thru, Plus a Carwash

Location:
24831 Clinton Keith Rd.
Wildomar, CA

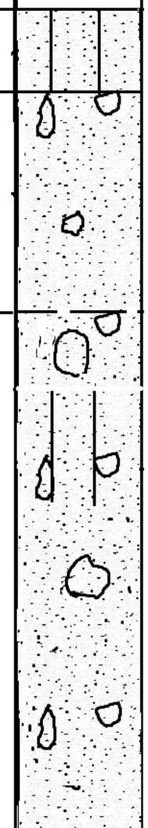
Excavation Method: Mini Excavator w/ 12" bucket

Logged By: Henry Olivier, P.G

Date: 6/23/2021

Total Depth: 10.5 Ft.

Groundwater: Not Encountered

Depth (Ft.)	Description	Graphic
	Top Soil: Silty Sand (SM) with sparse cobbles (≥ 6"), loose , light brown, dry, w/rootlets	
	Gravelly Sand (SP), fine to medium, with random coarse gravel and cobbles (≥ 6"), dense, brown, dry	
5'	Silty Sand (SM), with gravel and sparse cobbles, brown, dense, dry	
10'		
	Digging Refusal at 10.5 ft bgs using mini excavator	
15'		

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APPENDIX B

Geotechnical Laboratory Testing

Enviro – Chem, Inc.
1214 E. Lexington Avenue, Pomona, CA 91766 Tel (909) 590-5905 Fax (909) 590-5907

LABORATORY REPORT

CUSTOMER: **Soil Exploration Company**
7535 Jurupa Ave., Suite C
Riverside, CA 92504
Tel:(909)374-5429 **E-Mail: SoilExploration@yahoo.com**

PROJECT: **Beyond Food Mart / 20237-01**
MATRIX: SOIL
SAMPLING DATE: 02/25/21
REPORT TO: Mr. GENE K. LUU
DATE RECEIVED: 03/03/21
DATE ANALYZED: 03/03&04/21
DATE REPORTED: 03/08/21

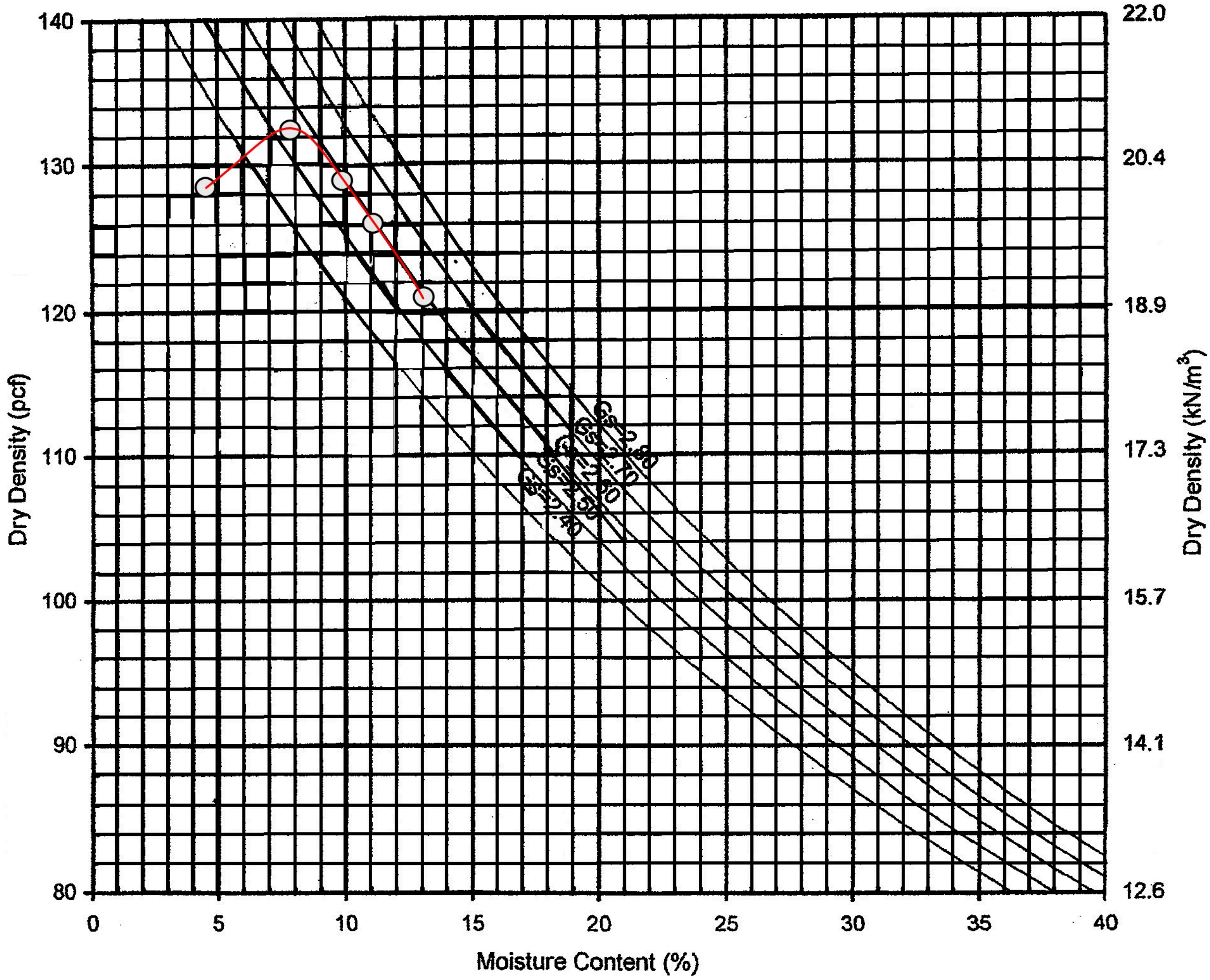
SAMPLE I.D.: **B-1 @ 0-5'**
LAB I.D.: 210303-2

PARAMETER	SAMPLE RESULT	UNIT	PQL	DF	TEST METHOD
RESISTIVITY	11100	OHMS-CM	100000*	--	CALTRANS
SULFATE	13.3	mg/Kg	10	1	EPA 9038
CHLORIDE	20.0	mg/Kg	10	1	EPA 9253
pH	7.45	pH/UNIT	--	--	EPA 9045C

COMMENTS
DF = DILUTION FACTOR
PQL = PRACTICAL QUANTITATION LIMIT
ACTUAL DETECTION LIMIT = DF X PQL
mg/Kg = MILLIGRAM PER KILOGRAM = PPM
OHMS-CM = OHMS-CENTIMETER
RESISTIVITY = 1/CONDUCTIVITY
* = HIGH LIMIT
pH ANALYSIS CONDUCTED ON 1;1 SOIL/DEIONIZED WATER EXTRACTION

DATA REVIEWED AND APPROVED BY: ________
CAL-DHS ELAP CERTIFICATE No.: 1555

MOISTURE DENSITY CURVE



Boring	Depth(ft)	Classification	γ_{max} (pcf)	w_{opt} (%)
B-1 + B-2	0'-5'	(SM) Silty Sand, Fine to medium, trace coarse	132	8.0

MOISTURE DENSITY CURVE (MDC) ASTM D 1557	
Project:	24831 Clinton Keith Rd
Location:	Wildomar, CA

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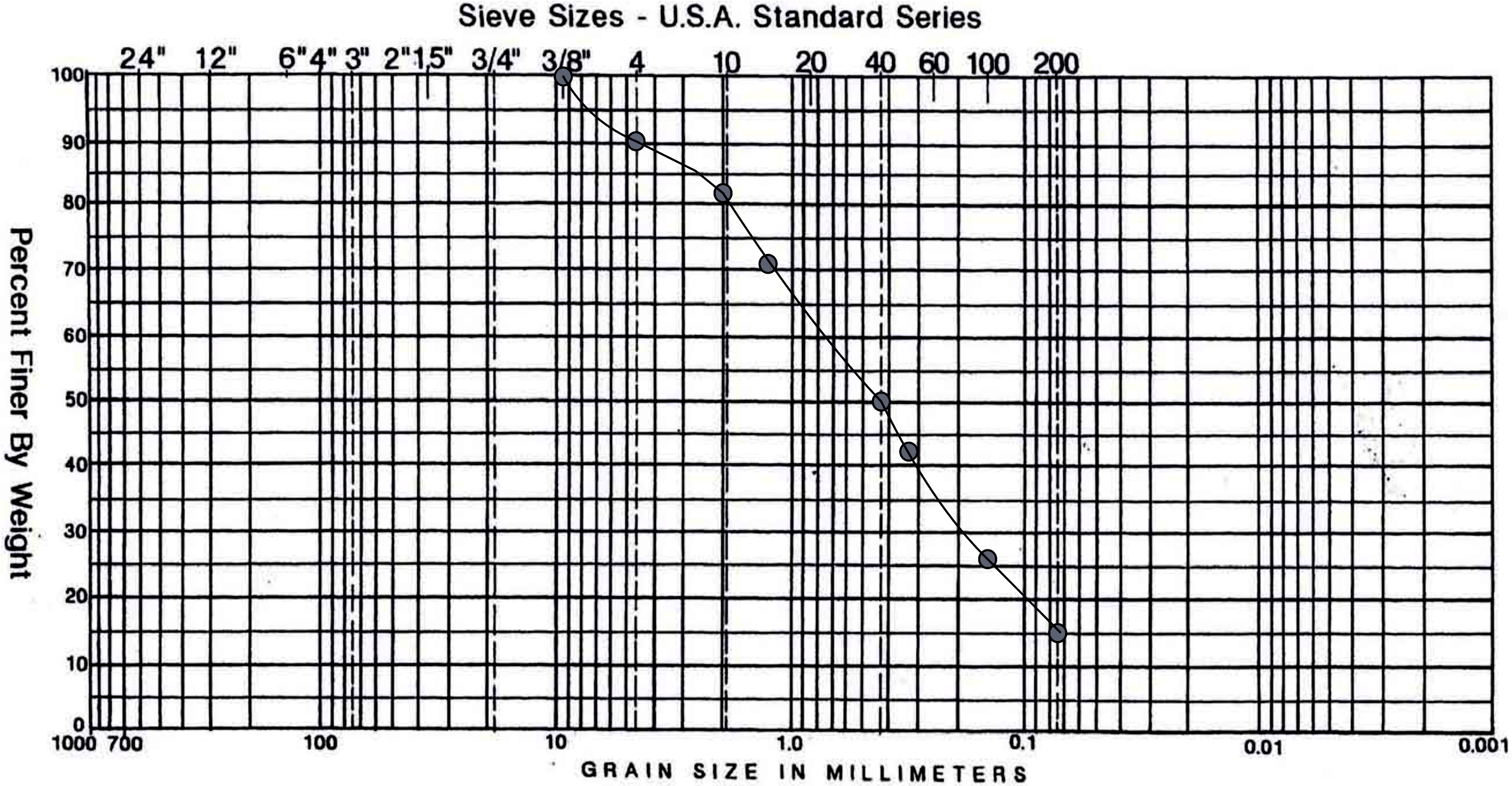
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-1 + B-2	0'-5'	(SM) Silty Sand, fine to medium, traces coarse and gravel to 3/8"
			15% fines

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Wildomar, CA

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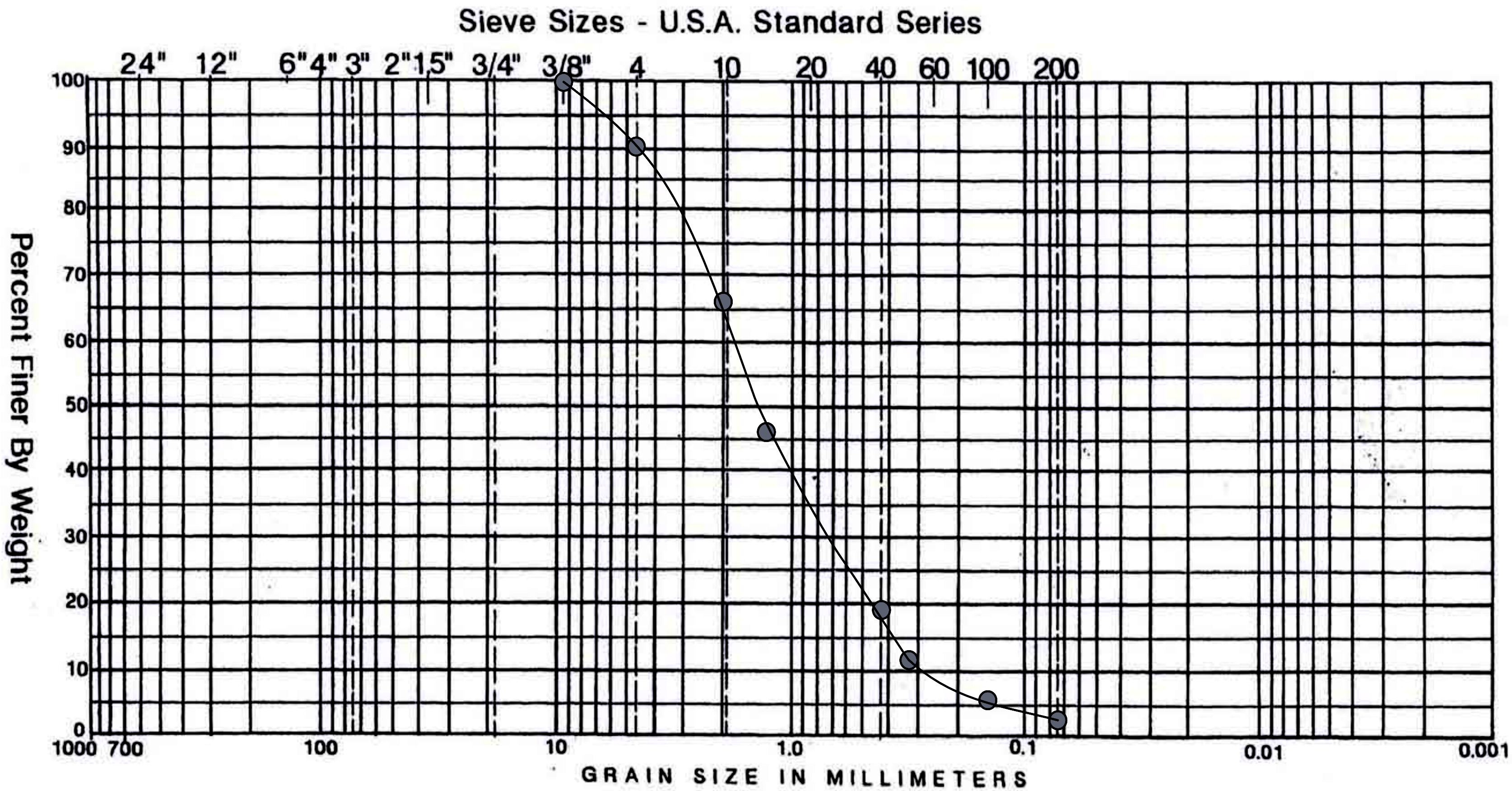
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-3 (PB-1)	8.5'-9.0'	(SW) Well graded Sand, fine to coarse, trace gravel to 3/8″ 2% fines,
			D10=0.29mm,
			Note: Sample Scalped at 3/8′

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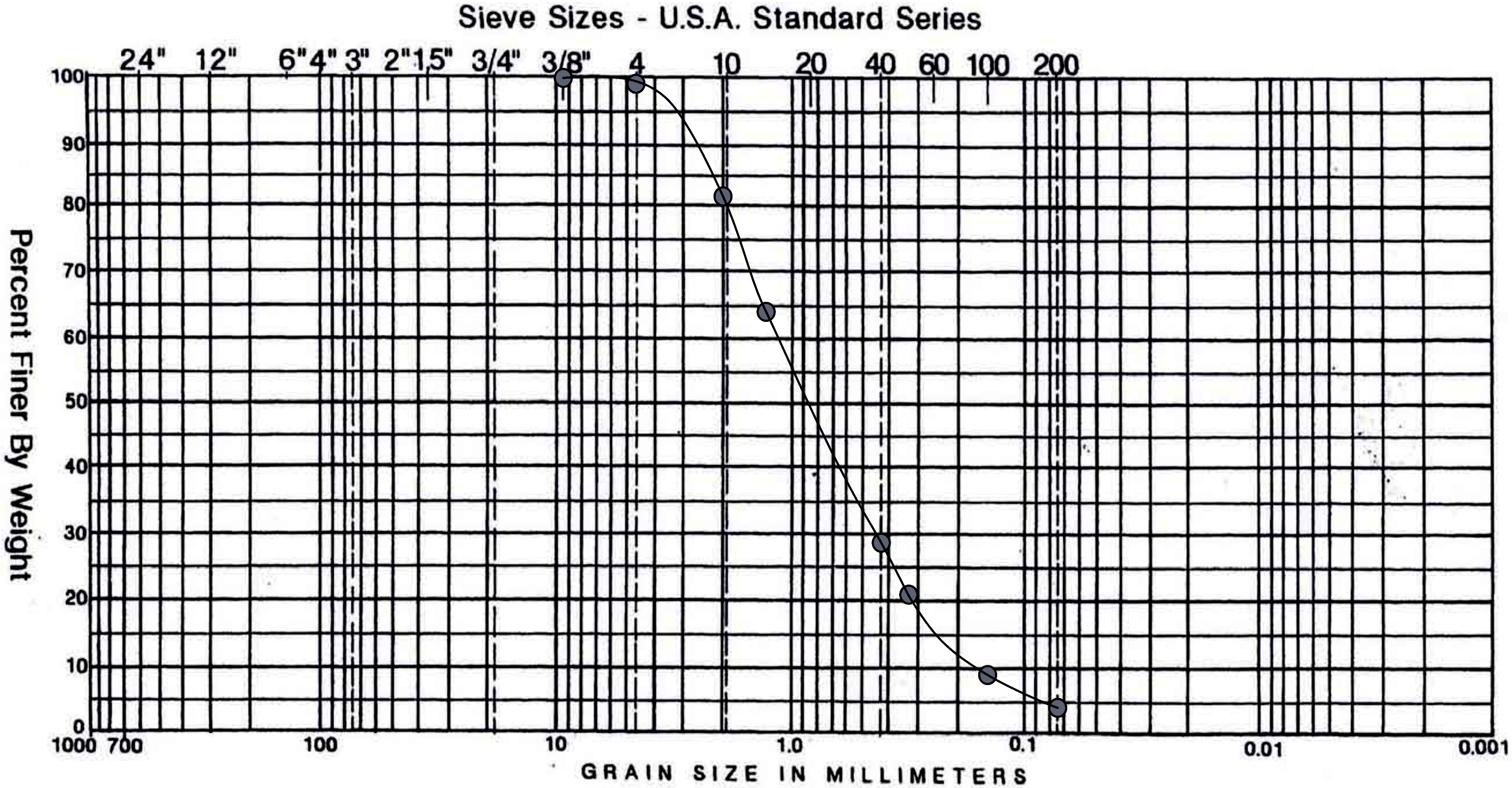
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-3 (PB-1)	9.0'-9.5'	(SW) Well Graded Sand, fine to coarse, 4% fines, D_{10}=0.18 mm

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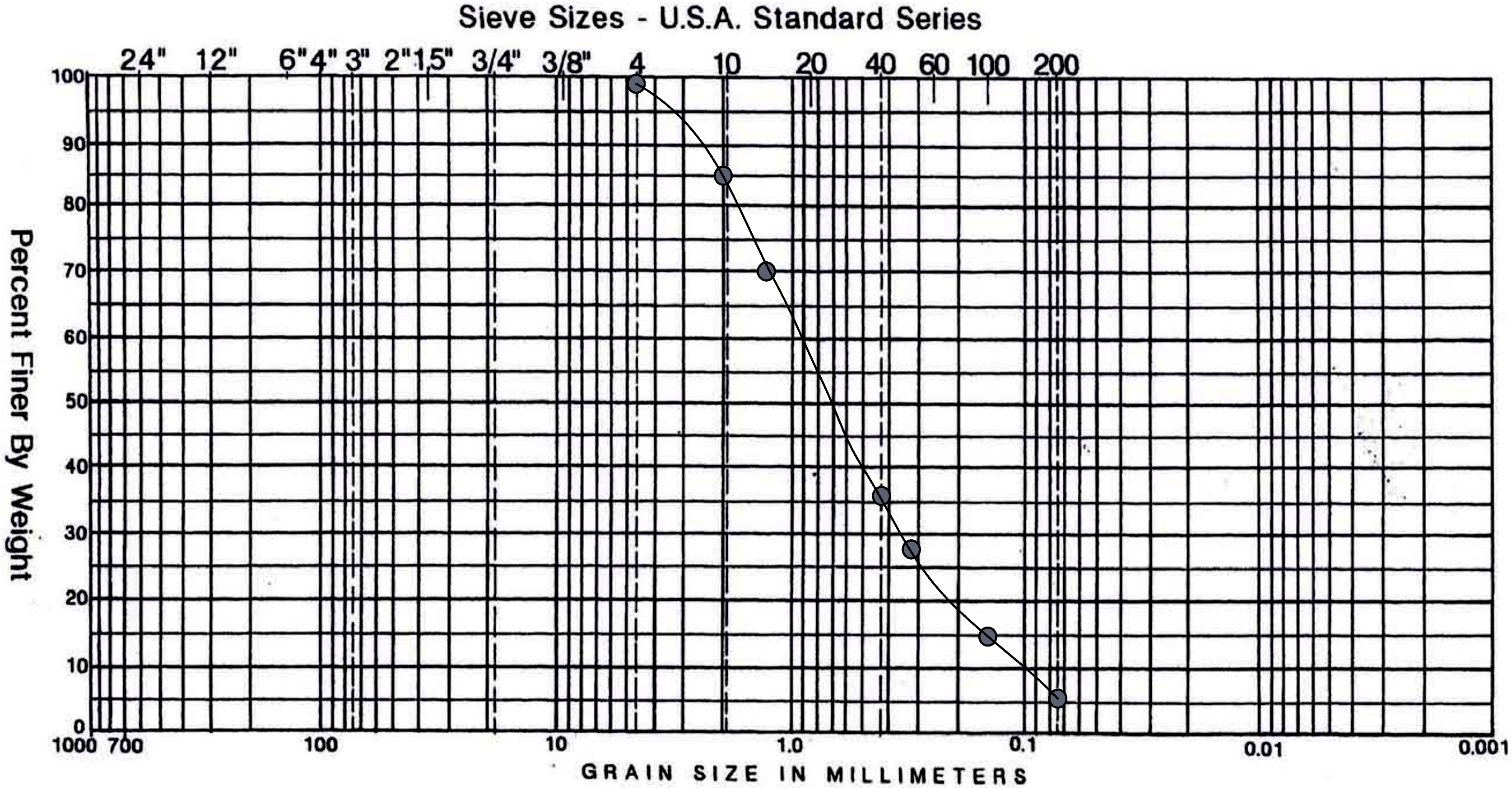
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-4 (PB-2)	5.0'-5.5'	(SW-SM) Well Graded Sand, fine to coarse with silt, **6% fines D10=0.1mm**

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Wildomar, CA

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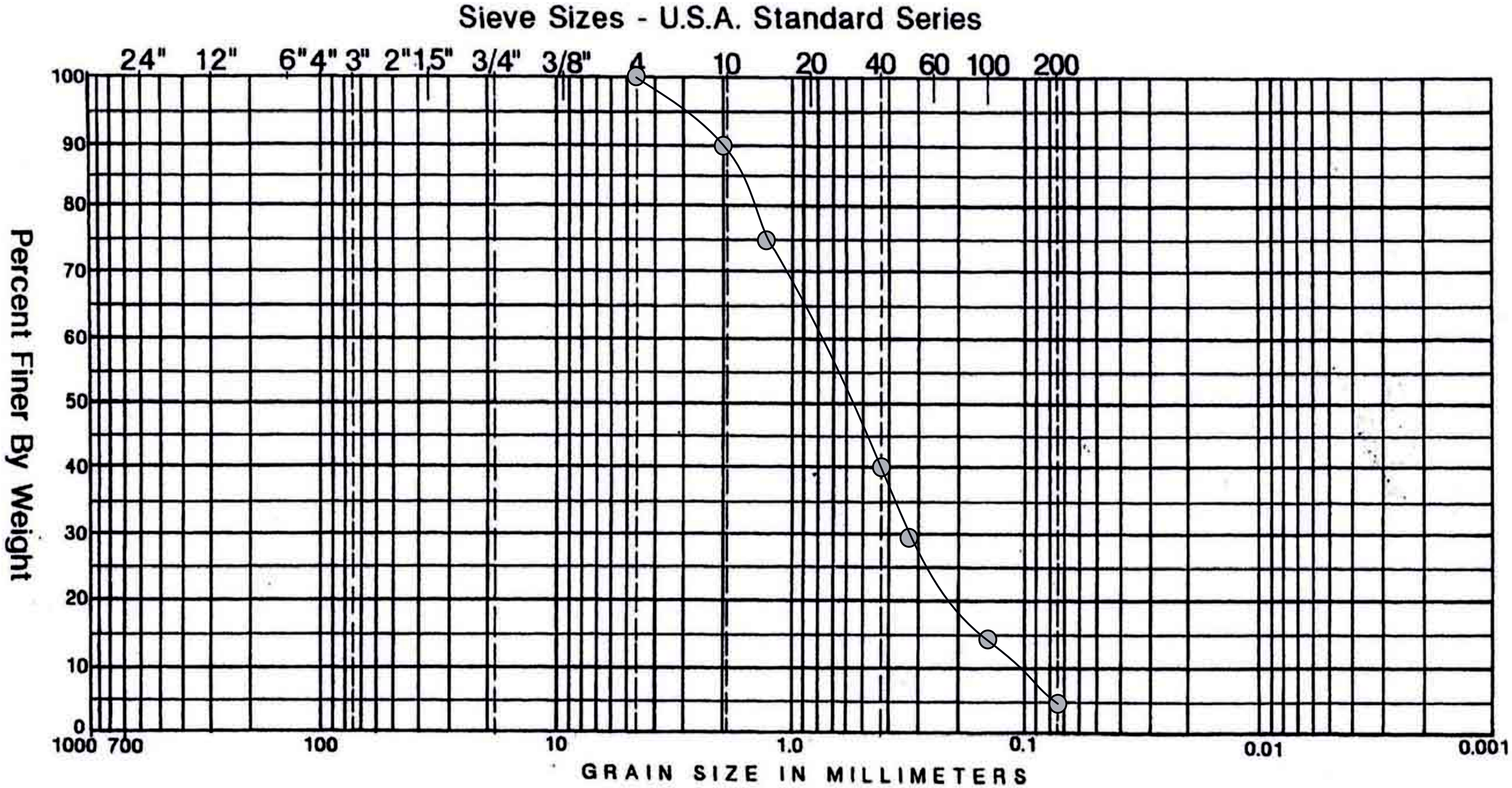
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-5 (PB-3)	5.5'-6.0'	(SP-SM) Poorly graded sand, fine to medium with silt, trace coarse 5% fines. D_{10}=0.12mm

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Wildomar, CA

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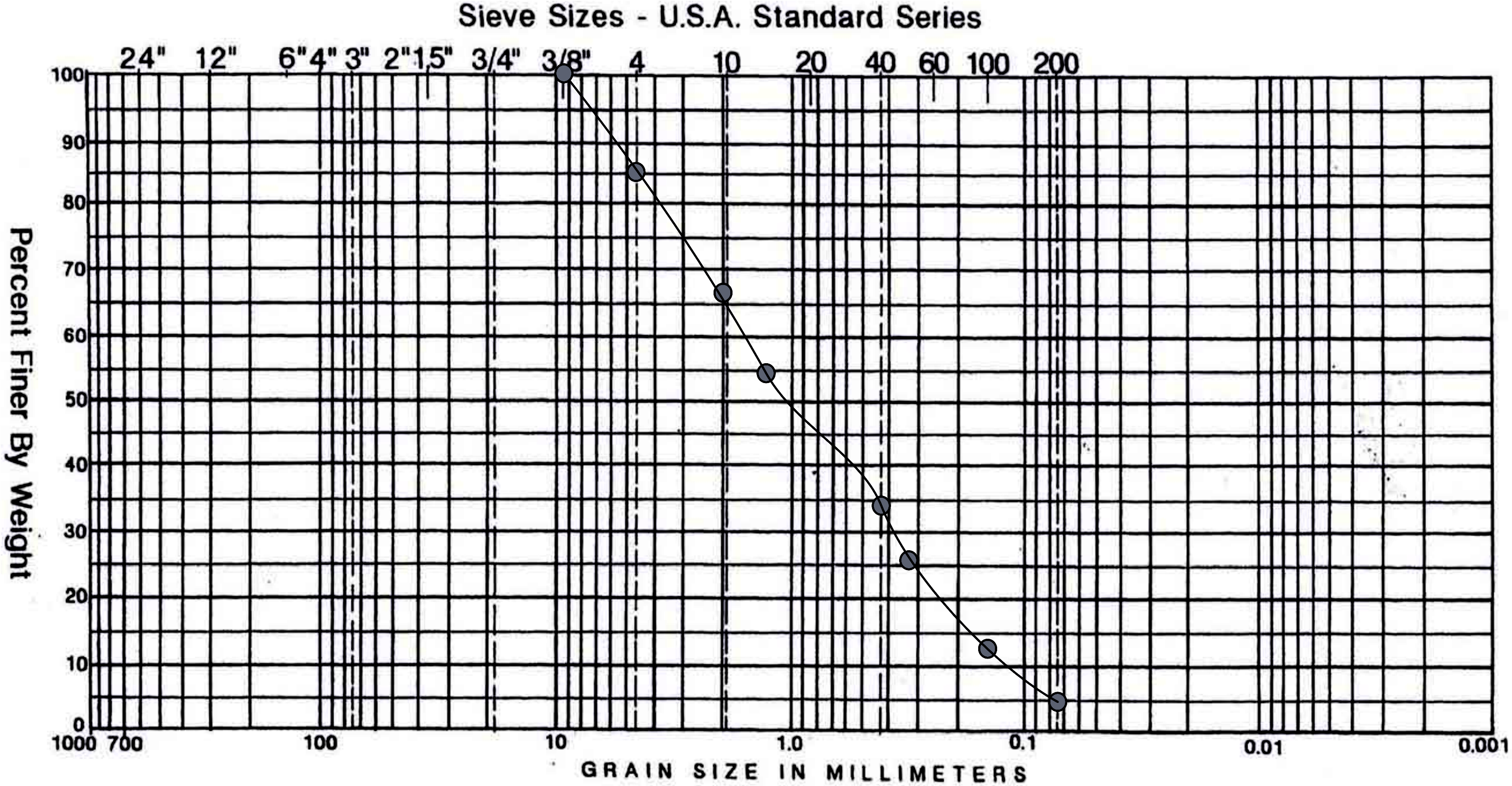
Sieve Analysis Results



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

Symbol	Boring	Depth	Classification
●	B-5 (PB-3)	6.0'-6.5'	(SW-SM) Well graded Sand, fine to coarse with silt and gravel to 3/8",
			5% fines, D10=0.14 mm
			Note: Sample Scalped at 3/8"

24831 Clinton Keith Rd
Wildomar, CA

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APPENDIX C

Liquefaction Analysis

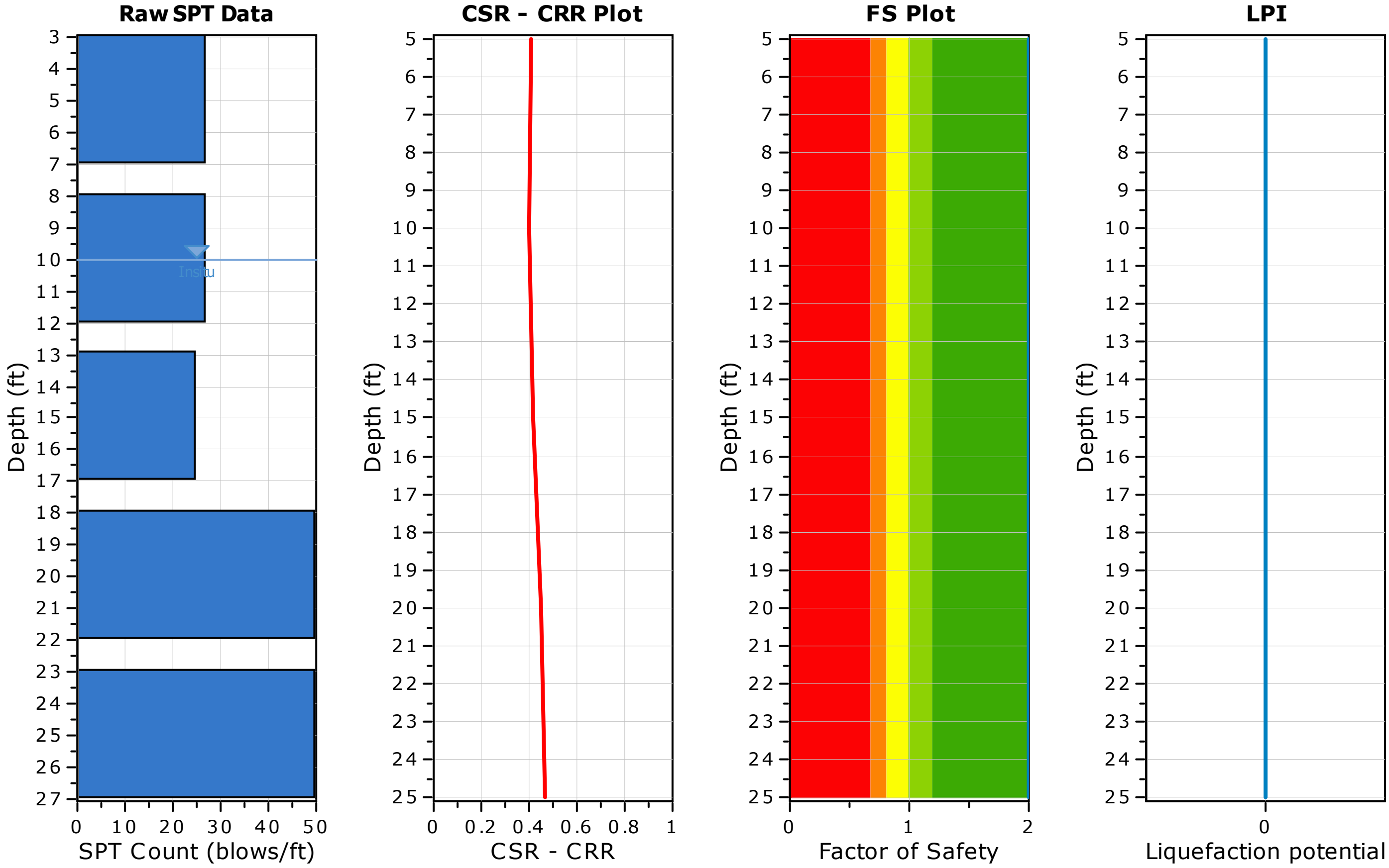
SPT BASED LIQUEFACTION ANALYSIS REPORT

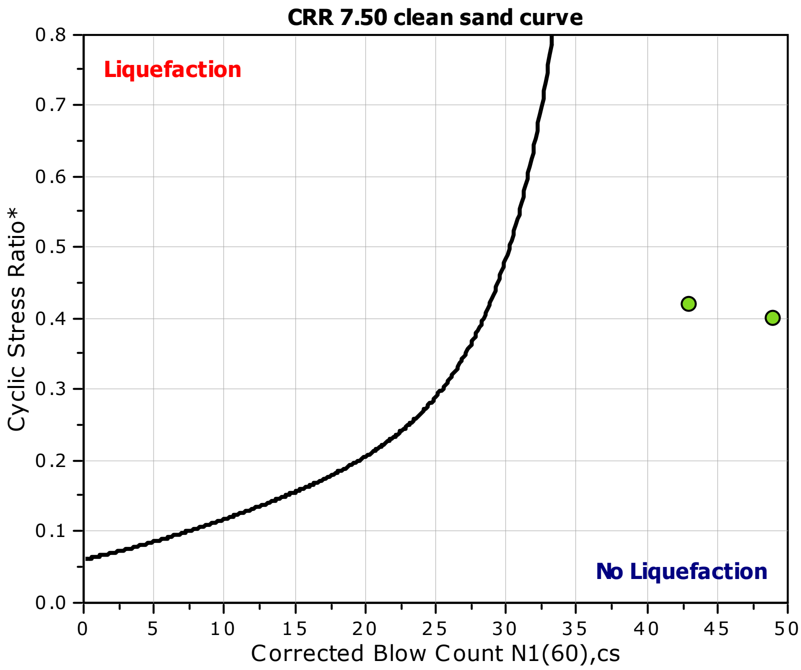
Project title : 24831 Clinton Keith Rd **SPT Name: B-2**

Location : Wildomar, CA

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	10.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	50.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	7.00
Borehole diameter:	200mm	Peak ground acceleration:	0.84 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.28		





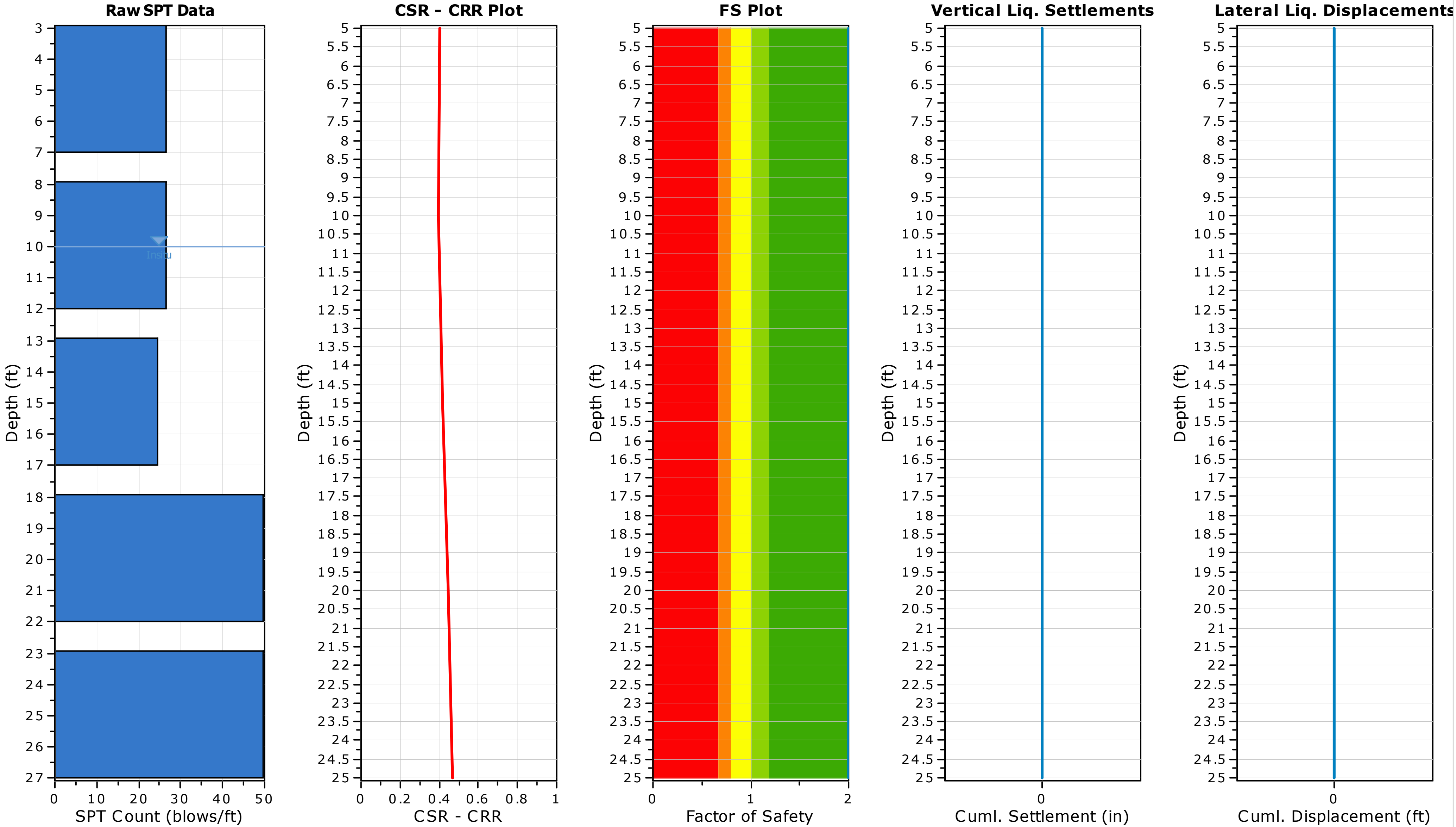
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	27	15.00	130.00	5.00	Yes
10.00	27	15.00	130.00	5.00	Yes
15.00	25	15.00	130.00	5.00	Yes
20.00	50	6.00	130.00	5.00	Yes
25.00	50	6.00	130.00	5.00	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	27	130.00	0.33	0.00	0.33	0.26	1.36	1.28	1.15	0.75	1.20	49	15.00	3.26	52	4.000
10.00	27	130.00	0.65	0.00	0.65	0.26	1.14	1.28	1.15	0.85	1.20	46	15.00	3.26	49	4.000
15.00	25	130.00	0.97	0.16	0.82	0.28	1.07	1.28	1.15	0.85	1.20	40	15.00	3.26	43	4.000
20.00	50	130.00	1.30	0.31	0.99	0.26	1.02	1.28	1.15	0.95	1.20	85	6.00	0.03	85	4.000
25.00	50	130.00	1.63	0.47	1.16	0.26	0.98	1.28	1.15	0.95	1.20	82	6.00	0.03	82	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
m:	Stress exponent normalization factor
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS	
5.00	130.00	0.33	0.00	0.33	0.99	1.00	0.542	2.20	52	1.21	0.447	1.10	0.406	2.000	●
10.00	130.00	0.65	0.00	0.65	0.97	1.00	0.532	2.20	49	1.21	0.439	1.10	0.399	2.000	●
15.00	130.00	0.97	0.00	0.97	0.95	1.00	0.520	2.20	43	1.21	0.429	1.02	0.419	2.000	●
20.00	130.00	1.30	0.00	1.30	0.93	1.00	0.508	2.20	85	1.21	0.419	0.94	0.446	2.000	●
25.00	130.00	1.63	0.00	1.63	0.90	1.00	0.494	2.20	82	1.21	0.408	0.87	0.467	2.000	●

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d: Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} weight factor	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	49	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	5.00	0.000
10.00	46	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	5.00	0.000
15.00	40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	5.00	0.000
20.00	85	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	5.00	0.000
25.00	82	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	5.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

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