LIMITED GEOTECHNICAL INVESTIGATION

CALIFORNIA HIGHWAY PATROL SAN GORGONIO PASS STATION REPLACEMENT OF EXISTING SCREEN WALL 195 HIGHLAND SPRINGS AVENUE BEAUMONT, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

DEPARTMENT OF GENERAL SERVICES
SACRAMENTO, CALIFORNIA

PROJECT NO. A8575-06-24
JULY 29, 2021

Project No. A8575-06-24
July 29, 2021

Mr. John Freeney
Real Estate Services Division
Project Management and Development Branch
707 3rd Street, 4th Floor
West Sacramento CA, 95605

Subject: LIMITED GEOTECHNICAL INVESTIGATION
CALIFORNIA HIGHWAY PATROL - SAN GORGONIO PASS STATION
PROPOSED REPLACEMENT OF EXISTING SCREEN WALL
195 HIGHLAND SPRINGS AVENUE, BEAUMONT, CALIFORNIA

Dear Mr. Freeney:

In accordance with your authorization of our proposal dated April 28, 2021, we have performed a limited geotechnical investigation for the proposed replacement of the existing property line screen wall for the California Highway Patrol San Gorgonio Pass Station located at 195 Highland Springs Avenue in the City of Beaumont, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the improvements can be constructed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

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LIMITED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a limited geotechnical investigation for a proposed replacement of the existing property line screen wall for the California Highway Patrol San Gorgonio Pass Station located at 195 Highland Springs Avenue, in the City of Beaumont, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of the proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was initially explored on May 18 and 19, 2020 by excavating four test pits using hand tools to expose the site wall foundation and record the foundation dimensions. Hand auger borings were excavated inside the test pits. The borings were excavated to depths ranging from approximately 10½ to 12½ feet below the existing ground surface. The site was further explored on June 28, 2021 by excavating two additional hand auger borings to depths of approximately 18 and 20 feet below the existing ground surface. The approximate locations of the site explorations are depicted on the Site Plan, Figure 2. A cross section view of the test pits with observed foundation dimensions is shown on Figure 3, Foundation Sections. A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 195 Highland Springs Avenue, in the City of Beaumont, California. The site is a rectangular-shaped parcel and is currently occupied by the California Highway Patrol San Gorgonio Pass Station and on-grade paved parking areas. The site is bounded by a paved parking lot and East 2nd Street to the north, by a one-story commercial structure and paved parking areas to the south, by Highland Springs Avenue to the east, and by a one-story commercial structure and paved parking areas to the west. The site gently slopes to the south and vegetation is located in planter areas around the perimeter of the building and parking areas. Surface water drainage at the site appears to be by sheet flow along the existing contours to the storm drains in the southern portion of the parking area.

Based on the information provided by the Client, it is our understanding that the proposed project will consist of replacing the existing property line screen wall with a new, taller wall. It is our further understanding that the west wall and western portion of the south wall will be supported on a drilled CIDH pile foundation system. The pile foundation system will likely consist of 18-inch diameter piles that extend to depths of approximately 17 feet below the existing ground surface. The remaining walls will be supported on a conventional foundation system.

Based on input provided by the project structural engineer, it is anticipated that wall loads will be up to 16 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill overlying Pleistocene age older alluvium that consists of varying amounts of gravel, sand and silt (CGS, 2012). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

3.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of approximately 2½ feet below the existing ground surface. The fill consists of reddish brown to dark brown silty sand. The artificial fill is characterized as dry to slightly moist to wet and soft to firm. The fill is likely the result of past grading and construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

3.2 Older Alluvium

The artificial fill is underlain by Pleistocene age older alluvial fan deposits that generally consist of dark reddish brown to reddish brown silty sand with varying amounts of gravel and brown silt. The soils are characterized as slightly moist to moist and firm or medium dense to dense.

4. SEISMIC DESIGN PARAMETERS

The following table summarizes the site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.952g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.668g	Figure 1613.2.1(2)
Site Coefficient, F_A	1	Table 1613.2.3(1)
Site Coefficient, F_V	1.7*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.952g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.135g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.302g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.757g*	Section 1613.2.4 (Eqn 16-39)

Note:

*Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicate that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.795g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.875	Section 11.8.3 (Eqn 11.8-1)

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive. Based on these considerations, the property owner should consider maintaining earthquake insurance for the structure.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

5.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed improvements provided the recommendations presented herein are followed and implemented during design and construction.

5.1.2 Up to 2½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is suitable for continued support of the existing foundations and structural loads, as well as existing building slabs-on-grade. However, the existing fill is not considered suitable for direct support of new foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 5.4).

5.1.3 The proposed property line screen wall may be supported on conventional foundations deriving support in the undisturbed, competent alluvial soils found at and below a depth of 3 feet below the ground surface. Foundations should be deepened as necessary to penetrate through any artificial fill and unsuitable soils to derive support in the competent alluvial soils. The presence of existing artificial fill in proposed foundation excavations must be field verified by Geocon during construction activities. Any soils unintentionally disturbed should be properly compacted. Recommendations for the design of a conventional foundation system are provided in Section 5.5.

5.1.4 As an alternative, the proposed property line screen wall may be supported on a deepened foundation system consisting of drilled, cast-in-place friction piles. Recommendations for the design and installation of friction piles are provided in Sections 5.6 and 5.7.

5.1.5 All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (or Representative of Geocon) prior to the placement of steel or concrete.

5.1.6 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match or exceed the depth of the existing foundation to prevent a surcharge on the existing foundation. Where a proposed foundation will be deeper than and immediately adjacent to an existing foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of the existing foundation.

5.1.7 Excavations up to 5 feet in vertical height are anticipated for construction of the conventional foundations. Drilled excavations on the order of 17 feet in height are anticipated for construction of pile foundations. Performing open excavations adjacent to or deeper than the existing foundation system could potentially remove lateral support and/or undermine the existing foundations. Excavations for construction of new foundations will likely require special excavation measures in order to providea stable excavation and to maintain lateral support of existing foundations. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 5.12).

5.1.8 Once the design and foundation loading configuration for the proposed improvements proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.

5.1.9 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

5.1.10 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in this report.

5.2 Soil and Excavation Characteristics

5.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in vertical excavations, especially where granular soils are encountered. In addition, the contractor should be aware that formwork and/or casing maybe required to prevent caving of foundation excavations.

5.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

5.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 5.12).

5.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

5.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B17) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

5.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B17) and indicate that the on-site materials possess a sulfate exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-14 Table 19.3.1.1.

5.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

5.4 Grading

5.4.1 No large scale grading activities are anticipated for this project. Grading consisting of foundation excavations and utility trench installation is anticipated.

5.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

5.4.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvium encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

5.4.4 It is recommended that proposed foundations penetrate through the existing fill and derive support exclusively in the underlying competent alluvial soils found at or below a depth of 3 feet. The presence of existing artificial fill in proposed foundation excavations will be field verified by Geocon during construction activities. Foundations should be deepened as necessary to penetrate through the existing artificial fill at the direction of the Geotechnical Engineer.

5.4.5 Performing open excavations adjacent to and deeper than existing foundations could potentially remove lateral support and/or undermine the existing foundations and are not acceptable. Excavation for grading and/or construction of new foundations adjacent to existing foundations will require special excavation measures. Recommendations for temporary excavations are provided in Section 5.12.

5.4.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to placing any fill, steel, gravel or concrete.

5.4.7 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

5.4.8 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

5.4.9 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 20 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B17). Import soils must be placed in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

5.4.10 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

5.5 Conventional Foundation Design

5.5.1 The proposed property line screen wall may be supported on conventional foundations deriving support in the undisturbed, competent alluvial soils found at or below a depth of 3 feet. Foundations should be deepened as necessary to penetrate through any existing artificial fill and unsuitable soils to derive support in the competent alluvial soils. The presence of existing artificial fill in proposed foundation excavations must be field verified by Geocon during construction activities.

5.5.2 Where new foundations are constructed immediately adjacent to existing foundations, the new foundation should be deepened to match or exceed the depth of the existing foundation to prevent a surcharge on the existing foundation. Where a proposed foundation will be deeper than an existing adjacent foundation, the proposed foundation must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation.

5.5.3 Continuous footings may be designed for an allowable bearing capacity of 2,500 pounds per square foot (psf) and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing materials.

5.5.4 The soil bearing pressure above may be increased by 300 psf and 600 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 3,700 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

5.5.5 Continuous footings should be reinforced with a minimum of four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. The reinforcement for isolated spread footings should be designed by the project structural engineer.

5.5.6 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

5.5.7 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

5.5.8 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

5.5.9 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

5.5.10 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

5.6 Friction Pile Design

5.6.1 The proposed property line screen wall may also be supported on drilled, cast-in-place friction piles. The piles should be a minimum of 18 inches in diameter and should derive support in undisturbed, competent alluvial soils found at or below a depth of 3 feet. Where piles penetrate through unsuitable fill materials at the surface, these materials should not be considered in the contribution of the pile capacity.

5.6.2 Friction piles may be designed based on a skin friction capacity of 230 psf. Piles may be assumed fixed at an embedment depth of 7 feet below the ground surface. Single pile uplift capacity may be assumed to be ½ of the allowable downward capacity. A one-third increase in the capacity may be used for wind or seismic loads.

5.6.3 A continuous grade beam may be placed across the top of the pile foundations to support the proposed wall, and the appropriate span between piles should be determined by a qualified structural engineer.

5.6.4 Where new grade beams are constructed immediately adjacent to existing foundations, it is recommended that the proposed grade beam match or exceed the depth of an existing foundation to prevent a surcharge on the existing foundation. Where the proposed grade beam will be deeper than the existing foundation, the proposed grade beam must be designed to resist the surcharge imposed by the existing foundation. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation.

5.6.5 If piles are spaced at least at least 3 diameters on center, no reduction in axial capacity is considered necessary for group effects. If pile spacing is closer than three pile diameters, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

5.6.6 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

5.7 Pile Installation

5.7.1 Casing may be required if caving occurs in the granular soil layers during drilled excavations. The contractor should have casing available and should be prepared to use it prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

5.7.2 Groundwater was not encountered during exploration, and groundwater is not expected to be encountered during construction. However, should groundwater or seepage be encountered during pile installation, the contractor should be prepared. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

5.7.3 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.

5.7.4 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required.

5.7.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Unless the holes are fully cased from top to bottom, it is not recommended that holes be left open overnight.

5.8 Foundation Settlement

5.8.1 The maximum expected settlement for the proposed wall supported on a conventional foundation system with a maximum allowable bearing value of 3,700 psf and deriving support in the recommended bearing material is estimated to be approximately 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is expected to be less than ½ inch over a distance of 20 feet.

5.8.2 The maximum expected total settlement for the pile supported wall deriving support in the competent undisturbed alluvium is estimated to be less than a ½ inch. Differential settlement between adjacent piles foundations is not expected to exceed ¼ inch.

5.8.3 Differential settlement between conventional foundations and pile foundations is expected to be less than ½ inch.

5.8.4 Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

5.9 Lateral Design

5.9.1 Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces in undisturbed alluvial soils.

5.9.2 Passive earth pressure for the sides of foundations and slabs poured against undisturbed alluvial soils may be computed as an equivalent fluid having a density of 250 pcf with a maximum earth pressure of 2,500 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. The allowable capacity may be doubled for isolated piles spaced more than three times the diameter.

5.9.3 If piles are spaced at least at least 8 diameters on-center when loaded in-line and at least three diameters on-center when loaded in parallel, no reduction in lateral capacity is considered necessary for group effects. If so spaced, piles may be considered isolated and the allowable passive pressure may be doubled based on isolated pile conditions. If pile spacing is closer, an evaluation for group effects including appropriate reductions should be incorporated into the pile design based on pile dimension, spacing, and the direction of loading.

5.10 Retaining Wall Design

5.10.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

5.10.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Conventional Foundation Design* and *Friction Pile Design* sections of this report (see Sections 5.5 and 5.6).

5.10.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 5	30	63

5.10.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soil. If import soil is used to backfill proposed walls, revised earth pressures may be required to account for the geotechnical properties of the soil placed as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

5.10.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained restrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

5.10.6 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

5.10.7 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}\!/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}\!/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

5.10.8 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\ (z) = \sigma_H(z)cos^2(1.1\theta)$$

where x is the distance from the face of the excavation/wall to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth z, θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth z.

5.10.9 In addition to the recommended earth pressure, the upper 10 feet of the retaining wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

5.11 Retaining Wall Drainage

5.11.1 Retaining walls not designed for hydrostatic pressure should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 4). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

5.11.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 5). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

5.11.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures.

5.11.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

5.12 Temporary Excavations

5.12.1 Excavations on the order of 5 feet in height and 17 feet in height for drilled excavations are anticipated during foundation excavations. The excavations are expected to expose artificial fill and alluvial soils that are considered suitable for vertical excavations up to 5 feet in height where not surcharged by adjacent traffic or structures.

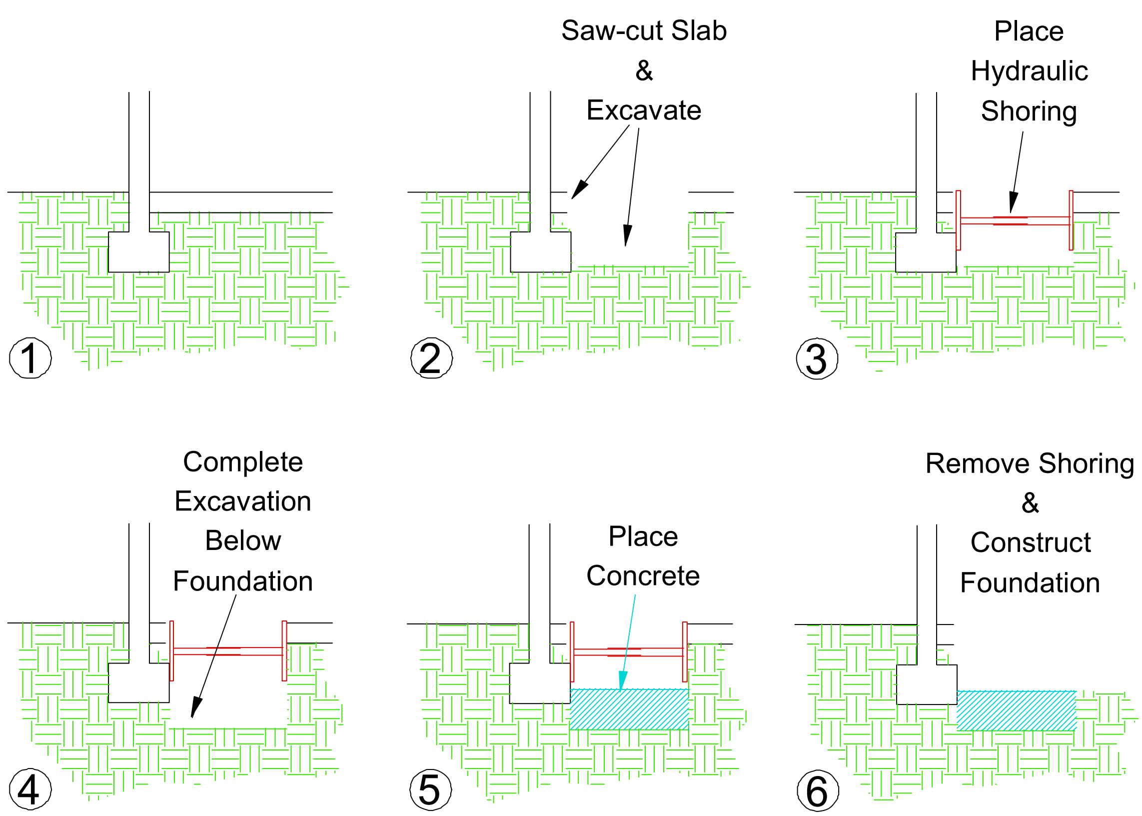
5.12.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 6 feet. A uniform slope does not have a vertical portion.

5.12.3 Performing open excavations adjacent to and deeper than existing foundations could potentially remove lateral support and/or undermine the existing foundations and are not acceptable. Special excavations measures such as slot-cutting or trench shoring will be required where the proposed excavation will be deeper than an existing adjacent foundation or adjacent to a property line. Recommendations for slot-cutting and trench shoring are provided in the following sections.

5.12.4 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

5.13 Trench Shoring

5.13.1 In order to maintain lateral support of existing footings and/or improvements, hydraulic trench shoring may be implemented where excavations will extend below existing foundations. The excavation may be conducted adjacent to the foundation but should not extend below the foundation until the shoring is installed. Once the concrete is placed to an elevation that is slightly above the bottom of the existing adjacent foundation, the shoring may be removed and the new foundation constructed. The selection of the shoring system is the responsibility of the contractor. Shoring systems should be designed by a California licensed civil or structural engineer with experience in designing shoring systems (see illustration below).



5.13.2 It is recommended that an equivalent fluid pressure based on the table below, be utilized for design of hydraulic shoring.

HEIGHT OF SHORED EXCAVATION (FEET)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (ACTIVE PRESSURE)	EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE)
Up to 5	25	63

5.13.3 It is very important to note that active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure, the at-rest pressure should be considered for design purposes.

5.13.4 Additional active pressure should be added for a surcharge condition due to sloping ground or adjacent structures and should be designed for each condition as the project progresses.

5.13.5 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, QL is the vertical line-load and σH is the horizontal pressure at depth z.

5.13.6 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\,(z) = \sigma_H(z) cos^2(1.1\theta)$$

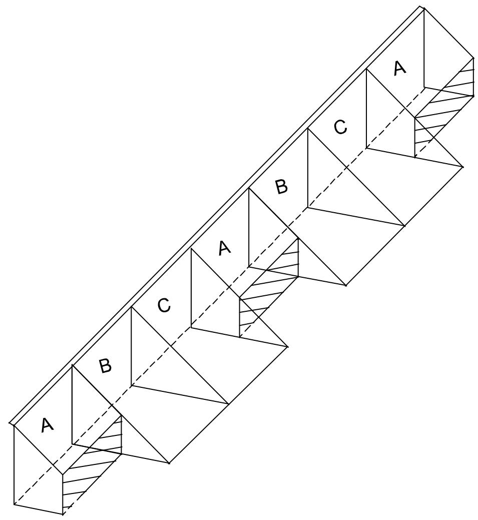
where x is the distance from the face of the excavation to the vertical point-load, H is distance from the outrigger/bottom of column footing to the bottom of excavation, z is the depth at which the horizontal pressure is desired, Q_P is the vertical point-load, σ is the vertical pressure at depth z, Θ is the angle between a line perpendicular to the bulkhead and a line from the point-load to half the pile spacing at the bulkhead, and σ_H is the horizontal pressure at depth z.

5.13.7 A qualified engineer should be retained to review and prepare a shoring plan in accordance with the shoring manufacture's specifications.

5.14 Slot-Cutting

5.14.1 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. Where slot-cutting is used for foundation construction, the proposed construction techniques should be discussed with the structural engineer so that appropriate modifications can be made to the foundation design; such as additional reinforcing or details for doweling.

5.14.2 Where insufficient space is available to perform uniform 1:1 (H:V) sloped excavations along a property line, slot-cutting methods can be used. It is recommended that the initial temporary excavation along the property line be sloped back at a uniform 1:1 (H:V) slope gradient or flatter for excavation of the existing soils to the necessary depth. The temporary slope may then be excavated using slot-cutting (see illustration on following page).



5.14.3 Alternate "A" slots of 8 feet in width may be worked. The remaining earth buttresses ("B" and "C" slots) should also be 8 feet in width. The wall, foundation, or backfill should be completed in the "A" slots to a point where support of the offsite property and/or any existing structures is restored before the "B" slots are excavated. After completing the wall, foundation, or backfill in the "B" slots, finally the "C" slots may be excavated. Slot-cutting is not recommended for vertical excavations greater than 5 feet in height. A surcharge load of 100 pounds per linear foot is included in the slot cut calculation to account for miscellaneous minor surcharges. The surcharge load from the existing structures adjacent to proposed slot-cuts should be evaluated by a qualified structural engineer, and the slot-cut calculation revised as necessary. A slot-cut calculation is provided on Figure 6.

5.15 Surface Drainage

5.15.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

5.15.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

5.15.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

5.15.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

5.16 Plan Review

5.16.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

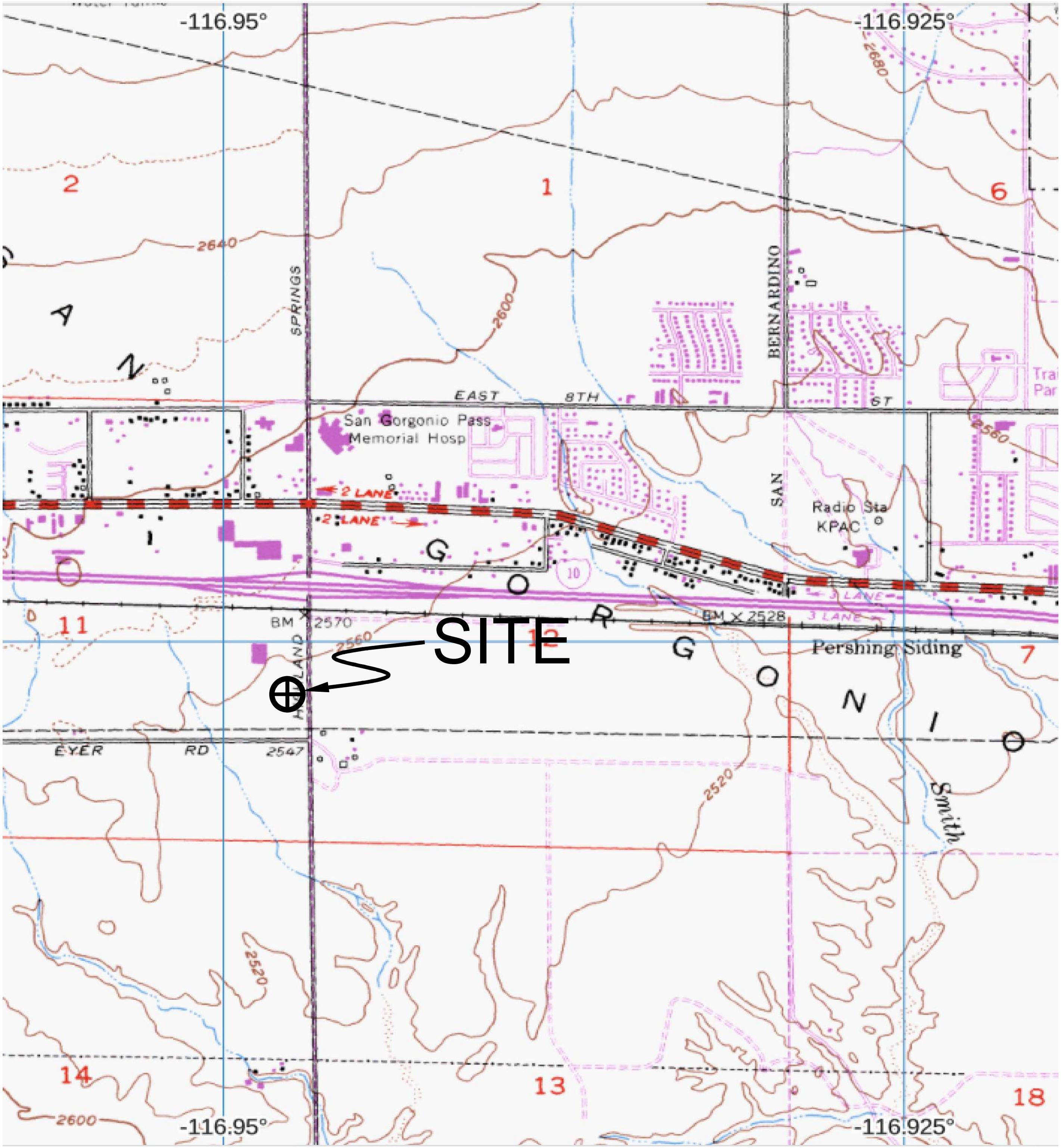
4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

California Department of Water Resources, 2020, *Groundwater Level Data by Township, Range, and Section*, http://www.water.ca.gov/waterdatalibrary/groundwater/ hydrographs/index_trs.cfm.

California Division of Mines and Geology, 1995, *State of California Earthquake Fault Zones, Beaumont Quadrangl*e, Official Map Released June 1, 1995.

California Geological Survey, 2012, *Preliminary Geologic Map of Quaternary Surficial Deposits in Southern California, Palm Springs 30' X 60' Quadrangle, A Project for the Department of Water Resources by the California Geological Survey*, Compiled from existing sources by Trinda L. Bedrossian, CEG, Jeremy T. Lancaster, CEG and Cheryl A. Hayhurst, PG, CGS Special Report 217, Plate 24, Scale 1:100,000.



LATITUDE = 33.922882
LONGITUDE = -116.947462
U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, BEAUMONT, CA QUADRANGLE

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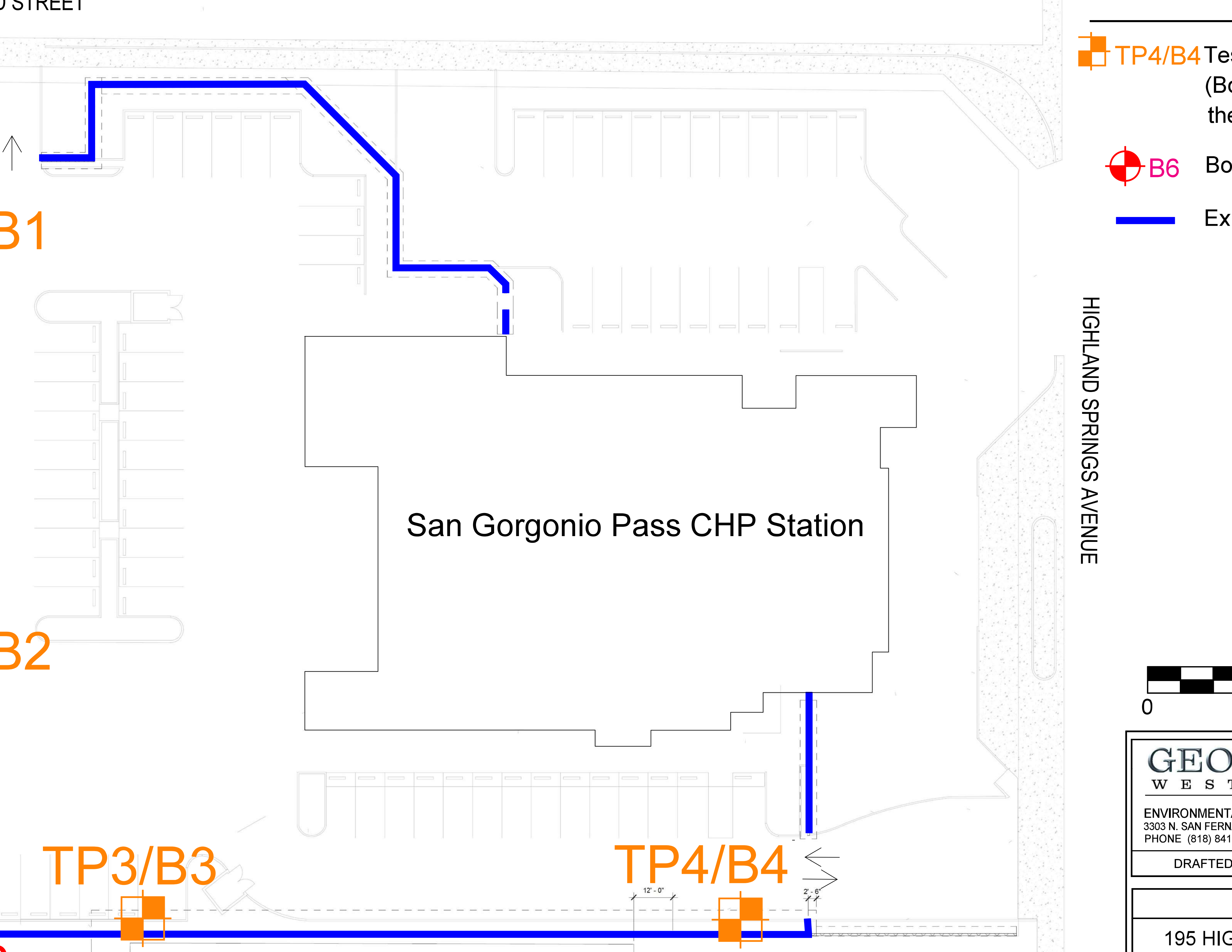
DRAFTED BY: JJK	CHECKED BY: GAK

VICINITY MAP

195 HIGHLAND SPRINGS AVENUE

BEAUMONT, CALIFORNIA

JULY 2021	PROJECT NO. A8575-06-24	FIG. 1



D STREET
HIGHLAND SPRINGS AVENUE
San Gorgonio Pass CHP Station
B1
B2
TP3/B3
TP4/B4
12' - 0"
2' - 6"
TP4/B4 Tes
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3303 N. SAN FERN
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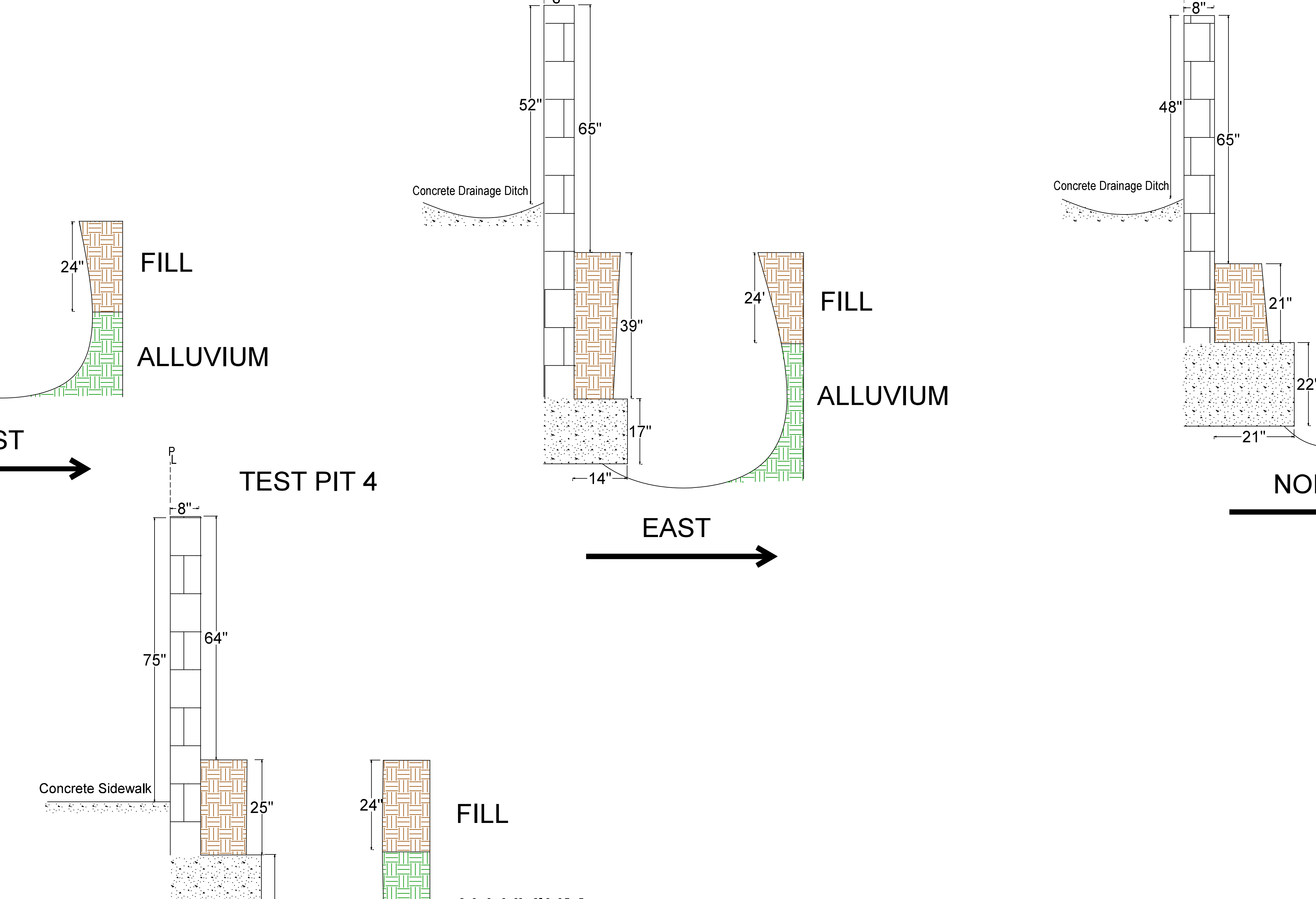

*FOUNDATION SECTIONS NOT TO SC

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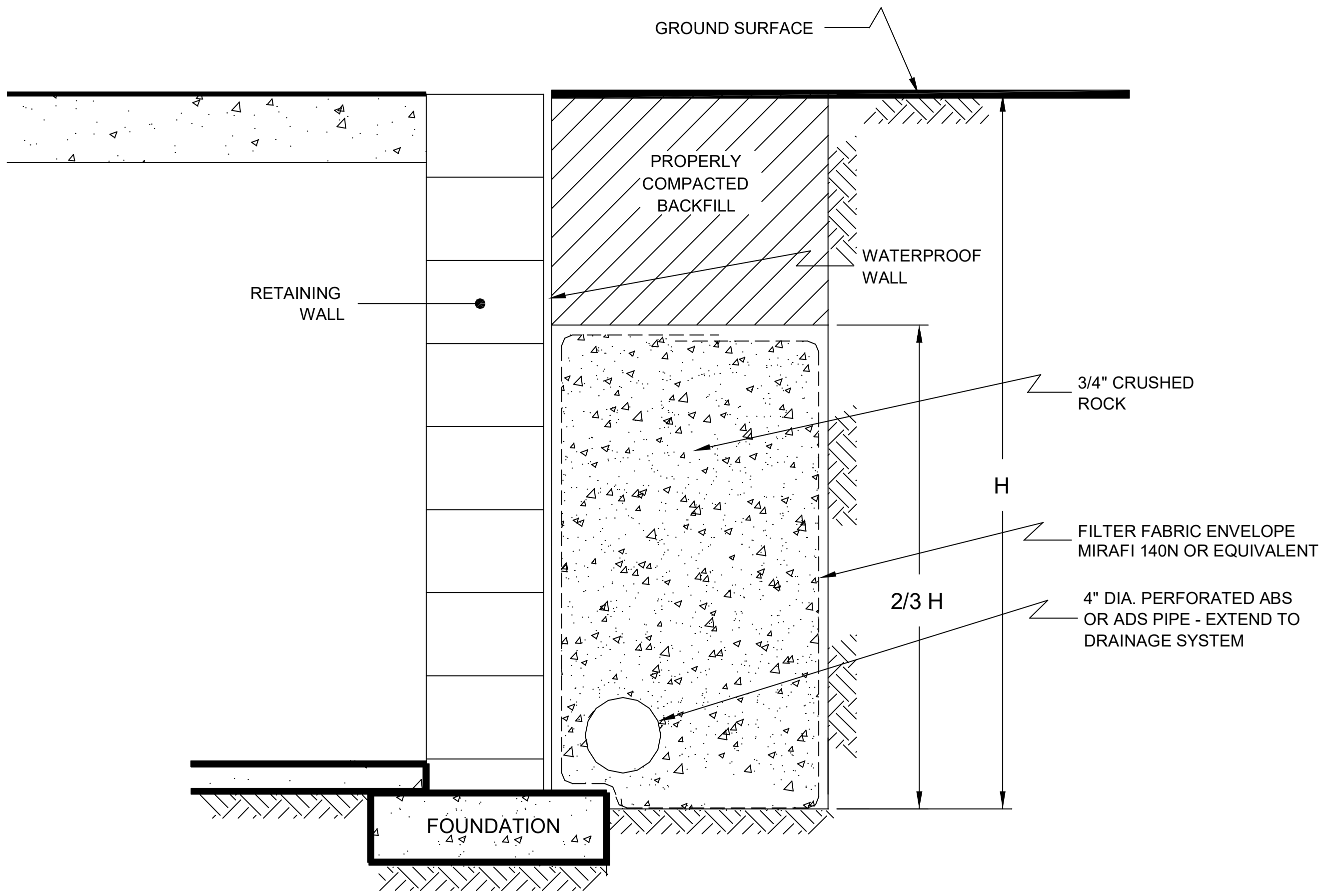

NO SCALE

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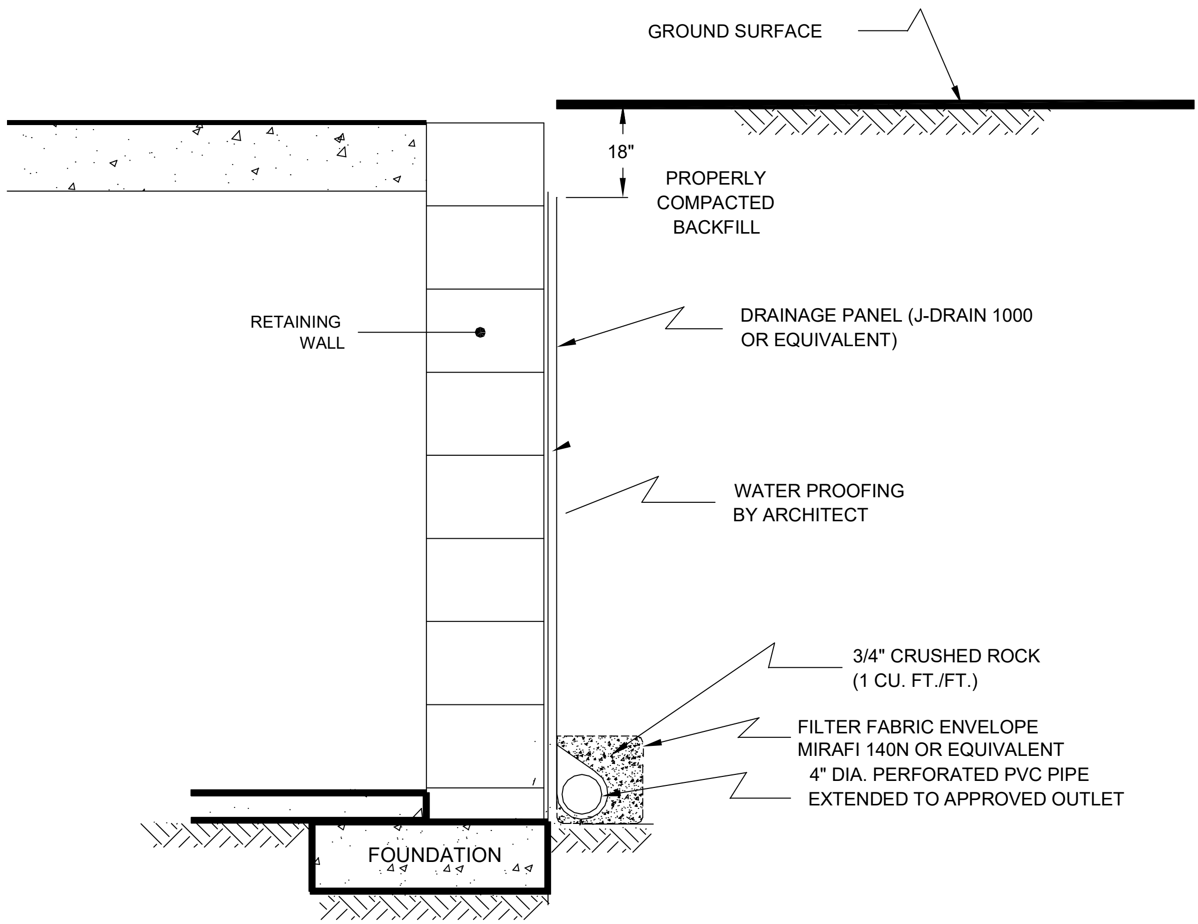
RETAINING WALL DRAIN DETAIL

195 HIGHLAND SPRINGS AVENUE

BEAUMONT, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	JULY 2021	PROJECT NO. A8575-06-24	FIG. 4



NO SCALE

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RETAINING WALL DRAIN DETAIL

195 HIGHLAND SPRING AVENUE
BEAUMONT, CALIFORNIA

DRAFTED BY: JJK	CHECKED BY: HHD	JUNLY 2021	PROJECT NO. A8575-06-24	FIG. 5

Slot Cut Calculation

Input:

Height of Slots	(H)	5.0	feet
Unit Weight of Soils	(γ)	125.0	pcf
Friction Angle of Soils	(ϕ)	31.0	degrees
Cohesion of Soils	(c)	250.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Design Equations

b = H/(tan α)

A = 0.5*H*b

W = 0.5*H*b*γ *(per lineal foot of slot width)*

F_1 = d*W*(sin α)

R_1 = d*[W*(cos α)*(tan ϕ)+(c*b)]

R_2 = 2*[(0.5*H*b)*c]

FS = Resistance Force/Driving Force

FS = (R_1+R_2)/(F_1)

Surcharge Pressure:

Line Load	(q_L)	100.0	plf
Distance Away from Edge of Excavation	(X)	3.0	feet

Failure Angle (α) degrees	Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet^2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force lbs	**Allowable Width of Slots*** (d) feet
45	5.0	13	1562.5	1175.6	2474.1	6250.0	8.0
46	4.8	12	1508.9	1157.3	2409.2	6035.6	8.0
47	4.7	12	1457.1	1138.8	2347.2	5828.2	8.0
48	4.5	11	1406.9	1119.8	2287.9	5627.5	8.0
49	4.3	11	1358.3	1100.6	2231.1	5433.0	8.0
50	4.2	10	1311.1	1081.0	2176.8	5244.4	8.0
51	4.0	10	1265.3	1061.0	2124.7	5061.2	8.0
52	3.9	10	1220.8	1040.8	2074.9	4883.0	8.0
53	3.8	9	1177.4	1020.2	2027.1	4709.7	8.0
54	3.6	9	1135.2	999.3	1981.3	4540.9	8.0
55	3.5	9	1094.1	978.1	1937.5	4376.3	8.0
56	3.4	8	1053.9	956.6	1895.5	4215.7	8.0
57	3.2	8	1014.7	934.9	1855.2	4058.8	8.0
58	3.1	8	976.4	912.8	1816.7	3905.4	8.0
59	3.0	8	938.8	890.5	1779.8	3755.4	8.0
60	2.9	7	902.1	781.3	1744.4	3608.4	8.0
61	2.8	7	866.1	757.5	1710.6	3464.4	8.0
62	2.7	7	830.8	733.5	1678.3	3323.2	8.0
63	2.5	6	796.1	709.4	1647.4	3184.5	8.0
64	2.4	6	762.1	685.0	1617.8	3048.3	8.0
65	2.3	6	728.6	660.3	1589.6	2914.4	8.0
66	2.2	6	695.7	635.5	1562.8	2782.7	8.0
67	2.1	5	663.2	610.5	1537.1	2653.0	8.0
68	2.0	5	631.3	585.3	1512.8	2525.2	8.0
69	1.9	5	599.8	559.9	1489.6	2399.2	8.0
70	1.8	5	568.7	534.4	1467.6	2274.8	8.0

* Width of Slots to achieve a minimum of 1.25 Factor of Safety, with a Maximum Allowable Slot Width of 8-feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.25:

d_{allow} = 8.0 feet

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SLOT CUT CALCULATION

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on May 18 and 19, 2020 by excavating four test pits using hand tools to expose the site wall foundation and record the foundation dimensions. Hand auger borings were excavated inside the test pits. The borings were excavated to depths ranging from approximately 10½ to 12½ feet below the existing ground surface. The site was further explored on June 28, 2021 by excavating two additional hand auger borings to depths of approximately 18½ feet and 20 feet below existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3 inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a slide hammer. The California Modified Sampler is equipped with 1-inch high by 2 $^{3}/_{8}$-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the test pits and borings are shown on Figure 2.

PROJECT NO. A8575-06-24

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1 ELEV. (MSL.) -- DATE COMPLETED 5/18/2020 EQUIPMENT HAND AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry, brown, fine-grained, fine to coarse gravel.			
2					**ALLUVIUM** Silty Sand, medium dense, slightly moist, dark reddish brown, fine-grained, rootlets.			
4	B1@3.5'						129.7	14.6
				SM				
6	B1@5.5'				- firm, reddish brown, some clay and rootlets		116.6	16.3
8	B1@7.5'				- loose to medium dense, slightly moist to moist		108.6	18.7
				ML	Silt, firm, slightly moist, brown, trace clay.			
10	B1@10'						100.2	22.7
					Total depth of boring: 10.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A1,
Log of Boring 1, Page 1 of 1

A9811-06-04 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A8575-06-24

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 5/18/2020

EQUIPMENT HAND AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-4'				**ARTIFICIAL FILL** Silty Sand, medium dense, slightly moist, dark brown, fine-grained, fine to coarse gravel, rootlets.			
2					**ALLUVIUM** Silty Sand, medium dense, slightly moist, reddish brown.			
4								
6	B2@5.5'				- slightly moist, some clay, trace medium- to coarse-grained sand		113.1	14.9
				SM				
8	B2@7.5'						115.4	16.9
10	B2@10'				- moist, brown, no clay, medium- to coarse-grained		109.1	17.8
12	B2@12'				- silty lense, reddish brown, grades coarser		110.2	20.0

Total depth of boring: 12.5 feet
Fill to 2 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

A9811-06-04 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8575-06-24

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 5/19/2020

EQUIPMENT HAND AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silty Sand, medium dense, dry to slightly moist, reddish brown, fine to coarse gravel and roots.			
2								
					ALLUVIUM Silty Sand, medium dense, slightly moist, dark reddish brown, fine-grained.			
4								
	B3@5'				- slightly moist, trace medium- to coarse-grained		114.0	13.0
6				SM				
8	B3@8'				- medium- to coarse-grained, trace fine gravel and clay		119.6	11.3
					Silt, firm, slightly moist to moist, brown.			
10	B3@10'			ML			102.0	25.6
					Silty Sand, dense, slightly moist, reddish brown, fine- to coarse-grained, fine gravel.			
12	B3@12'			SM			116.5	10.1

Total depth of boring: 12.5 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

Figure A3,
Log of Boring 3, Page 1 of 1

A9811-06-04 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8575-06-24

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) -- DATE COMPLETED 5/19/2020 EQUIPMENT HAND AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand, medium dense, dry to slightly moist, light reddish brown.			
2					**ALLUVIUM** Silty Sand, medium dense, slightly moist to moist, dark reddish brown, fine-grained.			
4								
6	B4@5.5'			SM	- moist, reddish brown, trace fine gravel and clay		120.3	15.0
8	B4@7.5'				- some medium- to coarse-grained, trace fine to coarse gravel, no clay		117.0	12.2
10					Silt, firm, slightly moist to moist, brown.			
	B4@10.5'			ML			99.6	23.2
12	B4@12'				- moist		96.4	24.0
					Total depth of boring: 12.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A4,
Log of Boring 4, Page 1 of 1

A9811-06-04 BORING LOGS.GPJ

SAMPLE SYMBOLS	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) -- DATE COMPLETED 6/28/2021 EQUIPMENT HAND AUGER BY: JJK	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 – 2.5					**ARTIFICIAL FILL** Sandy Silt, firm, dry, light reddish brown, some fine to coarse gravel. - dry to slightly moist, reddish brown			
2.5 – 7				ML	**ALLUVIUM** Sandy Silt, soft, slightly moist, reddish brown, trace clay and medium-grained sand.			
7 – 10				SC	Clayey Sand, loose to medium dense, slightly moist to moist, reddish brown, fine-grained, trace coarse-grained. - trace fine to coarse gravel			
10 – 13	B5@10.5'			ML	Silt, firm, moist, brown.		103.5	24.2
13 – 17	B5@15'			SC	Clayey Sand, medium dense, moist, reddish brown, fine-grained, some coarse-grained.		109.2	15.9
17 – 20	B5@18.5'			SM	Silty Sand, medium dense, moist, light reddish brown, fine-grained, some coarse-grained and fine gravel.		111.4	14.8

Total depth of boring: 20 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

A8575-06-24 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. A8575-06-24

BORING 6

ELEV. (MSL.) -- DATE COMPLETED 6/28/2021

EQUIPMENT HAND AUGER BY: JJK

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0 – 2.5					**ARTIFICIAL FILL** Sandy Silt, firm, dry, light brown to brown, trace fine gravel.			
2.5 – 8				ML	**ALLUVIUM** Sandy Silt, soft to firm, slightly moist, reddish brown, trace clay. - light brown to yellowish brown			
8 – 13.5	B6@9'			ML	Silt, firm, slightly moist to moist, light yellowish brown.		98.7	23.1
13.5 – 18	B6@14.5'			SM	Silty Sand, medium dense, slightly moist to moist, light reddish brown.		111.5	15.5

Depth scale (feet): 0, 2, 4, 6, 8, 10, 12, 14, 16, 18

Total depth of boring: 18 feet
Fill to 2.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A6,
Log of Boring 6, Page 1 of 1

A8575-06-24 BORING LOGS.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

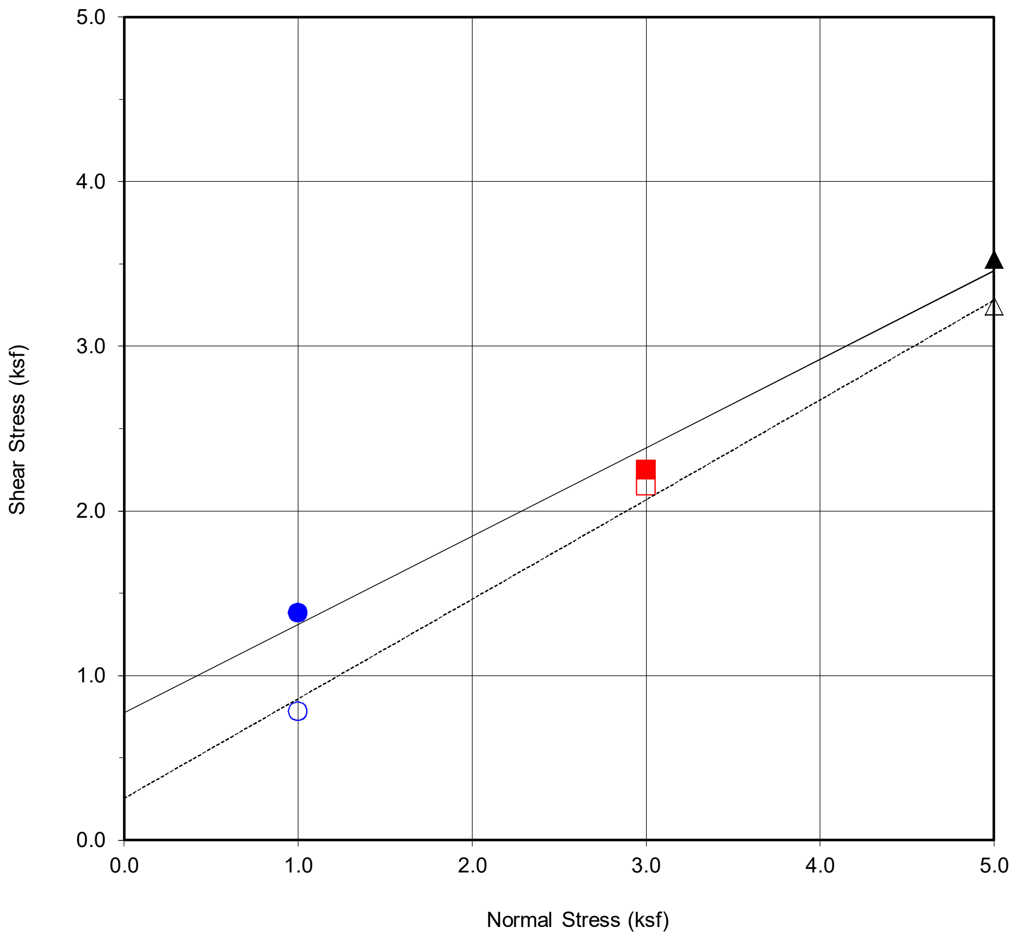
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, maximum dry density, corrosivity content, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B17. The in-place dry density and moisture content of the samples are presented on the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@3.5**
Depth (ft)	**3.5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	773	28.2
Ultimate	254	31.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.38	■ 2.25	▲ 3.53
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.15	△ 3.24
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.1	14.6	15.2
Initial Dry Density (pcf)	117.4	112.2	110.3
Initial Degree of Saturation (%)	87.2	78.7	77.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.4	16.4	16.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

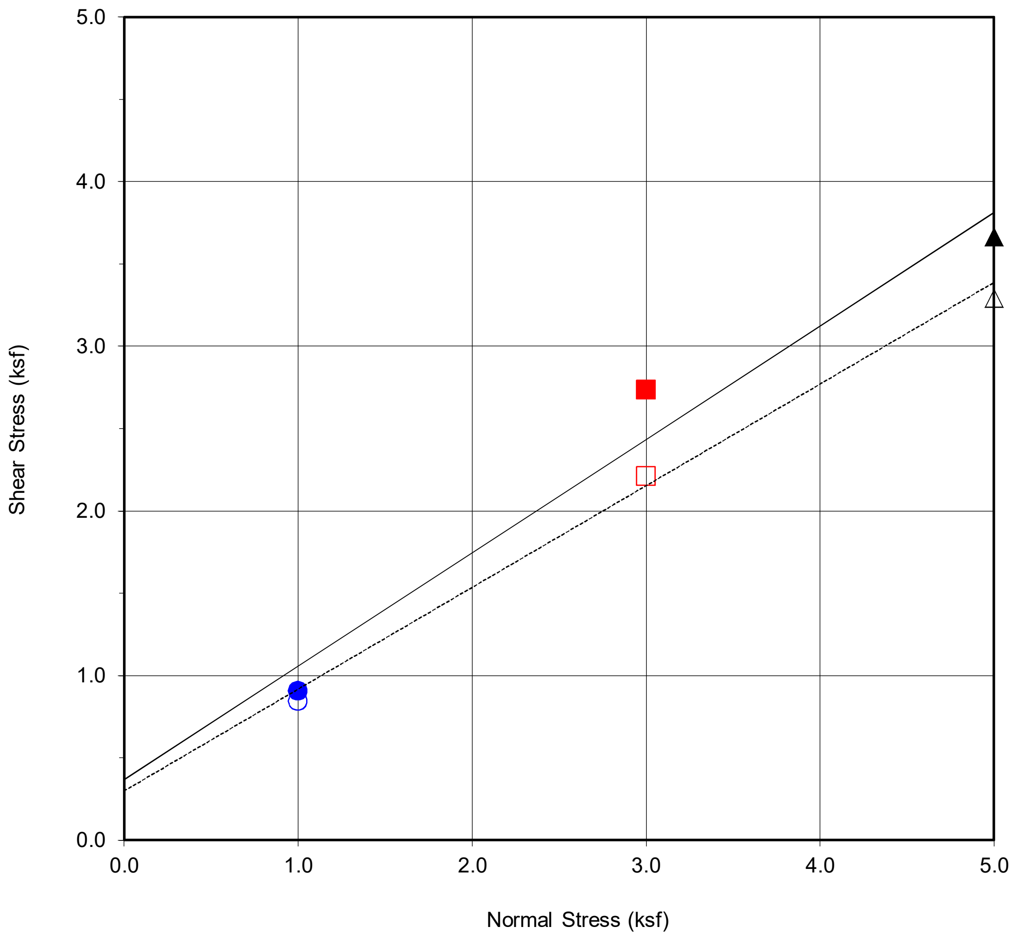
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B1



Boring No.	B2
Sample No.	B2@5.5
Depth (ft)	5.5
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	366	34.6
Ultimate	299	31.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.91	■ 2.73	▲ 3.66
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.21	△ 3.29
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.9	15.3	15.7
Initial Dry Density (pcf)	110.6	115.2	108.4
Initial Degree of Saturation (%)	77.0	89.4	76.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.7	16.8	17.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

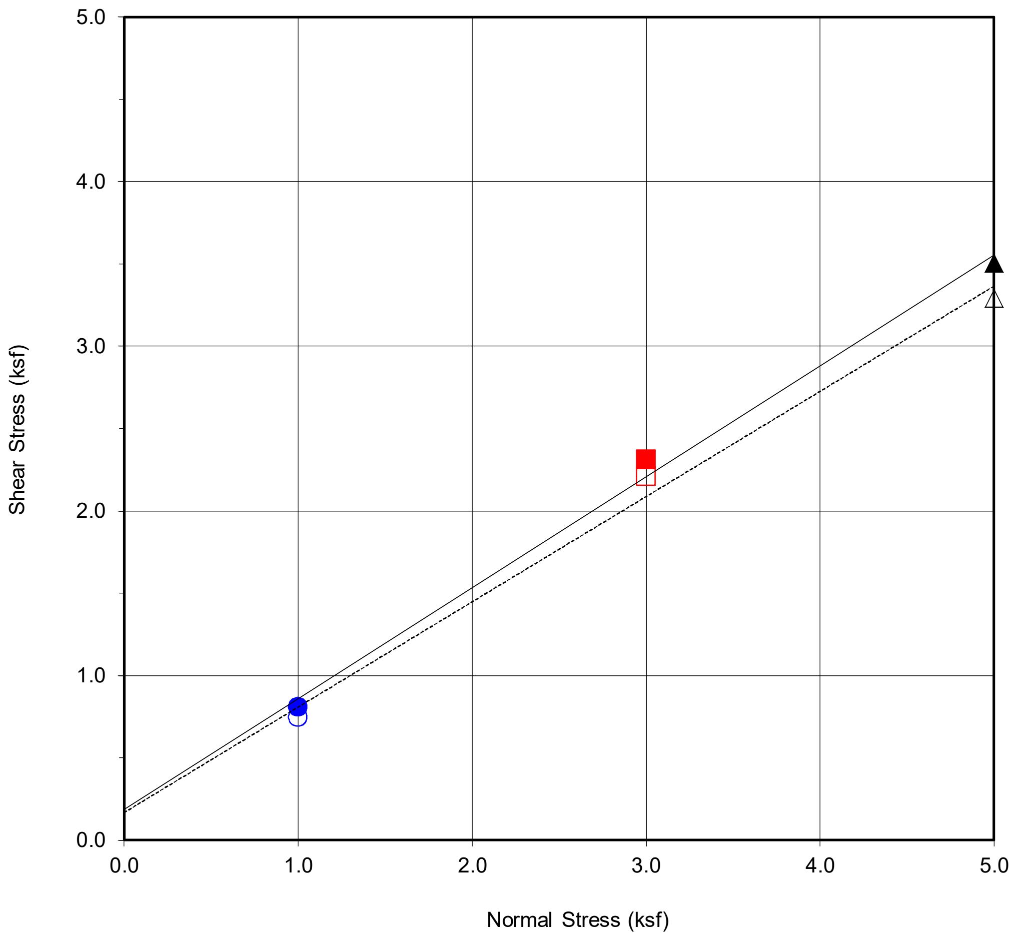
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B2



Boring No.	**B3**
Sample No.	**B3@5**
Depth (ft)	**5**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.81	■ 2.31	▲ 3.50
Shear Stress @ End of Test (ksf)	○ 0.75	□ 2.21	△ 3.29
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.0	14.3	12.7
Initial Dry Density (pcf)	113.6	111.1	114.6
Initial Degree of Saturation (%)	72.5	74.9	72.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.0	15.5	14.6

Soil Identification:

Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	185	34.0
Ultimate	168	32.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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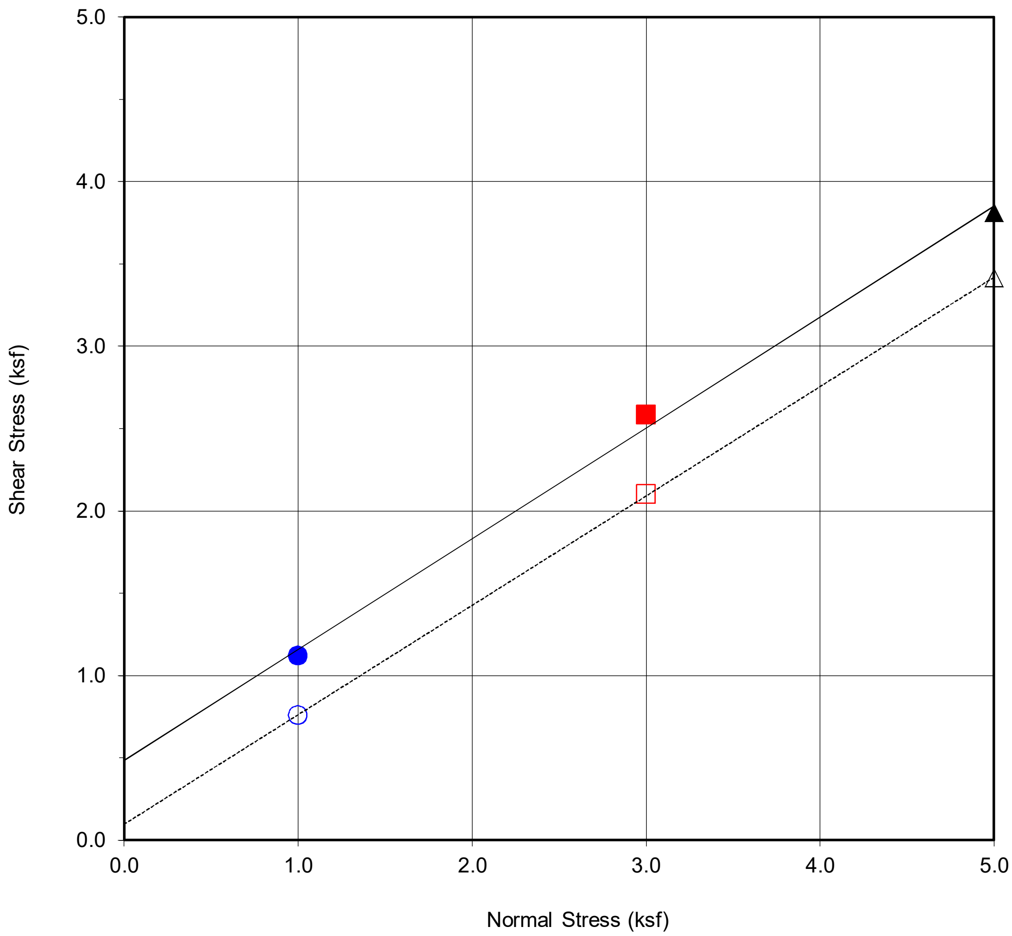
195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B3



Boring No.	**B4**
Sample No.	**B4@5.5**
Depth (ft)	**5.5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	484	34.0
Ultimate	96	33.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.12	■ 2.58	▲ 3.81
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.10	△ 3.42
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.8	15.7	15.3
Initial Dry Density (pcf)	113.8	114.4	116.3
Initial Degree of Saturation (%)	88.6	89.2	91.9
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.2	16.2	15.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

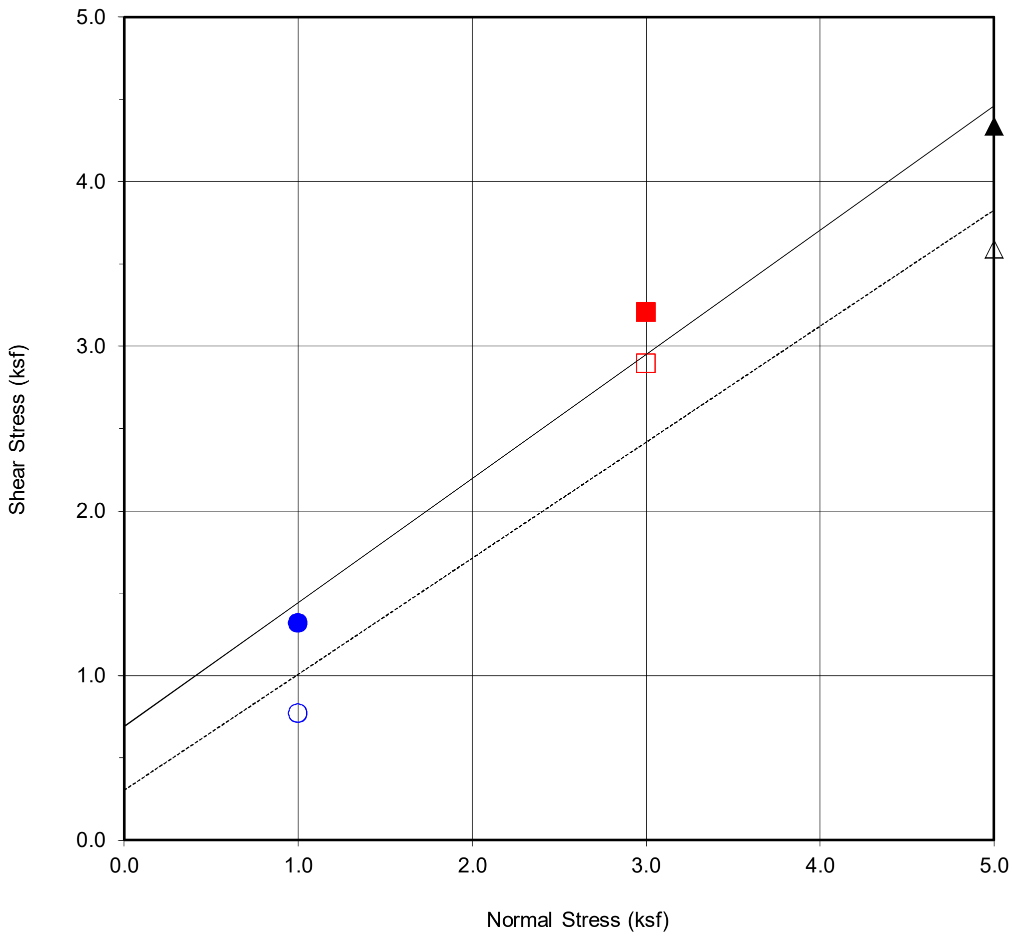
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B4



Boring No.	B5
Sample No.	B5@15'
Depth (ft)	15'
Sample Type:	Ring

Soil Identification:		
Clayey Sand (SC)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	688	37.0
Ultimate	303	35.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.32	■ 3.20	▲ 4.33
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.89	△ 3.59
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.9	14.4	15.1
Initial Dry Density (pcf)	108.2	113.2	110.4
Initial Degree of Saturation (%)	77.0	79.4	77.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.2	16.6	17.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

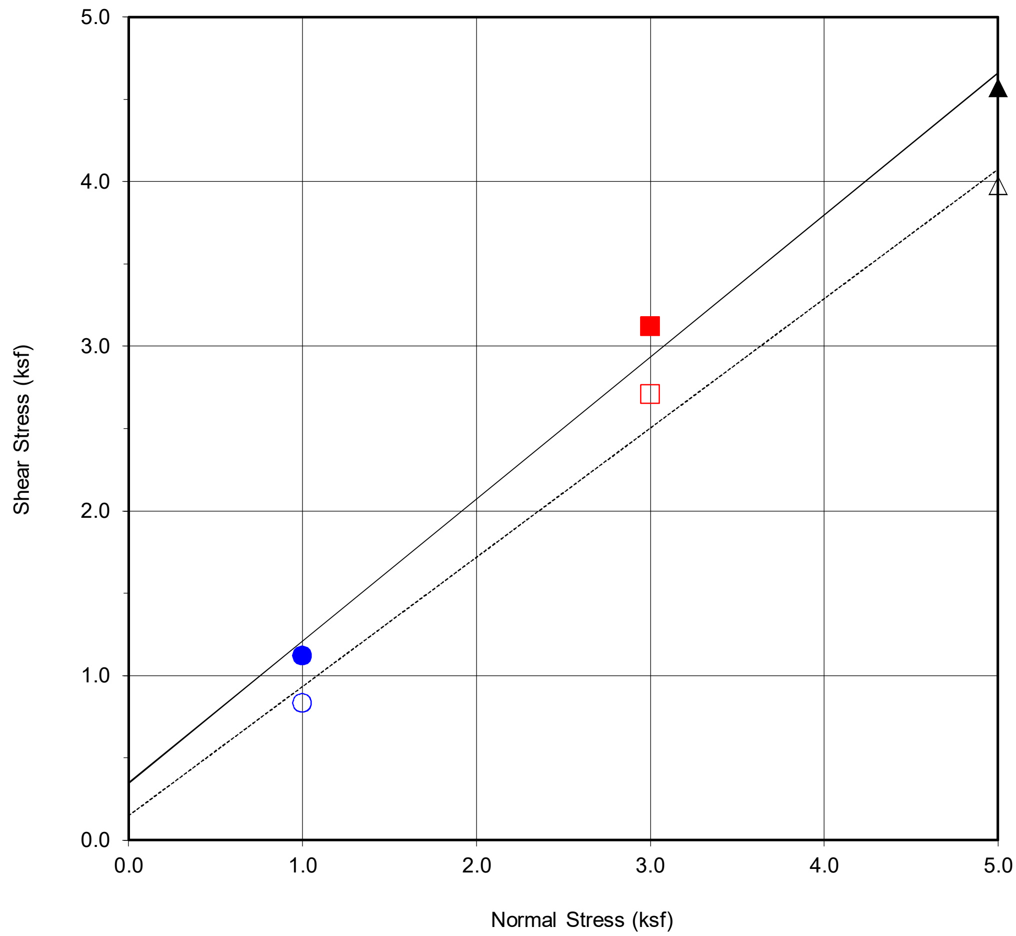
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B5



Boring No.	**B5**
Sample No.	**B5@18.5**
Depth (ft)	**18.5'**
Sample Type:	Ring

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.12	■ 3.12	▲ 4.57
Shear Stress @ End of Test (ksf)	○ 0.83	□ 2.71	△ 3.97
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	14.8	14.1	14.8
Initial Dry Density (pcf)	108.4	109.7	111.7
Initial Degree of Saturation (%)	71.9	70.7	78.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.4	17.4	16.6

Soil Identification:

Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	346	40.8
Ultimate	148	38.1

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

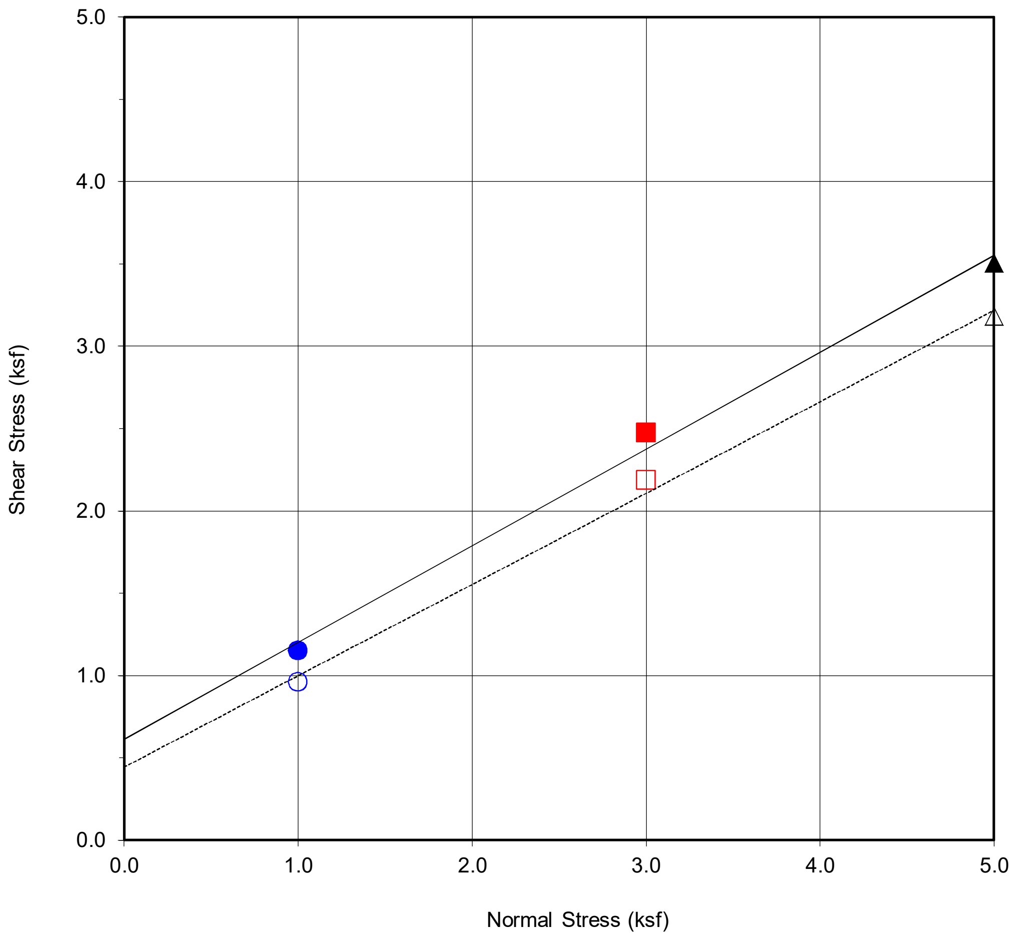
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B6



Boring No.	B6
Sample No.	B6@9'
Depth (ft)	9
Sample Type:	Ring

Soil Identification:		
Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	612	30.5
Ultimate	443	29.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.15	■ 2.47	▲ 3.50
Shear Stress @ End of Test (ksf)	○ 0.96	□ 2.18	△ 3.18
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	22.1	21.6	23.1
Initial Dry Density (pcf)	94.7	96.0	101.4
Initial Degree of Saturation (%)	76.4	77.1	94.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	27.0	25.3	23.7

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

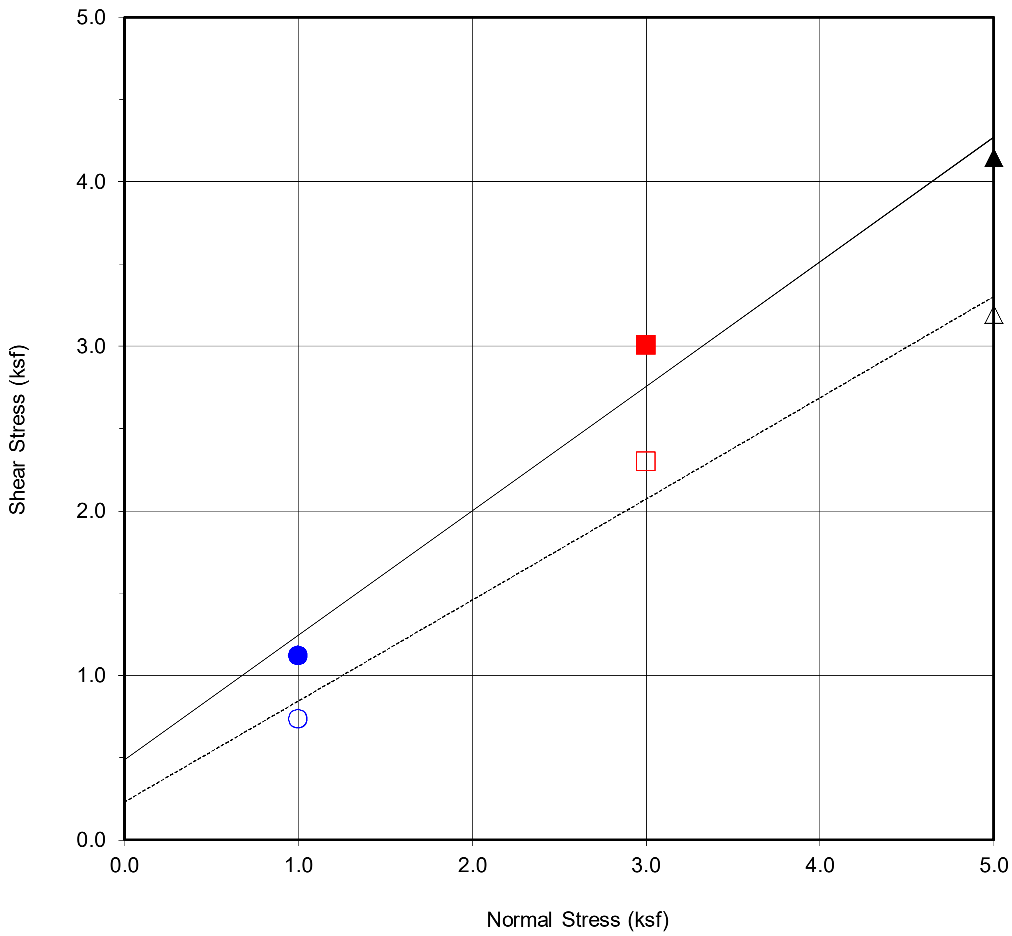
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B7



Boring No.	B6
Sample No.	B6@14.5'
Depth (ft)	14.5'
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	485	37.1
Ultimate	230	31.6

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.12	■ 3.01	▲ 4.15
Shear Stress @ End of Test (ksf)	○ 0.73	□ 2.30	△ 3.19
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	15.2	14.0	15.5
Initial Dry Density (pcf)	108.3	112.3	111.4
Initial Degree of Saturation (%)	73.8	75.7	81.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.8	16.7	17.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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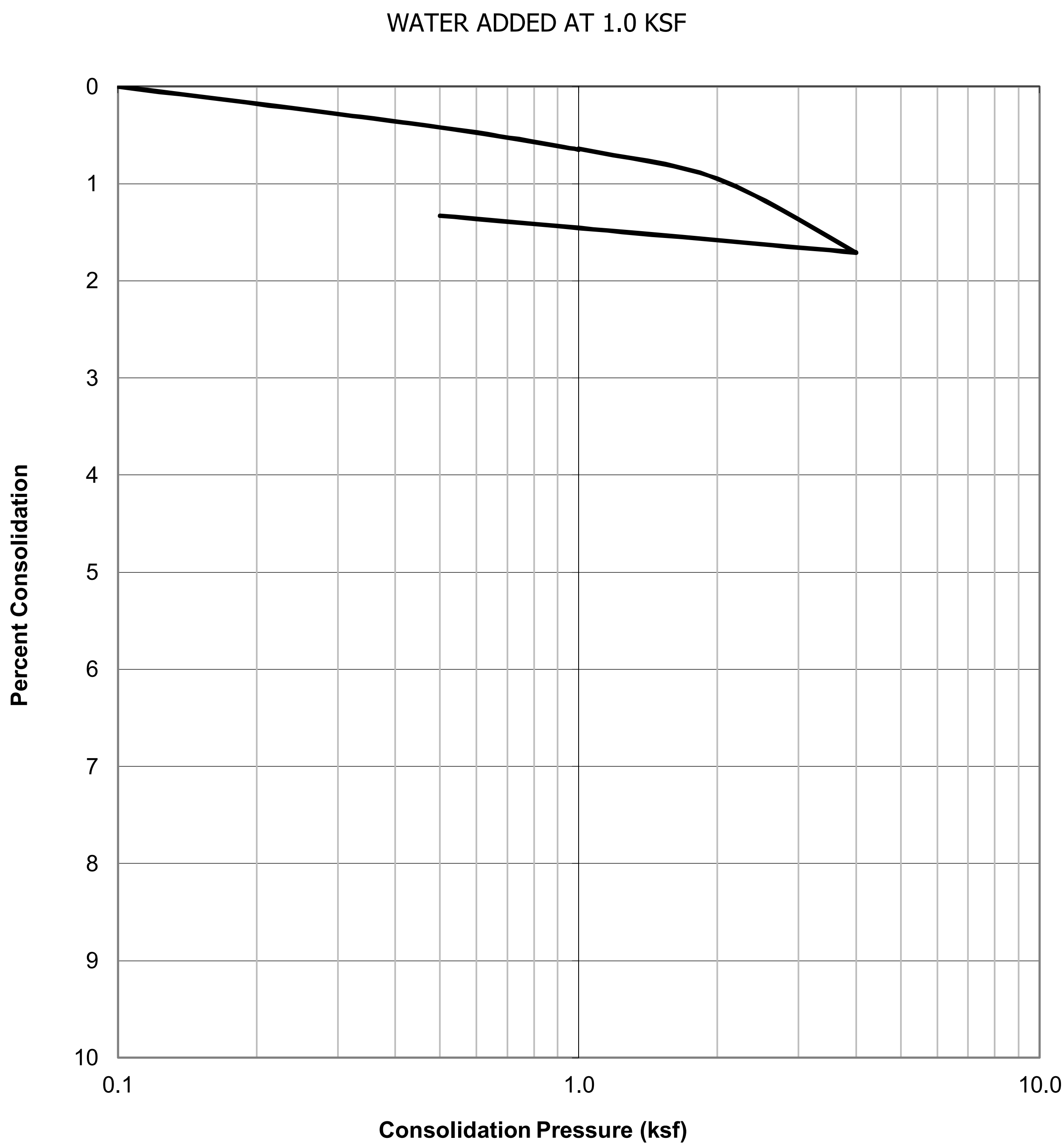
Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

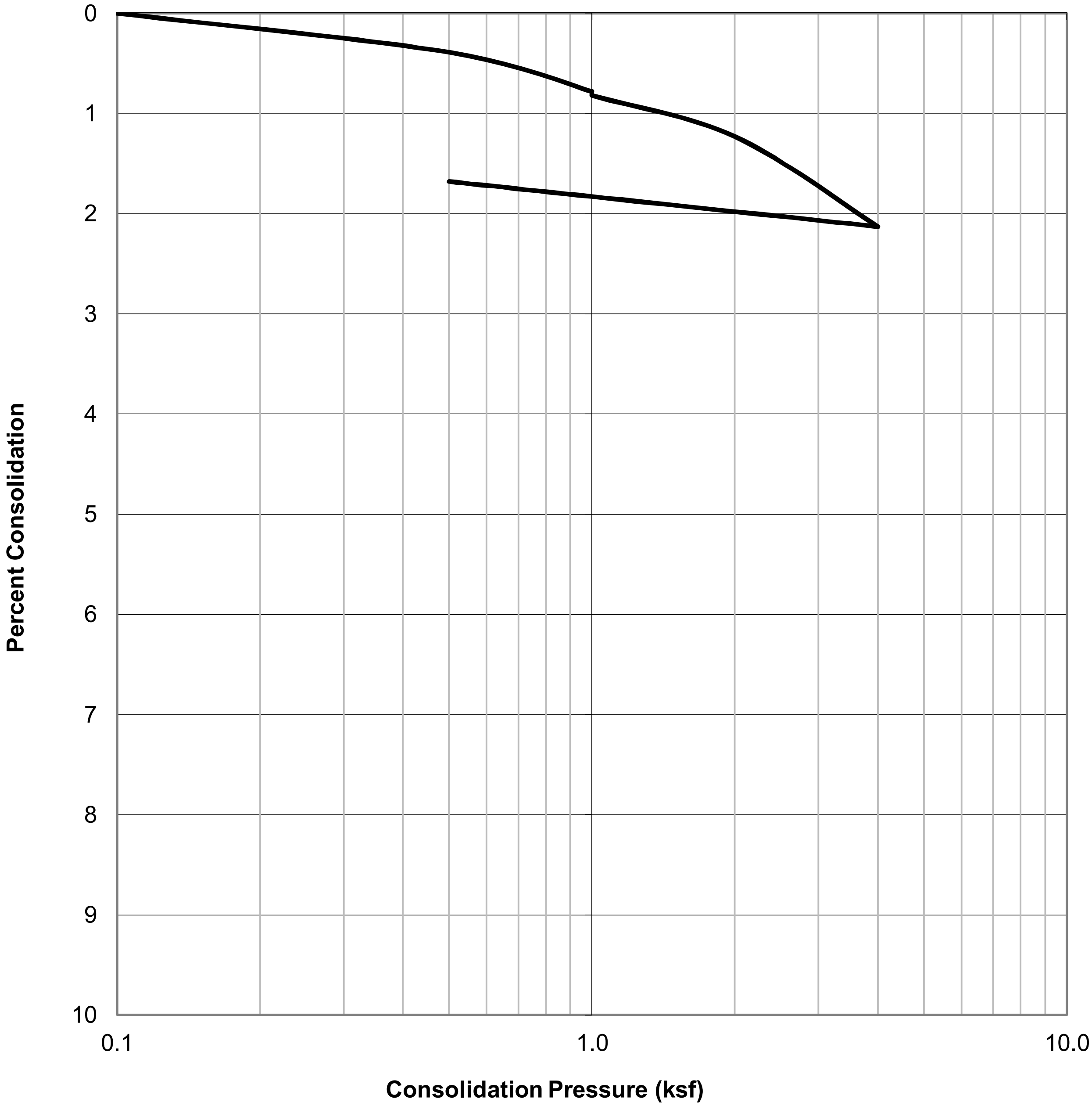
JULY, 2021



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@3.5	Silty Sand (SM)	117.7	14.6	15.1

GEOCON	**CONSOLIDATION TEST RESULTS** ASTM D-2435	Project No.: A8575-06-24
		195 HIGHLAND SPRINGS AVENUE BEAUMONT, CALIFORNIA
	Checked by: JJK	JULY, 2021 Figure B9

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5.5	Silty Sand (SM)	113.9	16.3	17.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: A8575-06-24

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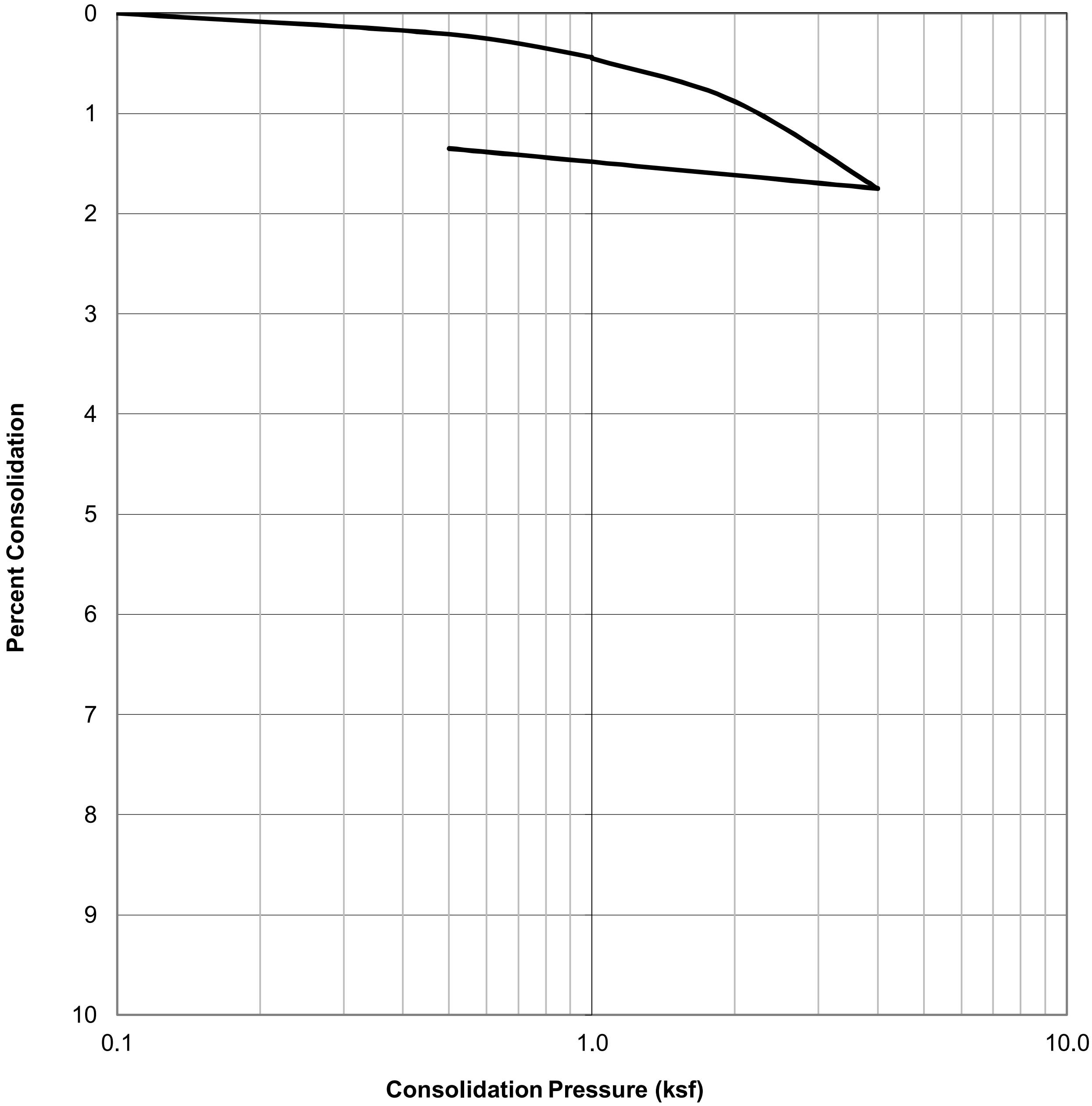
BEAUMONT, CALIFORNIA

JULY, 2021

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@7.5	Silty Sand (SM)	106.3	18.7	19.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
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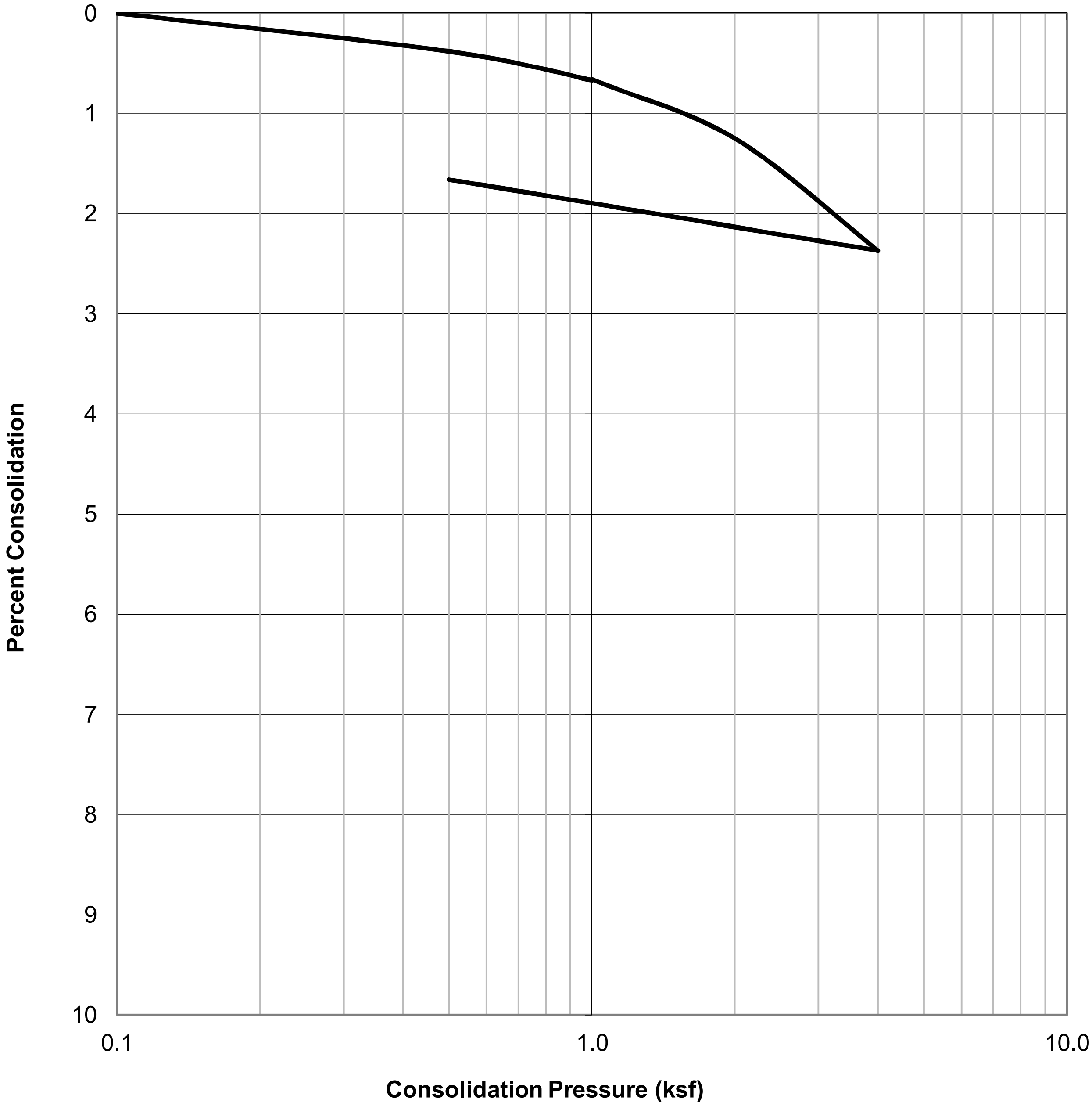
Figure B11

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10	Silt (ML)	98.6	22.7	24.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

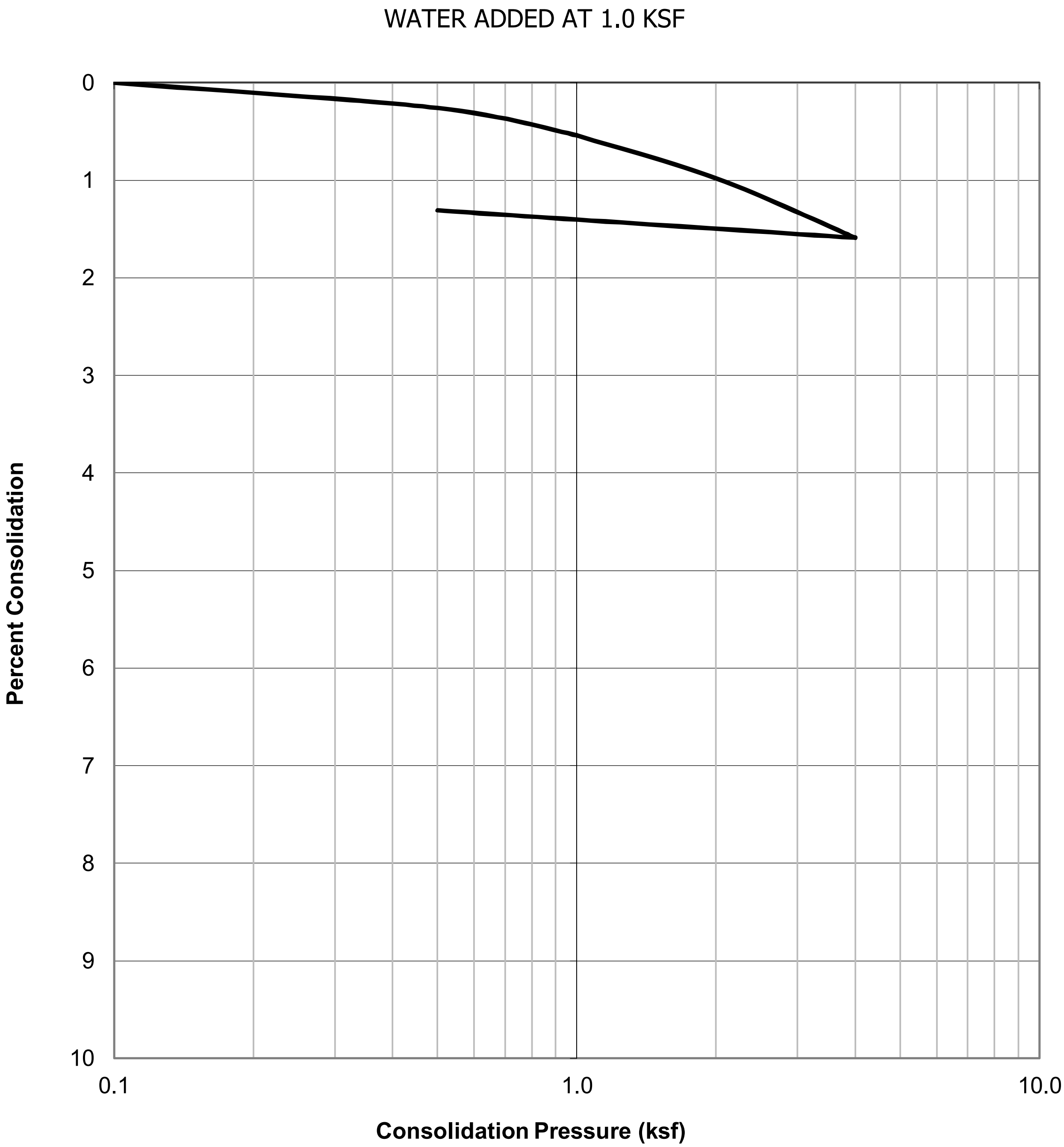
Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B12



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@5.5	Silty Sand (SM)	119.8	13.9	14.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

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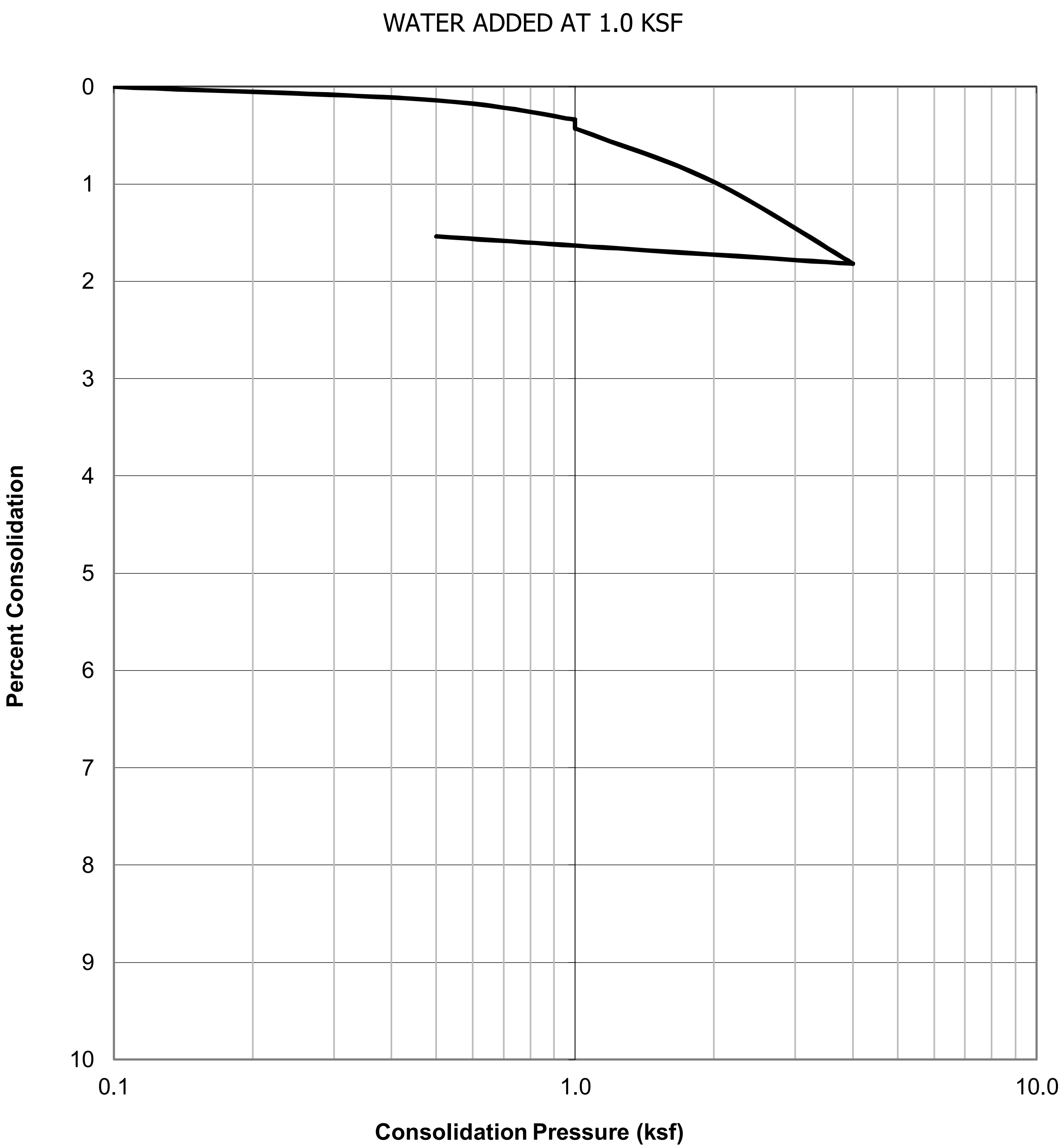
195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@7.5	Silty Sand (SM)	111.6	12.2	15.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

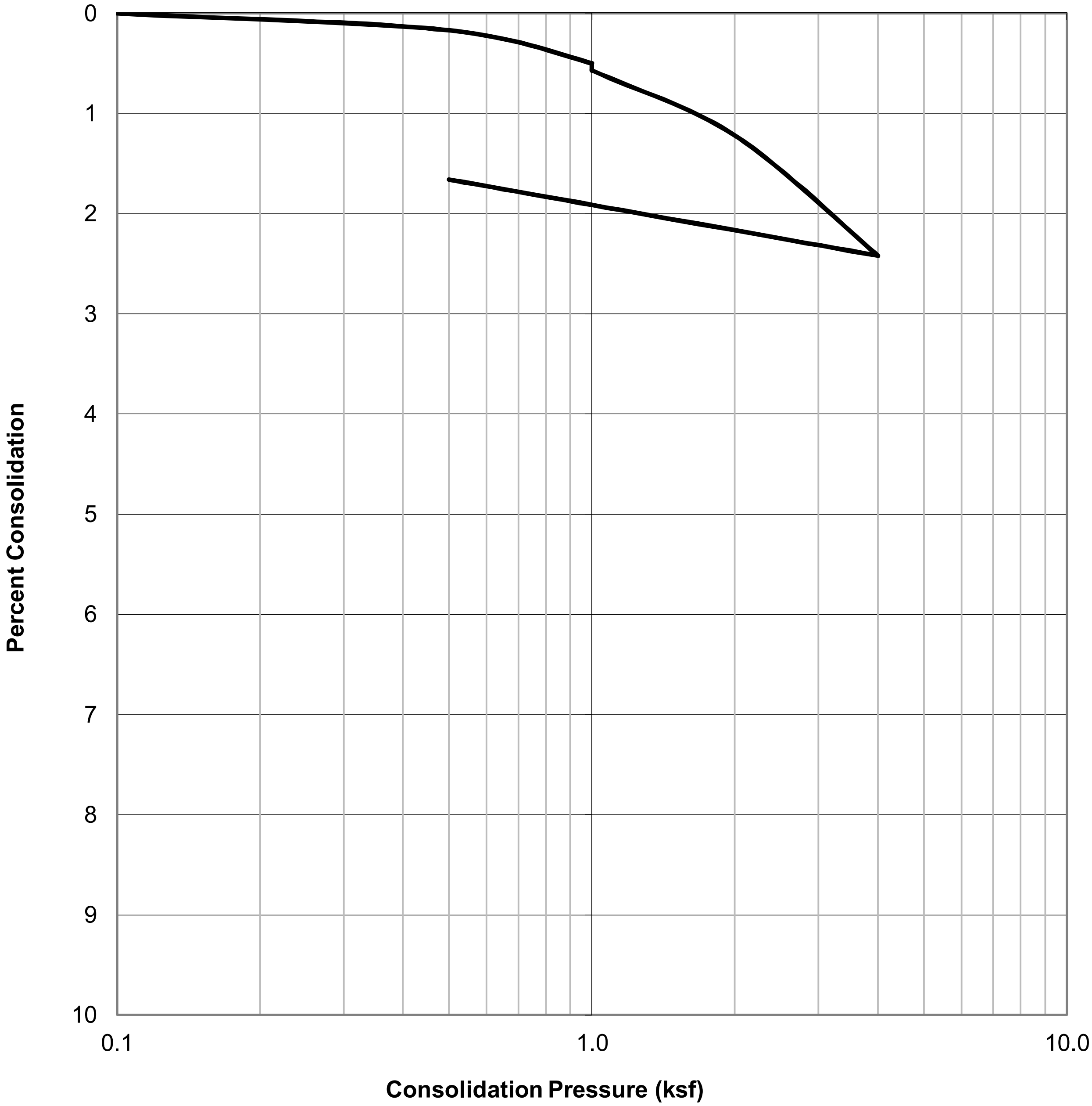
Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

Figure B14

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@10.5	Silt (ML)	97.5	23.2	25.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JJK

Project No.: A8575-06-24

195 HIGHLAND SPRINGS AVENUE
BEAUMONT, CALIFORNIA

JULY, 2021

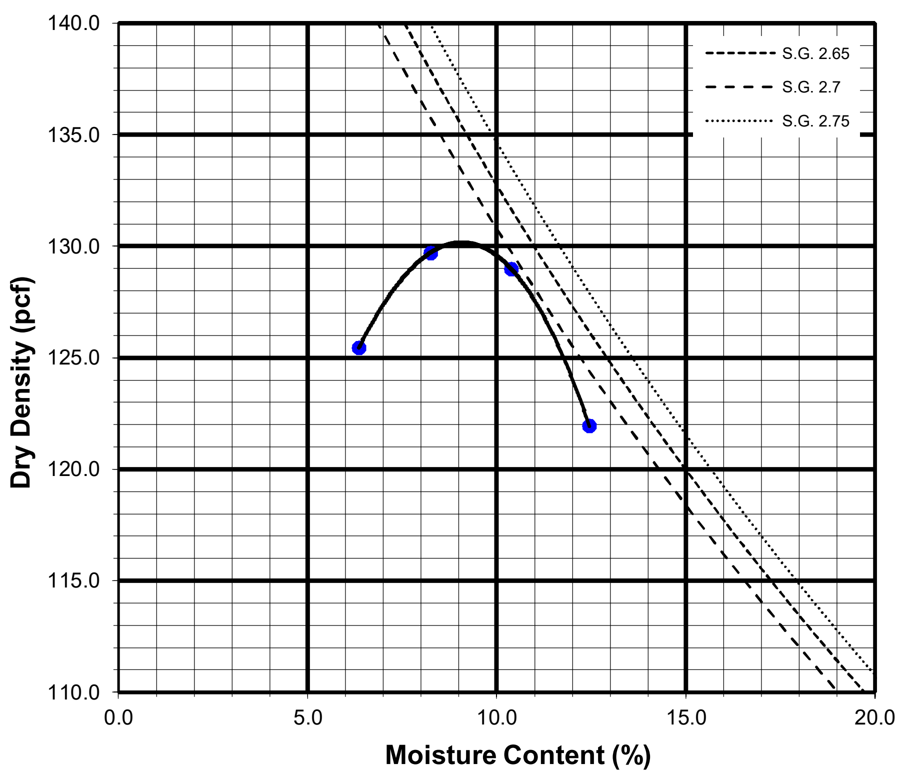
Figure B15

Sample No:

B4@0-5'	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6159	6265	6294	6215		
Weight of Mold	(g)	4144	4144	4144	4144		
Net Weight of Soil	(g)	2015	2121	2151	2071		
Wet Weight of Soil + Cont.	(g)	679.8	717.5	754.0	649.8		
Dry Weight of Soil + Cont.	(g)	648.0	674.0	696.7	591.8		
Weight of Container	(g)	147.3	146.2	144.7	125.8		
Moisture Content	(%)	6.4	8.2	10.4	12.4		
Wet Density	(pcf)	133.4	140.4	142.4	137.1		
Dry Density	(pcf)	125.4	129.7	129.0	121.9		

Maximum Dry Density (pcf)	**130.5**	**Optimum Moisture Content (%)**	**9.5**



Preparation Method: A

GEOCON	**MODIFIED COMPACTION TEST OF SOILS** ASTM D-1557	Project No.: A8575-06-24
		195 HIGHLAND SPRINGS AVENUE BEAUMONT, CALIFORNIA
	Checked by: JJK	JULY, 2021 Figure B16

SUMMARY OF LABORATORY POTENTIAL
OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	**pH**	**Resistivity (ohm centimeters)**
B4 @ 0-5'	7.3	4200 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B4 @ 0-5'	0.005

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B4 @ 0-5'	0.003	S0