APPENDIX G:
Geotechnical Report

Project No. E8940-04-01
August 15, 2016

Jeff Katz Architecture
6353 Del Cerro Boulevard
San Diego, California 92120

Attention: Mr. Jeff Katz

Subject: COASTSIDE FIRE STATION NO. 41
OBISPO ROAD AND AVENUE ALHAMBRA
HALF MOON BAY, CALIFORNIA
GEOTECHNICAL INVESTIGATION

Dear Mr. Katz:

In accordance with your authorization of our proposal dated July 8, 2015, we have performed a geotechnical investigation for the subject Coastside Fire Protection District project in Half Moon Bay, California. Our investigation was performed to observe the soil and geologic conditions that may impact site development and construction for the project as presently planned. The accompanying report presents the results of our investigation and conclusions and recommendations pertaining to the geotechnical aspects of the proposed project. The findings of this study indicate the site is suitable for development as planned provided the recommendations of this report are 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 at your convenience.

Sincerely,

GEOCON CONSULTANTS, INC.

DRAFT

Shane Rodacker, GE
Senior Engineer

(1/e-mail) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed new Coastside Fire Station No. 41 in Half Moon Bay, California (see Vicinity Map, Figure 1). The purpose of this investigation was to evaluate the subsurface soil and geologic conditions in the area of planned development and provide conclusions and recommendations pertaining to the geotechnical aspects of project design and construction, based on the conditions encountered during our study.

The scope of this investigation included field exploration, laboratory testing, engineering analysis, and the preparation of this report. Our field exploration was performed on July 13, 2016 and included 5 soil borings to maximum depths of approximately 40 feet or less at the site. The locations of our exploratory borings are depicted on the Site Plan, Figure 2. A detailed discussion of our field investigation and soil boring logs are presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent geotechnical parameters. Appendix B presents the laboratory test results in tabular format and graphical format.

The opinions expressed 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 CONDITIONS AND PROJECT DESCRIPTION

The project is proposed on the northwestern side of Coronado Street between Obispo Road and Avenue Alhambra in Half Moon Bay. Topographically, the irregularly-shaped site slopes moderately to the southwest with ground surface elevations on the order of 45 to 50 feet MSL along Avenue Alhambra and 30 to 35 feet MSL along Obispo Road, according to web-based mapping. The site is generally undeveloped with native grasses and a few mature trees. Overhead utility lines are present along the bordering streets.

The information provided by Jeff Katz Architecture indicates the new fire station will be situated approximately 350 feet north of Coronado Street with associated driveways and parking areas to the northwest and southeast of the new fire station building. Details of the fire station building were not provided but we anticipate the structure will be one to two stories with no significant underground levels.

Grading plans were not provided but we anticipate site grading will be significant with cuts and fills on the order of 10 feet or less to create a building pad and establish rough subgrade for pavement and parking areas. Grade breaks resulting from cuts on the northeastern side of the site (along Avenue Alhambra) will be accomplished with site retaining walls on the order of 10 feet in height.

3. GEOLOGIC SETTING

Half Moon Bay is located within the Coast Ranges Geomorphic Province of California, which is characterized by a series of northwest trending mountains and valleys along the north and central coast of California. Topography is controlled by the predominant geological structural trends within the Coast Range that generally consist of northwest trending synclines, anticlines and faulted blocks. The dominant structure is a result of both active northwest trending strike-slip faulting, associated with the San Andreas Fault system, and east-west compression within the province.

The San Andreas Fault (SAF) is a major right-lateral strike-slip fault that extends from the Gulf of California in Mexico to Cape Mendocino in northern California. The SAF forms a portion of the boundary between two tectonic plates on the surface of the earth. To the west of the SAF is the Pacific Plate, which moves north relative to the North American Plate, located east of the fault. In the San Francisco Bay Area, movement across this plate boundary is concentrated on the SAF and also distributed, to a lesser extent, across a number of other faults including the Hayward, Calaveras and Rodgers Creek faults, among others. Together, these faults are referred to as the SAF system.

Basement rock west of the SAF is generally granitic, while to the east it consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. Both are typically Jurassic to Cretaceous in age (205 to 65 million years old). Overlying the basement rocks are Cretaceous (about 140 to 65 million years old) marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks have typically been extensively folded and faulted largely as a result of movement along the SAF system, which has been ongoing for about the last 25 million years, and regional compression during the last about 4 million years. The inland valleys, as well as the structural depression within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary age (about the last 1.6 million years). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay and gravel, while the bay deposits typically consist of soft organic-rich silt and clay (bay mud) or sand.

Available geologic information published by the United States Geological Survey (USGS) indicates the suite is underlain by Pleistocene-age marine terrace deposits.

4. GEOLOGIC HAZARDS

4.1 Faulting and Seismicity

The site is not located within an Alquist-Priolo Earthquake Fault Zone as established by the State of California around known active faults. A review of the referenced geologic materials and our knowledge of the general area indicate that the site is not underlain by active faults.

To determine the distance to known active faults within 100 miles of the site, we used the computer program *EQFAULT*. Site latitude is 37.5009° N; site longitude is -122.4681° W. Active faults within 30 miles of the site are summarized in Table 4.1.

TABLE 4.1
REGIONAL FAULT SUMMARY

Fault Name	Distance to Site (miles)	Maximum Earthquake Magnitude, M_w
San Gregorio	1.2	7.3
San Andreas (Peninsula)	5.8	7.1
San Andreas (1906)	5.8	7.9
Monte-Vista Shannon	12.3	6.8
San Andreas (North Coast)	22.7	7.6
Hayward (South)	24.3	7.1
Hayward (Total Length)	24.3	7.1
Hayward (North)	24.5	6.9

The faults tabulated above are sources of potential ground motion. However, earthquakes that might occur on other faults within northern and central California are also potential generators of significant ground motion and could subject the site to intense ground shaking.

4.2 Surface Fault Rupture

The site is not within a currently established State of California Earthquake Fault Zone for surface fault rupture hazards. No active or potentially-active faults are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. The California Geological Survey defines an active fault as a fault that shows evidence for activity within the last 11,000 years. A potentially active fault is generally defined as a fault that has shown evidence of displacement between 11,000 and 1.6 million years ago. Faults that have not demonstrated evidence of movement with the past 1.6 million years are generally considered inactive.

4.3 Ground Shaking

We used the USGS web-based application *2008 Interactive Deaggregations* that is based on various NGA (New Generation Attenuation) models. We estimated the peak ground acceleration (PGA) and modal (most probable) magnitude associated with a 2,475-year return period. This return period corresponds to an event with 2% chance of exceedance in a 50-year period. The USGS-estimated PGA is 0.90g and the modal magnitude is 7.4 for Seismic Site Class D (estimated V_s30 of 295 m/sec).

While listing PGA is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including frequency and duration of motion and soil conditions underlying the site.

4.4 Liquefaction

The site is not located within a State of California Seismic Hazard Zone Hazard Zone for liquefaction. Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with intense earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction of granular soils is generally limited to the upper 50 feet of a soil profile.

Web-based mapping by the USGS indicates the subject site possesses a "moderate" susceptibility to liquefaction. Our liquefaction analysis identified a potentially liquefiable sand layer at Boring B3. The layer is located below a depth of approximately 23 feet and appears to be less than approximately 2 feet in thickness. Consequences of liquefaction can include ground surface settlement, ground loss (sand boils) and lateral slope displacements (lateral spreading). For liquefaction-induced sand boils or fissures to occur, pore water pressure induced within liquefied strata must exert enough force to break through overlying, non-liquefiable layers. Based on methodology recommended by Youd and Garris (1995), which modified and advanced original research by Ishihara (1985), a capping layer of non-liquefiable soil can prevent the occurrence of sand boils and fissures. In our opinion, based on the presence of the clay layer that mantles the abutment areas and the depth to significant liquefiable layers, the potential for ground loss due to sand boils or fissures at the existing abutment areas is considered low.

A likely consequence of potential liquefaction at the site is ground surface settlement. We evaluated the potential for liquefaction and resultant settlements at the site using the soil boring data and the methodology of Youd et. al. (2001). We used a ground motion of 0.88g as required by 2013 California Building Code (CBC) and related publications, an earthquake moment magnitude (M_w) of 7.4, and a groundwater depth of 13 feet. If liquefaction were to occur, we estimate that it may result in total foundation settlements on the order of ½ inch or less.

4.5 Landslides

There are no known landslides near the site nor is the site in the path of any known or potential landslides. We did not observe overt indications of landslide or slope instability during our site reconnaissance. We do not consider the potential for a landslide to be a significant hazard to this project.

4.6 Tsunamis and Seiches

Based on mapping by the California Emergency Management Agency, the site would be inundated by run-up during an extreme tsunami. The potential for inundation should be considered in project planning and design.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Flooding from a seismically-induced seiche is considered unlikely.

5. SOIL AND GROUNDWATER CONDITIONS

5.1 Terrace Deposits

Our soil borings encountered marine terrace deposits observed to be medium stiff to hard silts and clays with variable amounts of sand and medium dense to very dense fine to coarse sands with variable amounts of clay. Based on our laboratory testing, some of the clays encountered in our borings are highly plastic and may possess significant expansion potential. Our borings encountered Terrace Deposits to the maximum depth explored – approximately 40 feet below existing grade.

5.2 Groundwater

Groundwater was encountered in Boring B2 and B3 at depths of approximately 23 feet and 13 feet, respectively. Groundwater levels will vary seasonally and fluctuate with variations in rainfall, temperature and other factors and may be higher or lower than observed during our study.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 It is our opinion that neither soil nor geologic conditions were encountered during our investigation that would preclude the project as presently proposed.

6.1.2 A key geotechnical consideration for the project is potentially expansive nature of clayey soils within the native terrace deposits. The recommendations provided below are intended to mitigate the potential effects of soil expansion.

6.1.3 All references to relative compaction and optimum moisture content in this report are based on ASTM D 1557 (latest edition).

6.1.4 Updated project civil and structural plans should be provided for our review. Supplemental recommendations and/or modifications to the recommendations presented herein may be required.

6.1.5 Based on site topography, we anticipate that site grading may create a cut-fill transition within the building pad. Recommendations to mitigate the potential effects of the cut-fill transition (primarily the potential for adverse differential settlements) are provided herein.

6.1.6 Provided the site is graded in accordance with the recommendations of this report and foundation systems are constructed as described herein, we estimate that post-construction settlement due foundation loads will be less than approximately ¾ inch, and corresponding differential settlement will be less than ½ inch across a horizontal distance of 50 feet. Final design foundation loadings should be reviewed by Geocon. In addition to the settlement estimates above, site improvements should be designed to accommodate up to ½ inch of seismically-induced settlement across a horizontal distance of 50 feet.

6.1.7 Any changes in the design, location or elevation of the proposed improvements, 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.

6.2 Seismic Design Criteria

6.2.1 We understand that seismic structural design will be performed in accordance with the provisions of the 2013 CBC which is based on the American Society of Civil Engineers (ASCE) publication *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10). We used the USGS web-based application *US Seismic Design Maps* to evaluate site-specific seismic design parameters in accordance with the 2013 CBC and ASCE 7-10. Results are summarized in Table 6.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 6.2.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2013 CBC / ASCE 7-10 Reference
Site Class	D	Section 1613.3.2/ Table 20.3-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.288g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.987g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.0	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.5	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.288g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	1.481g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.525g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.987g	Eq. 16-40 / Eq. 11.4-4

6.2.2 Table 6.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 6.2.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.88g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.88g	Section 11.8.3 (Eq. 11.8-1)

6.2.3 Conformance to the criteria presented in Tables 6.2.1 and 6.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

6.3 Soil and Excavation Characteristics

6.3.1 Based on the soils conditions encountered in our exploratory borings, the majority of onsite soils can be excavated with moderate to heavy effort using conventional excavation equipment. We do not anticipate excavations in the native residual soils will generate oversize material (greater than 6 inches in nominal dimension).

6.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable Occupational Safety and Health Administration (OSHA) rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

6.3.3 Some of the site soils should be considered expansive as defined by 2013 CBC. The recommendations presented in this report assume that foundations for the project will derive support in properly compacted fills.

6.4 Materials for Fill

6.4.1 Excavated soils generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they do not contain deleterious matter, organic material, or cementations larger than 6 inches in maximum dimension.

6.4.2 Import or low-expansive material should be well-graded, primarily granular with a "very low" expansion potential (Expansion Index less than 20), a Plasticity Index less than 15, be free of organic material and construction debris, and not contain rock larger than 6 inches in greatest dimension.

6.4.3 Environmental characteristics and corrosion potential of import soil materials may also be considered. Proposed import materials should be sampled, tested, and approved by Geocon prior to its transportation to the site.

6.5 Grading

6.5.1 All earthwork should be observed and all fills tested for recommended compaction and moisture content by representatives of Geocon.

6.5.2 Structural building pad areas should be considered as areas extending a minimum of 5 feet horizontally beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads.

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

6.5.4 Site preparation should commence with stripping existing vegetation and any organic-laden topsoil. The root balls for all trees and shrubs should be grubbed to remove all roots greater than approximately 1 inch in diameter. Organics generated by clearing and grubbing should not be used within fills in structural areas.

6.5.5 Although not expected, any active or inactive utilities within the construction area should be protected, relocated, or abandoned. Any pipelines to be abandoned that are greater than 2 inches and less than 18 inches in diameter should be removed or filled with sand-cement slurry. Utilities larger than 18 inches in diameter should be removed. Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.5.6 The cut portion of the building pad for the fire station should be over-excavated to a depth of approximately 2 feet below the bottom of footings. The resultant bottom surface should be scarified to a depth of at least 12 inches and compacted to at least 90% relative compaction at least 2% above moisture content (at least 92% relative compaction near optimum where predominantly sandy). In general, remedial grading should result in at least three feet of properly compacted fill materials (including scarified and recompacted bottoms) across the building pad. Due to the expansive nature of site soils, the upper 18 inches of subgrade for the fire station should be comprised of low-expansive fill as defined in Section 6.4.2.

6.5.7 All structural fill (including scarified ground surfaces and backfill) should be placed in layers no thicker than will allow for adequate bonding and compaction (typically 8 inches). Fill soils should be placed, moisture conditioned to at least 2% above optimum moisture content (near optimum where sands and gravels) and compacted to at least 90% relative compaction (at least 92% relatively compaction where sands and gravels). Fill areas with in-place density tests showing moisture contents less than optimum moisture content may require additional moisture conditioning prior to placing additional fill.

6.5.8 If grading commences in winter or spring, or in periods of precipitation, excavated and in-place soils may be, or become, wet. Earthwork contractors should be aware of moisture sensitivity of fine-grained soils and potential compaction/workability difficulties. It has been our experience the subgrade soils protected by pavement are typically moist to wet and may require significant drying prior to re-use as engineered fill. The most effective site preparation alternatives will depend on site conditions prior to and during grading operations; we should evaluate site conditions at those times and provide supplemental recommendations, if necessary.

6.6 Temporary Excavations

6.6.1 We anticipate that much of the native terrace deposits can be considered a Type B soil in accordance with OSHA guidelines. If free water, clean and/or loose sandy soils or undocumented fills are encountered the materials should be downgraded to Type C. The contractor should have a “competent person” as defined by OSHA evaluate all excavations. 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 and possibly shoring.

6.6.2 It is the contractor’s responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.7 Shallow Foundations

6.7.1 The fire station building may use conventional shallow foundations consisting of continuous strip and isolated spread footings bearing entirely in competent terrace deposits or entirely in properly compacted fill following the remedial grading discussed in Section 6.5. The following recommendations are based on the assumption that the soils within 5 feet of finish grade will possess a very low to high expansion potential (Expansion Index less than 130).

6.7.2 It is recommended that strip and spread footings have a minimum embedment depth of 24 inches below lowest adjacent pad grade. The footings should be at least 12 inches wide. Footings should be founded such that outside edge of footing bottoms are at least 10 feet horizontally from any slope face.

6.7.3 Footings proportioned as recommended may be designed for an allowable soil bearing pressure of 3,000 pounds per square foot (psf). The allowable bearing pressure is for dead + live loads may be increased by up to one-third for transient loads due to wind or seismic forces.

6.7.4 The allowable passive pressure used to resist lateral movement may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf) for footings poured neat against properly compacted fills or undisturbed natural soils. The allowable passive pressure assumes a horizontal surface extending at least 5 feet or 3 times the surface generating the passive pressure, whichever is greater. The allowable coefficient of friction to resist sliding is 0.30 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%. Where not protected by flatwork or pavement, the upper 1 foot of soil should be neglected when calculating passive resistance to lateral loads.

6.7.5 Minimum reinforcement for continuous footings should consist of four No. 5 steel reinforcing bars; two placed near the top of the footing and two near the bottom.

6.7.6 Underground utilities running parallel to footings should not be constructed in the zone of influence of footings. The zone of influence may be taken to be the area beneath the footing and within a 1:1 plane extending out and down from the bottom edge of the footing.

6.7.7 The foundation subgrade should be sprinkled as necessary to maintain a moist condition without significant shrinkage cracks as would be expected in any concrete placement. Prior to placing rebar reinforcement, foundation excavations should be evaluated by our representatives for appropriate support characteristics and moisture content. Moisture conditioning may be required for the materials exposed in footing excavations, particularly if foundation excavations are left open for an extended period.

6.8 Underground Utilities

6.8.1 Underground utility trenches should be backfilled with properly compacted material. The material excavated from the trenches should be adequate for use as backfill provided it does not contain deleterious matter, vegetation or rock larger than six inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding eight inches and should be compacted to at least 90% relative compaction at least 2% above optimum moisture content (near optimum where sands and gravels).

6.8.2 Bedding and pipe zone backfill typically extends from the bottom of the trench excavations to a minimum of 6 inches above the crown of the pipe. Pipe bedding and backfill material should conform to the requirements of the governing utility agency. Proposed bedding and pipe zone materials should be reviewed by Geocon prior to construction; materials such as ¾-inch drain rock may require wrapping with filter fabric to mitigate the potential for piping.

6.9 Concrete Slabs-on-Grade

6.9.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in Section 6.12 of this report.

6.9.2 Slabs-on-grade should be underlain by at least 18 inches of low-expansive fill meeting the requirements of Section 6.4.2 to reduce the potential for slab distress due shrink/swell in the native expansive soils.

6.9.3 Concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 5 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

6.9.4 Interior slabs should be underlain by 3 inches of ½-inch or ¾-inch crushed rock with no more than 5% passing the No. 200 sieve to serve as a capillary break.

6.9.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Due to the expansive soil conditions, we recommend that at least 6 inches of Class 2 Aggregate Base (AB) compacted to at least 95% relative compaction be used below exterior concrete slabs and pavements. Prior to placing AB, the subgrade should be moisture conditioned to at least 2% over optimum and properly compacted to at least 90% relative compaction.

6.9.6 Crack control joints should be spaced at intervals not greater than 8 feet for 4-inch-thick slabs (10 feet for 5-inch slabs) and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.9.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to soil movement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to soil movement. This is common for project areas that contain expansive soils since designing to eliminate potential soil movement is cost prohibitive. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.10 Moisture Protection Considerations

6.10.1 A vapor barrier is not required beneath slab-on-grade for geotechnical purposes. Further, the migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for convenience of the design-build team, we are providing the following recommendations. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field. If a vapor barrier is used beneath mat slab foundations, the frictional contribution to sliding resistance should be neglected.

6.10.2 A vapor barrier meeting ASTM E 1745-09 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) should be used. The vapor barrier, if used, should extend to the edges of the slab, and should be sealed at all seams and penetrations.

6.10.3 The concrete water/cement ratio should be as low as possible. The water/cement ratio should not exceed 0.45 for concrete placed directly on the vapor barrier. Midrange plasticizers could be used to facilitate concrete placement and workability.

6.10.4 Proper finishing, curing, and moisture vapor emission testing should be performed in accordance with the latest guidelines provided by the American Concrete Institute, Portland Cement Association, and ASTM.

6.11 Pavement Recommendations

6.11.1 The upper 12 inches of pavement subgrade should be scarified, moisture conditioned to at least 2% over optimum and compacted to at least 92% relative compaction (near optimum and at least 95% relative compaction where predominantly sandy). Prior to placing aggregate base, the finished subgrade should be proof-rolled with a laden water truck (or similar equipment with high contact pressure) to verify stability.

6.11.2 We recommend the following asphalt concrete (AC) pavement sections for design to establish subgrade elevations in pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) based on anticipated traffic conditions. The flexible pavement sections below are based on estimated design TIs. We can provide additional sections based on other TIs if necessary.

TABLE 6.11
FLEXIBLE PAVEMENT SECTION RECOMMENDATIONS

Location	Estimated Traffic Index (TI)	AC (inches)	AB (inches)
Parking Stalls	4.5	3	8
Driveways	6.0	3½	12½
Heavy Duty	7.0	4½	15½
Heavy Duty	8.0	5	17½

Note: The recommended flexible pavement sections are based on the following assumptions:

1. Subgrade soil has an R-Value of 5.
2. AB: Class 2 AB with a minimum R-Value of 78 and meeting the requirements of Section 26 of the latest Caltrans Standard Specifications.
3. AB is compacted to 95% or higher relative compaction at or near optimum moisture content. Prior to placing AB, the subgrade should be proof-rolled with a loaded water truck to verify stability.
4. AC: Asphalt concrete conforming to local agency standards or Section 39 of the latest Caltrans Standard Specifications.

6.11.3 The AC sections in Table 6.11 are final, minimum thicknesses. If staged-pavements are used, the construction bottom AC lift should be at least 2 inches thick. Following construction, the finish top AC lift should be at least 1½ inches thick.

6.11.4 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, we recommend the concrete be a minimum of 8 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. In addition, doweling, reinforcing steel or other load-transfer mechanism should be provided at joints if desired to reduce the potential for vertical offset.

6.11.5 We recommend that at least 12 inches of Class 2 aggregate base be used below rigid concrete pavements. The aggregate base should be compacted to at least 95% relative compaction near optimum moisture content.

6.11.6 In general, we recommend that concrete pavements be designed, constructed and maintained in accordance with industry standards such as those provided by the American Concrete Pavement Association.

6.11.7 Crack control joints should be spaced at intervals not greater than 16 feet for 8-inch-thick slabs and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness and should be constructed using saw-cuts or other methods as soon as practical after concrete placement. Construction joints should be designed by the project structural engineer.

6.11.8 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 6 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving. Alternatives such as plastic moisture cut-offs or modified drop-inlets may also be considered in lieu of deepened curbs.

6.11.9 Consideration should be given to the use of edge drains or similar mechanisms to control subsurface water and mitigate the potential for pavement subgrade to become wet and unstable. In general, the uphill side of pavements constructed in a cut condition would be most susceptible to seepage infiltrating the pavement subgrade and aggregate base layer. If implemented the edge drains should be outlet to a controlled drainage facility or other location deemed suitable by the project civil engineer.

6.11.10 Asphalt pavement section recommendations for driveways and parking areas are based on the design procedures of Caltrans' Highway Design Manual (HDM). It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The HDM indicates that the resulting pavement sections for parking lots are minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems. It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.12 Retaining Wall Design

6.12.1 Lateral earth pressures may be used in the design of retaining walls and buried structures. Lateral earth pressures against these facilities may be assumed to be equal to the pressure exerted by an equivalent fluid. The unit weight of the equivalent fluid depends on the design conditions. Table 6.12 summarizes the weights of the equivalent fluid based on the different design conditions.

TABLE 6.12
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	45 pcf
At-Rest	65 pcf

6.12.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement such as basement walls should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and no surcharges within that same area.

Where backfill surfaces are inclined up to 2:1, an additional 15 pcf should be added to the equivalent fluid density values listed in Table 6.12. Unless project-specific loading information is provided by the structural engineer, where vehicle loads are expected atop the wall backfill, an additional uniform surcharge pressure equivalent to 2 feet of backfill soil should be used for design. Where the vehicle loading will be limited to passenger cars, the additional uniform surcharge equivalent may be reduced to 1 foot of backfill soil.

6.12.3 If deemed necessary by the project structural engineer or required by building code, retaining walls should be designed considering seismic lateral earth pressure. The seismic lateral earth pressure increment exerted on the wall should be a triangular distribution with a pressure of 25H (where H is the height of the wall, in feet, resulting in psf) exerted at the base of the wall and zero at the top of the wall.

6.12.4 Retaining walls greater than 2 feet tall (retained height) should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and should be waterproofed as required by the project architect. Positive drainage for retaining walls should consist of a vertical layer of permeable material positioned between the retaining wall and the soil backfill. The permeable material may be composed of a composite drainage geosynthetic or a natural permeable material such as crushed gravel at least 12 inches thick and capped with at least 12 inches of native soil. A geosynthetic filter fabric should be placed between the gravel and the soil backfill. Provisions for removal of collected water should be provided for either system by installing a perforated drainage pipe along the bottom of the permeable material which leads to suitable drainage facilities.

6.12.5 Retaining wall foundations may be designed in accordance with Section 6.7 or with the drilled shaft recommendations below.

6.12.6 Drilled shaft (pier) foundations for retaining walls should have a minimum diameter of 18 inches and minimum embedment depth of 15 feet. The upper 1 foot of piers below the ground surface should be neglected when calculating for vertical capacities. Allowable skin friction to resist axial compression loads may be used at 400 pounds per square foot. For uplift capacity, allowable skin friction may be assumed to be ⅔ of that in compression.

6.12.7 Piers should have a minimum center-to-center spacing of at least three pier diameters and any end bearing contribution should be ignored. Allowable passive pressure used to resist lateral movement may be assumed to be equal to a fluid weighing 300 pounds per cubic foot (pcf). The passive pressure may be applied over two diameters for drilled piers. Where not protected by pavement, passive resistance should be ignored for the upper 1 foot of site soils. Passive soil resistance should also be ignored where less than 10 feet of cover (measured horizontally) exists between the drilled shaft and a slope face.

6.12.8 Pier excavations should be clear of loose soil, debris, and standing water prior to placing reinforcing steel. However, groundwater may be encountered within the foundation construction depths and wet construction methods may be required if localized dewatering is not successful. Temporary casings may be needed if loose or flowing sands are encountered.

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6.12.9 We recommend that all retaining wall designs be reviewed by Geocon to confirm the incorporation of the recommendations provided herein. In particular, potential surcharges from adjacent structures and other improvements should be reviewed by Geocon.

6.13 Surface Drainage

6.13.1 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 to important engineering properties. Proper drainage should be maintained at all times.

6.13.2 All site drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundations or retaining walls. Drainage should not be allowed to flow uncontrolled over any descending slope. The proposed structures should be provided with roof gutters. Discharge from downspouts, roof drains and scuppers not permitted onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed or properly drained to prevent moisture intrusion into the materials providing foundation support. Landscape irrigation within five feet of the building perimeter footings should be kept to a minimum to just support vegetative life.

6.13.3 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. Final soil grade should slope a minimum of 2% away from structures.

6.13.4 We recommend implemented measures to reduce infiltrating surface water near buildings and slabs-on-grade. Such measures may include:

- Selecting drought-tolerant plants that require little or no irrigation, especially within three feet of buildings, slabs-on-grade, or pavements.
- Using drip irrigation or low-output sprinklers.
- Using automatic timers for irrigation systems.
- Appropriately spaced area drains.
- Hard-piping roof downspouts to appropriate collection facilities.

7. FURTHER GEOTECHNICAL SERVICES

7.1 Plan and Specification Review

7.1.1 We should review project plans and specifications prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

7.2 Testing and Observation Services

7.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase and provide compaction testing and observation services and foundation observations throughout the project. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for others interpretation of our recommendations, and therefore the future performance of the project.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 Consultants, 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 geotechnical scope of services provided by Geocon Consultants, Inc.

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.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or 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.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices used in the site area at this time. No warranty is provided, express or implied.

0 1

Scale in Miles

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Coastside Fire Station No. 41

Obispo Road and Avenue Alhambra
Half Moon Bay, California

VICINITY MAP

E8940-04-01	August 2016	Figure 1

Ref: JKA Architecture, 7/16/15 & Goggle Earth, 4/5/16

LEGEND:

B5 ⊗ Approximate Boring Location

0 60
Scale in Feet

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SITE PLAN

E8940-04-01	August 2016	Figure 2

APPENDIX A

APPENDIX A
FIELD EXPLORATION

Fieldwork for our investigation included a site visit, subsurface exploration, and soil sampling. The locations of our exploratory borings are shown on the Site Plan, Figure 2. Soil boring logs are presented as figures following the text in this appendix. The borings were located in the field using a measuring tape and existing reference points. Therefore, the exploration locations shown on Figure 2 are approximate.

Our subsurface exploration was performed on July 13, 2016 and included the drilling and sampling of existing soils with a Mobile B-56 drill rig equipped with 8-inch hollow-stem augers. Sampling in the borings was accomplished using a 140-pound wireline hammer with a 30-inch drop. Samples were obtained with a 3-inch outside-diameter (OD), split spoon (California Modified) sampler, and a 2-inch OD, Standard Penetration Test (SPT) sampler. The number of blows required to drive the sampler the last 12 inches (or fraction thereof) of the 18-inch sampling interval were recorded on the boring logs. The blow counts shown on the boring logs should not be interpreted as standard SPT “N” values; corrections have not been applied. Samples were collected at appropriate intervals, classified by our field geologist, retained in moisture-tight containers and transported to the laboratory for testing and further classification. The applicable type of each sampling interval is noted on the exploratory boring logs. Upon completion, our borings were backfilled in accordance with San Mateo County permit requirements.

Subsurface conditions encountered in the exploratory boring were visually examined, classified and logged in general accordance with the American Society for Testing and Materials (ASTM) Practice for Description and Identification of Soils (Visual-Manual Procedure D2488). This system uses the Unified Soil Classification System (USCS) for soil designations. The log depicts soil and geologic conditions encountered and depths at which samples were obtained. The log also includes 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, drill rig penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field log was revised based on subsequent laboratory testing.

UNIFIED SOIL CLASSIFICATION

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO.4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS WITH OR WITHOUT SAND, LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, SILTY GRAVELS WITH SAND
			GC	CLAYEY GRAVELS, CLAYEY GRAVELS WITH SAND
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO.4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS WITH OR WITHOUT GRAVEL, LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS WITH OR WITHOUT GRAVEL
			SC	CLAYEY SANDS WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTS WITH SANDS AND GRAVELS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, CLAYS WITH SANDS AND GRAVELS, LEAN CLAYS
			OL	ORGANIC SILTS OR CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OR CLAYS OF MEDIUM TO HIGH PLASTICITY
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample
- Bulk Sample
- SPT Sample
- Modified California Sample
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

SAND AND GRAVEL			SILT AND CLAY			
RELATIVE DENSITY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	CONSISTENCY	BLOWS PER FOOT (SPT)*	BLOWS PER FOOT (MOD-CAL)*	COMPRESSIVE STRENGTH (tsf)
VERY LOOSE	0 - 4	0 - 6	VERY SOFT	0 - 2	0 - 3	0 - 0.25
LOOSE	5 - 10	7 - 16	SOFT	3 - 4	4 - 6	0.25 - 0.50
MEDIUM DENSE	11 - 30	17 - 48	MEDIUM STIFF	5 - 8	7 - 13	0.50 - 1.0
DENSE	31 - 50	49 - 79	STIFF	9 - 15	14 - 24	1.0 - 2.0
VERY DENSE	OVER 50	OVER 79	VERY STIFF	16 - 30	25 - 48	2.0 - 4.0
			HARD	OVER 30	OVER 48	OVER 4.0

*NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE LAST 12 INCHES OF AN 18-INCH DRIVE

MOISTURE DESCRIPTIONS

FIELD TEST	APPROX. DEGREE OF SATURATION, S (%)	DESCRIPTION
NO INDICATION OF MOISTURE; DRY TO THE TOUCH	$S<25$	DRY
SLIGHT INDICATION OF MOISTURE	$25\leq S<50$	DAMP
INDICATION OF MOISTURE; NO VISIBLE WATER	$50\leq S<75$	MOIST
MINOR VISIBLE FREE WATER	$75\leq S<100$	WET
VISIBLE FREE WATER	100	SATURATED

QUANTITY DESCRIPTIONS

APPROX. ESTIMATED PERCENT	DESCRIPTION
<5%	TRACE
5 - 10%	FEW
11 - 25%	LITTLE
26 - 50%	SOME
>50%	MOSTLY

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

CRITERIA	DESCRIPTION
PASS THROUGH A 3-INCH SIEVE AND BE RETAINED ON A NO. 4 SIEVE (#4 TO 3")	GRAVEL
PASS A 12-INCH SQUARE OPENING AND BE RETAINED ON A 3-INCH SIEVE (3"-12")	COBBLE
WILL NOT PASS A 12-INCH SQUARE OPENING (>12")	BOULDER

BEDDING SPACING DESCRIPTIONS

THICKNESS/SPACING	DESCRIPTOR
GREATER THAN 10 FEET	MASSIVE
3 TO 10 FEET	VERY THICKLY BEDDED
1 TO 3 FEET	THICKLY BEDDED
3 5/8-INCH TO 1 FOOT	MODERATELY BEDDED
1 1/4-INCH TO 3 5/8-INCH	THINLY BEDDED
3/8-INCH TO 1 1/4-INCH	VERY THINLY BEDDED
LESS THAN 3/8-INCH	LAMINATED

STRUCTURE DESCRIPTIONS

CRITERIA	DESCRIPTION
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS AT LEAST 1/4-INCH THICK	STRATIFIED
ALTERNATING LAYERS OF VARYING MATERIAL OR COLOR WITH LAYERS LESS THAN 1/4-INCH THICK	LAMINATED
BREAKS ALONG DEFINITE PLANES OF FRACTURE WITH LITTLE RESISTANCE TO FRACTURING	FISSURED
FRACTURE PLANES APPEAR POLISHED OR GLOSSY, SOMETIMES STRIATED	SLICKENSIDED
COHESIVE SOIL THAT CAN BE BROKEN DOWN INTO SMALLER ANGULAR LUMPS WHICH RESIST FURTHER BREAKDOWN	BLOCKY
INCLUSION OF SMALL POCKETS OF DIFFERENT SOIL, SUCH AS SMALL LENSES OF SAND SCATTERED THROUGH A MASS OF CLAY	LENSED
SAME COLOR AND MATERIAL THROUGHOUT	HOMOGENOUS

CEMENTATION/INDURATION DESCRIPTIONS

FIELD TEST	DESCRIPTION
CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE	WEAKLY CEMENTED/INDURATED
CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE	MODERATELY CEMENTED/INDURATED
WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE	STRONGLY CEMENTED/INDURATED

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/8-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

DEGREE OF DECOMPOSITION	FIELD RECOGNITION	ENGINEERING PROPERTIES
SOIL	DISCOLORED, CHANGED TO SOIL, FABRIC DESTROYED	EASY TO DIG
COMPLETELY WEATHERED	DISCOLORED, CHANGED TO SOIL, FABRIC MAINLY PRESERVED	EXCAVATED BY HAND OR RIPPING (Saprolite)
HIGHLY WEATHERED	DISCOLORED, HIGHLY FRACTURED, FABRIC ALTERED AROUND FRACTURES	EXCAVATED BY HAND OR RIPPING, WITH SLIGHT DIFFICULTY
MODERATELY WEATHERED	DISCOLORED, FRACTURES, INTACT ROCK-NOTICEABLY WEAKER THAN FRESH ROCK	EXCAVATED WITH DIFFICULTY WITHOUT EXPLOSIVES
SLIGHTLY WEATHERED	MAY BE DISCOLORED, SOME FRACTURES, INTACT ROCK-NOT NOTICEABLY WEAKER THAN FRESH ROCK	REQUIRES EXPLOSIVES FOR EXCAVATION, WITH PERMEABLE JOINTS AND FRACTURES
FRESH	NO DISCOLORATION, OR LOSS OF STRENGTH	REQUIRES EXPLOSIVES

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

FIELD TEST	DESCRIPTION
NO OBSERVED FRACTURES	UNFRACTURED/UNJOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1 TO 3 FOOT INTERVALS	SLIGHTLY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 4-INCH TO 1 FOOT INTERVALS	MODERATELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT 1-INCH TO 4-INCH INTERVALS WITH SCATTERED FRAGMENTED INTERVALS	INTENSELY FRACTURED/JOINTED
MAJORITY OF JOINTS/FRACTURES SPACED AT LESS THAN 1-INCH INTERVALS; MOSTLY RECOVERED AS CHIPS AND FRAGMENTS	VERY INTENSELY FRACTURED/JOINTED

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KEY TO LOGS

E8940-04-01 | August 2016 | Figure A1

BORING B1

ELEV. (MSL.) ____________ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CH	TERRACE DEPOSITS Medium stiff, dry to damp, black, CLAY with sand			
1								
2					-stiff, moist, orange-brown mottled gray -pp=1¾-2¼			
	B1-2.5-3					16		
3	B1-3						94.2	28.8
					-more sand -pp=3½-4½			
4	B1-4-4.5					20		
	B1-4.5						106.6	20.3
5								
6								
7								
8								
				SC	Medium dense, moist, orange-gray speckled varicolored, Clayey (f-c) SAND			
9	B1-9-9.5					21		
	B1-9.5							
10					END OF BORING AT APPROXIMATELY 10 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A2, Log of Boring B1, page 1 of 1

GEOCON BORING LOG E8940-04-01 BORING LOGS.GPJ 08/14/16

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SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... 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. **E8940-04-01** PROJECT NAME: **Coastside Fire Station No. 41**

BORING B2

ELEV. (MSL.) ____ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	TERRACE DEPOSITS Stiff, damp, brown, (f-m) Sandy CLAY			
1								
2					-very stiff, orange-gray and black, sand (f-c) -pp>4½			
	B2-2.5-3					36		
3	B2-3						122.5	10.0
					-stiff, black			
4	B2-4-4.5			SC	Medium dense, moist, black, Clayey (f-c) SAND with few (f) gravels -gravels generally sub-rounded to sub-angular	20		
	B2-4.5							
5								
6								
7								
8								
					-damp to moist, grayish-orange speckled varicolored, more clay, less gravels -pp>4½			
9	B2-9-9.5					41		
	B2-9.5						113.9	16.5
10								
11								
12								

Figure A3, Log of Boring B2, page 1 of 4

GEOCON BORING LOG E8940-04-01 BORING LOGS.GPJ 08/14/16

GEOCON

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... 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.

BORING B2

ELEV. (MSL.)		DATE COMPLETED	7/13/2016
ENG./GEO.	JBM	DRILLER	EGI
EQUIPMENT	Mobile B56 w/ 8-inch HSA	HAMMER TYPE	Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
13								
14	B2-14-14.5				-same	35		
	B2-14.5						111.5	18.3
15								
16								
17								
18								
19	B2-19-19.5				-dense	57		
	B2-19.5						119.7	13.8
20								
21								
22								
23			▼					
				CL	Hard, moist, orange-brown, CLAY with (f-c) sand			
24	B2-24-25				-pp>4½	37		

Figure A3, Log of Boring B2, page 2 of 4

GEOCON

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

BORING B2

ELEV. (MSL.) ____ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25								
26								
27								
28								
29	B2-29-30				-same	54		
30								
31								
32								
33				SC	Medium dense, moist, orange-gray, (f-m) SAND with little clay			
34	B2-34-34.5 B2-34.5					47		
35								
36								
37								

Figure A3, Log of Boring B2, page 3 of 4

GEOCON

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

BORING B2

ELEV. (MSL.)		DATE COMPLETED	**7/13/2016**
ENG./GEO.	**JBM**	DRILLER	**EGI**
EQUIPMENT	**Mobile B56 w/ 8-inch HSA**	HAMMER TYPE	**Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
38								
39	B2-39-40			SP-SC	Very dense, moist to wet, brown, (f-m) SAND with few clays	78		
40					END OF BORING AT APPROXIMATELY 40 FEET GROUNDWATER INTIALLY ENCOUNTERED AT APPROXIMATELY 23 FEET BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT			

Figure A3, Log of Boring B2, page 4 of 4

GEOCON

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.

BORING B3

ELEV. (MSL.) ____ DATE COMPLETED 7/13/2016
ENG./GEO. JBM DRILLER EGI
EQUIPMENT Mobile B56 w/ 8-inch HSA HAMMER TYPE Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	TERRACE DEPOSITS Stiff, damp, dark brown, CLAY with (f-c) sand			
1								
2								
	B3-2.5-3				-damp to moist, orange-brown -pp=3¼-3¾	17		
3	B3-3						113.1	17.5
4	B3-4-4.5				-speckled white, more sand -pp=3½-4	23		
	B3-4.5						109.4	18.1
5								
6								
7								
8								
					-very stiff			
9	B3-9-10					22		
10								
11								
12								

Figure A4, Log of Boring B3, page 1 of 3

GEOCON

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

BORING B3

ELEV. (MSL.) ____________ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
13			▼					
14	B3-14-14.5			SC	Medium dense, moist, gray mottled orange-brown streaked rust, Clayey (f-c) SAND	41		
	B3-14.5						119.3	15.9
15								
16								
17								
18								
19	B3-19-20			CL	Very stiff, moist, orange-brown, CLAY with (f-c) sand	28		
20								
21								
22								
23								
24				SP-SC	Medium dense, moist to wet, brown, (f-m) SAND with few clays -no recovery	30		

Figure A4, Log of Boring B3, page 2 of 3

GEOCON

SAMPLE SYMBOLS

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

BORING B3

ELEV. (MSL.) ______ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
25					-very dense			
	B3-25.5-26.5					56		
26								
27								
28								
29	B3-29-30					74		
30					END OF BORING AT APPROXIMATELY 30 FEET GROUNDWATER INTIALLY ENCOUNTERED AT APPROXIMATELY 13 FEET BACKFILLED WITH COMPACTED CUTTINGS AND CEMENT			

Figure A4, Log of Boring B3, page 3 of 3

GEOCON

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

BORING B4

ELEV. (MSL.) ____________ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	TERRACE DEPOSITS Stiff, damp to moist, orange-brown speckled varicolored, (f-c) Sandy CLAY			
1								
2					-pp>4½			
	B4-2.5-3					16		
3	B4-3			ML	Stiff, damp to moist, black, SILT with clay and (f) sand -pp=3-4½			
4	B4-4-4.5					16		
	B4-4.5			CL	Stiff, moist, orange-brown speckled varicolored, CLAY with little (f-c) sand -pp=2½-3		105.9	20.5
5	B4-5-10							
6								
7								
8				SC	Dense, moist, gray mottled orange streaked rust, Clayey (f-c) SAND			
9	B4-9-9.5					64		
	B4-9.5						116.1	15.0
10								
11								
12								

Figure A5, Log of Boring B4, page 1 of 2

GEOCON

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

BORING B4

ELEV. (MSL.) ______ DATE COMPLETED **7/13/2016**

ENG./GEO. **JBM** DRILLER **EGI**

EQUIPMENT **Mobile B56 w/ 8-inch HSA** HAMMER TYPE **Downhole-Wireline**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
13								
					-medium dense, orange-gray			
14	B4-14-14.5					42		
	B4-14.5						111.7	18.5
15								
16								
17								
18								
					-same			
19	B4-19-19.5					25		
	B4-19.5							
20					END OF BORING AT APPROXIMATELY 20 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A5, Log of Boring B4, page 2 of 2

GEOCON

SAMPLE SYMBOLS

... SAMPLING UNSUCCESSFUL

... DISTURBED OR BAG SAMPLE

... STANDARD PENETRATION TEST

... CHUNK SAMPLE

... DRIVE SAMPLE (UNDISTURBED)

... WATER TABLE OR SEEPAGE

BORING B5

ELEV. (MSL.) ____ DATE COMPLETED 7/13/2016
ENG./GEO. JBM DRILLER EGI
EQUIPMENT Mobile B56 w/ 8-inch HSA HAMMER TYPE Downhole-Wireline

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B5-0-5	CL	TERRACE DEPOSITS Stiff, damp, dark brown to black speckled varicolored, (f-c) Sandy CLAY			
2			-very stiff -pp>4½			
	B5-2.5			41		
3	B5-3		-orange-brown mottled gray		114.2	16.0
4	B5-4-4.5		-hard, damp to moist -pp>4½	50		
	B5-4.5				108.8	20.4
9	B5-9-9.5		-more sand	49		
	B5-9.5	SC	Dense, damp to moist, orange-brown, Clayey (f-c) SAND			
10			END OF BORING AT APPROXIMATELY 10 FEET NO FREE WATER ENCOUNTERED BACKFILLED WITH COMPACTED CUTTINGS			

Figure A6, Log of Boring B5, page 1 of 1

GEOCON BORING LOG E8940-04-01 BORING LOGS.GPJ 08/14/16

GEOCON

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

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 in-situ dry density and moisture content, laboratory maximum dry density and optimum moisture content, grain size distribution, plasticity, expansion, unconfined compressive strength, shear strength and R-value. The results of our testing are summarized in tabular format below and the following figures. In-situ dry density and/or moisture content test results are included on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY ATTERBERG LIMITS TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index
B1-2.5-3	71	28	43
B3-2.5	22	15	8

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density* (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B5-0-5	11.0	22.6	106.2	26

*Before saturation

TABLE B-III
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B4-5-10	Orange-brown CLAY with sand	117.9	11.8

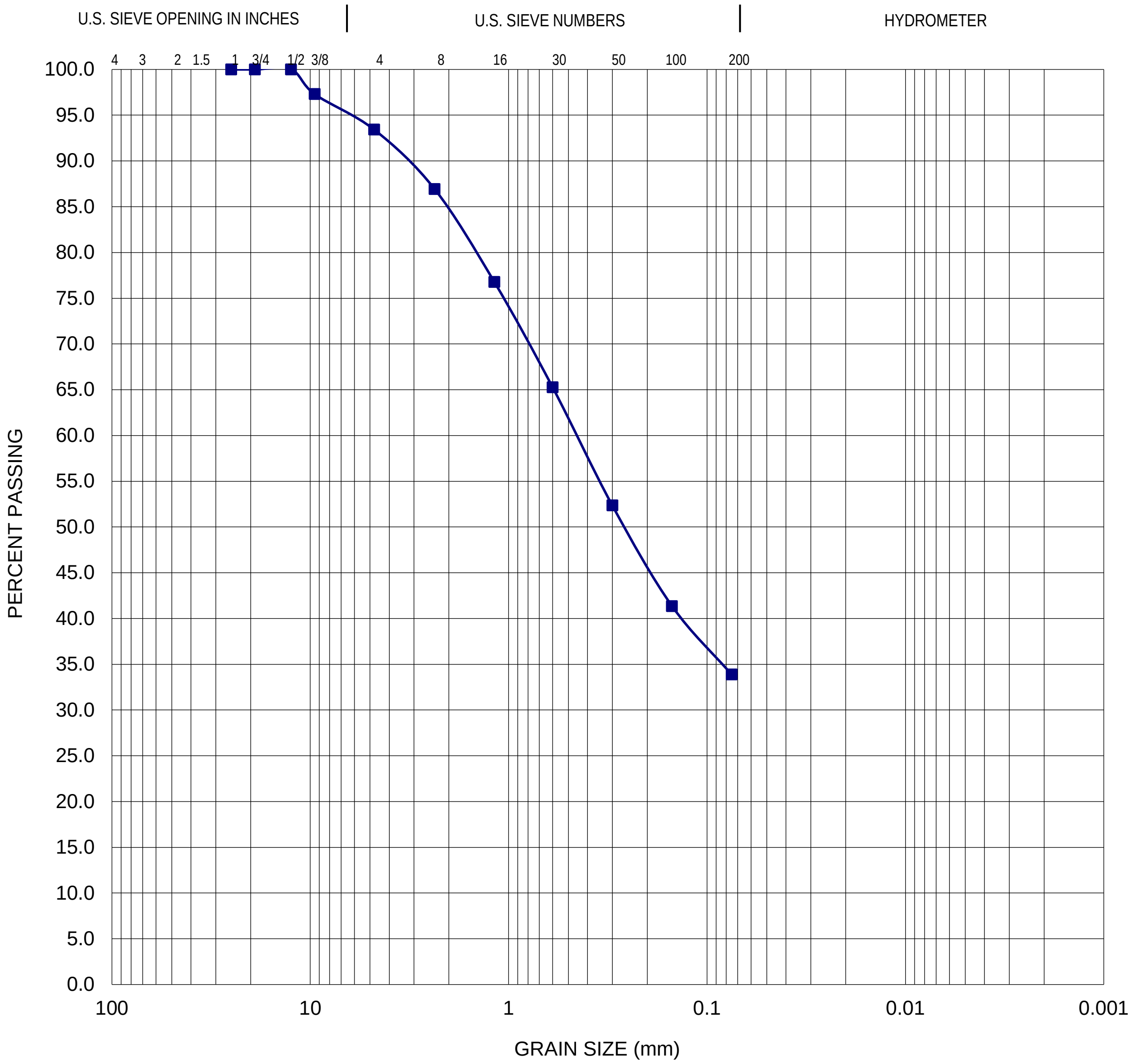
APPENDIX B
LABORATORY TESTING (cont.)

TABLE B-IV
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Boring No.	Sample Depth (feet)	Initial Average Dry Density (pcf)	Initial Average Moisture Content (%)	Cohesion (psf)	Angle of Shear Resistance (degrees)
B3	4.5	109.4	18.1	620	26
B4	5 -10	107.8	13.0	270	26
B4	9.5	116.1	15.0	1180	24

TABLE B-V
SUMMARY OF LABORATORY R-VALUE TEST RESULTS
ASTM D 2844

Sample No.	Soil Type (USCS Classification)	R-Value
B5-0-5	Sandy CLAY (CL)	5



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B2

Depth To Sample: 4-4.5'

Sieve Date: 08/01/2016

Tested and Computed by: VV/CO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	97.3	93.4	86.9	76.8	65.3	52.3	41.3	33.9

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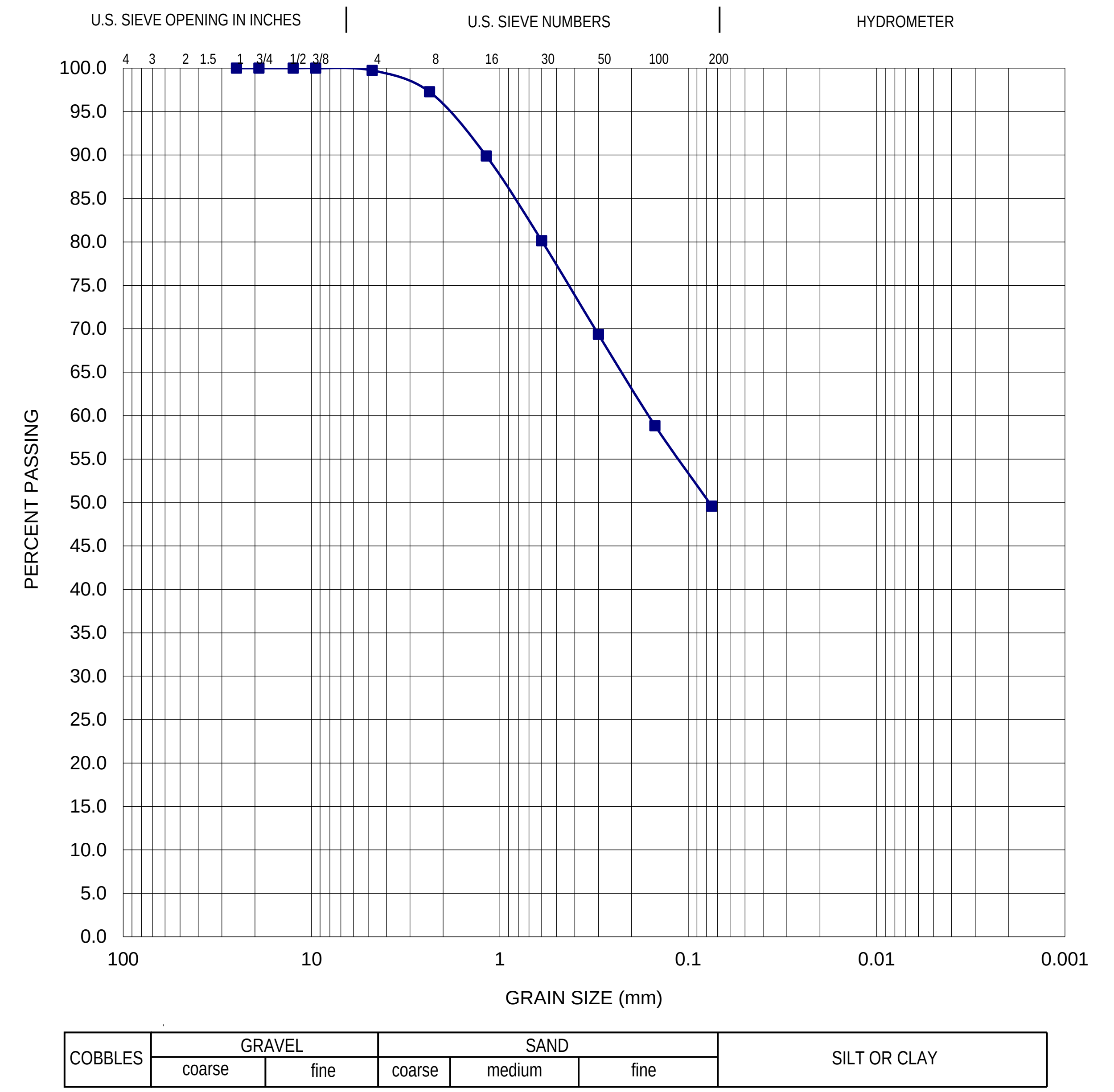
Particle Size Analysis - ASTM D422

Project: Coastside Fire Station No. 41

Location: Half Moon Bay, California

Project No.: E8940-04-01

Figure B1



Boring: B2

Depth To Sample: 14-14.5'

Sieve Date: 08/01/2016

Tested and Computed by: VV/CO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	100.0	99.7	97.3	89.9	80.1	69.3	58.8	49.6

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Particle Size Analysis - ASTM D422

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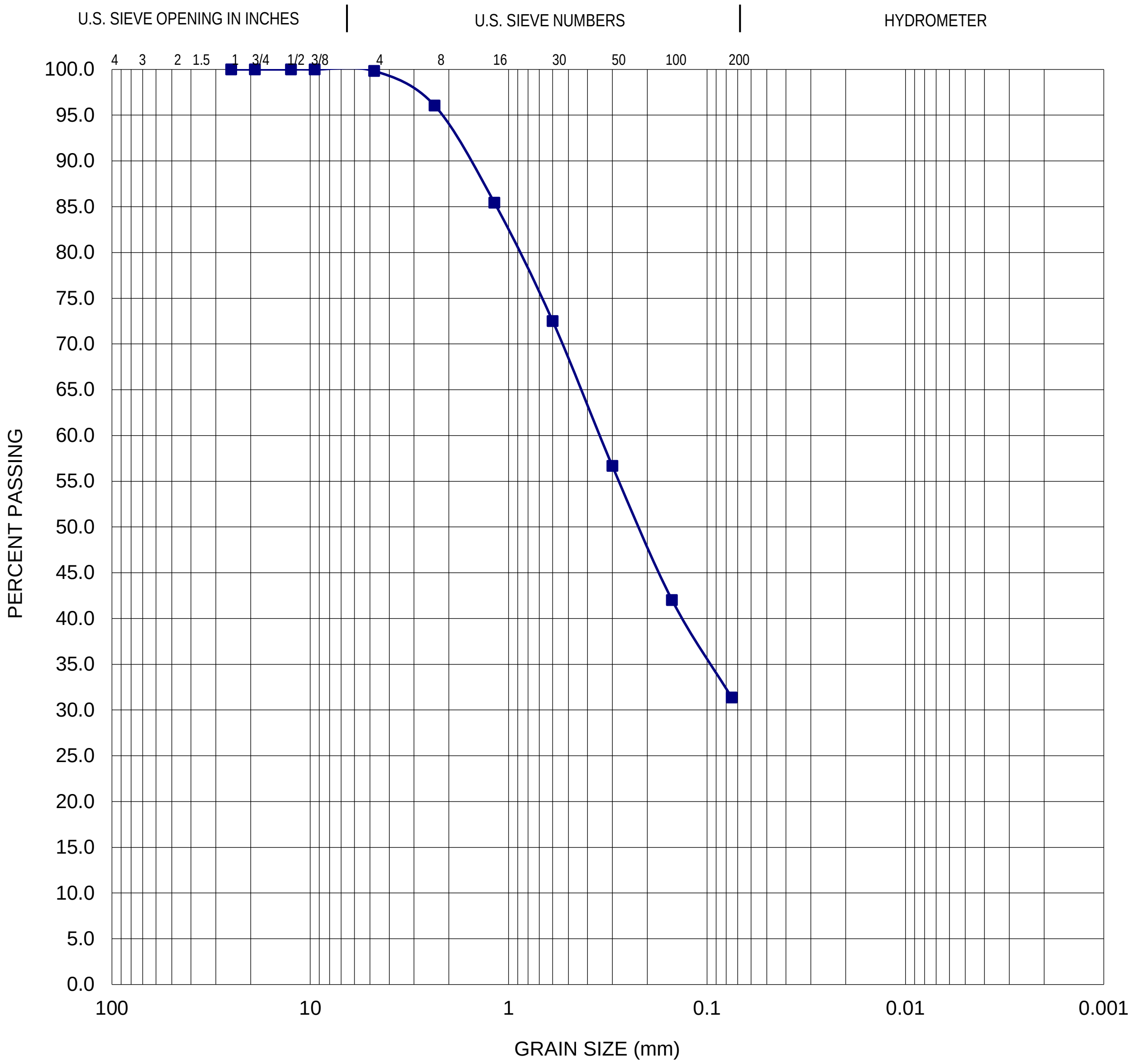
Location: Half Moon Bay, California

Project No.: E8940-04-01

Figure B2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B3
Depth To Sample: 14-14.5'

Sieve Date: 08/01/2016
Tested and Computed by: VV/CO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	100.0	99.8	96.0	85.4	72.5	56.7	42.0	31.3

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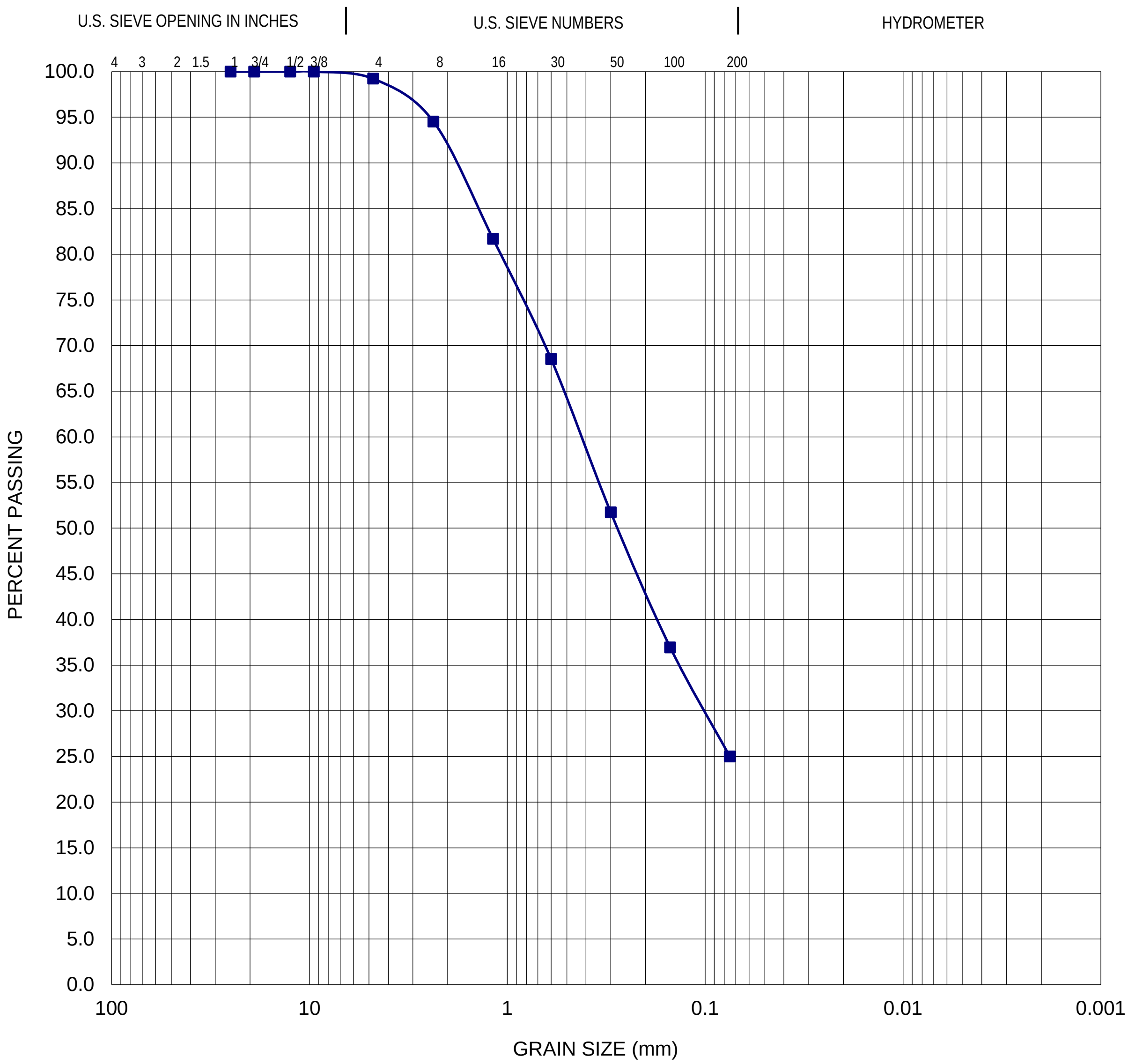
Particle Size Analysis - ASTM D422

Project: Coastside Fire Station No. 41
Location: Half Moon Bay, California
Project No.: E8940-04-01

Figure B3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B4

Depth To Sample: 9-9.5'

Sieve Date: 08/01/2016

Tested and Computed by: VV/CO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	100.0	99.2	94.5	81.7	68.5	51.7	36.9	25.0

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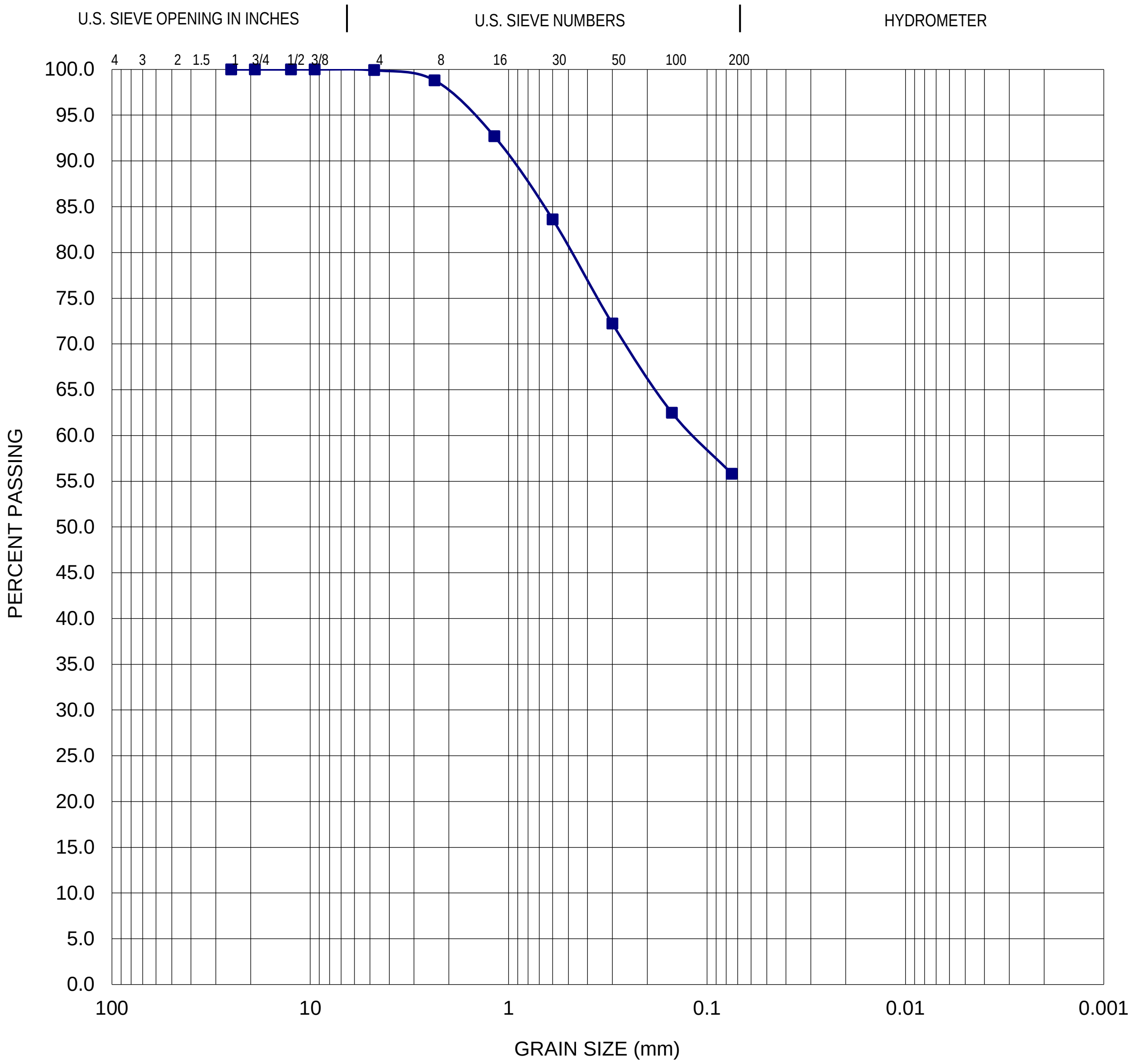
Particle Size Analysis - ASTM D422

Project: Coastside Fire Station No. 41

Location: Half Moon Bay, California

Project No.: E8940-04-01

Figure B4



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring: B5

Depth To Sample: 2.5'

Sieve Date: 08/01/2016

Tested and Computed by: VV/CO

Test Data

Sieve Number	1 1/2"	1"	3/4"	1/2"	3/8"	#4	#8	#16	#30	#50	#100	#200
% Passing	100.0	100.0	100.0	100.0	100.0	99.9	98.8	92.7	83.6	72.2	62.5	55.8

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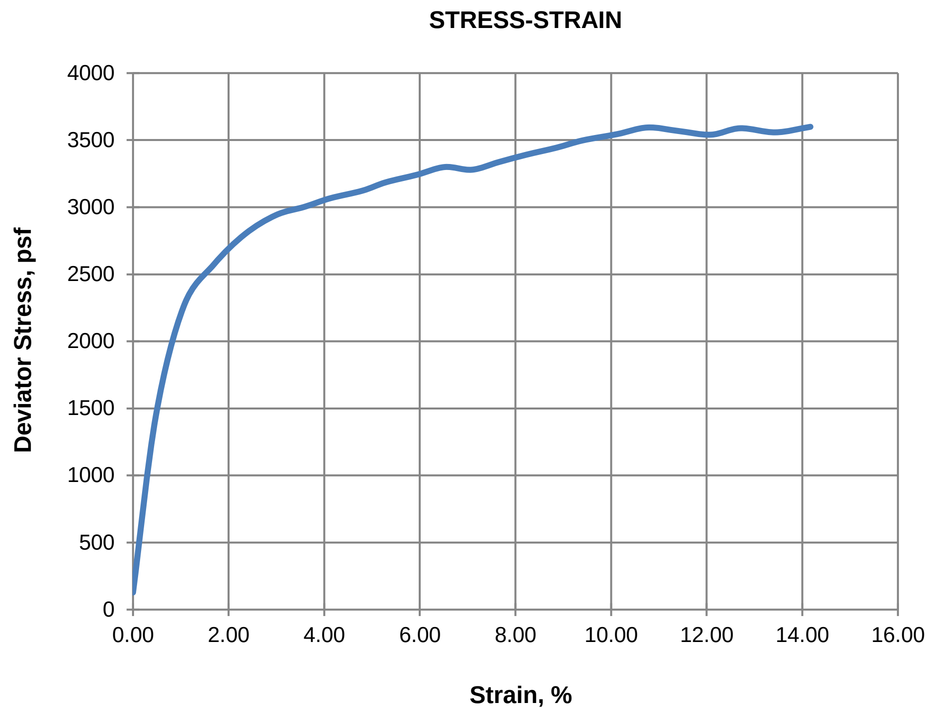
Particle Size Analysis - ASTM D422

Project: Coastside Fire Station No. 41

Location: Half Moon Bay, California

Project No.: E8940-04-01

Figure B5



Failure Photo

Sample Description	
Boring Number	B4
Sample Depth (feet)	4.5'
Material Description	Dark Orange-brown CLAY with (f-c) Sand
Initial Conditions at Start of Test	
Height (inch) average of 3	5.86
Diameter (inch) average of 3	2.39
Moisture Content (%)	20.5
Dry Density (pcf)	105.9
Estimated Specific Gravity	2.7
Saturation (%)	93.8
Shear Test Conditions	
Strain Rate (%/min)	1.1810
Major Principal Stress at Failure (psf)	3600
Strain at Failure (%)	15.9
Test Results	
Unconfined Compressive Strength (tons/ft^2)	1.8
Unconfined Compressive Strength (lbs/ft^2)	3600
Shear Strength (tons/ft^2)	0.9
Shear Strength (lbs/ft^2)	1800

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Unconfined Compressive Strength (ASTM D2166)

Project: Coastside Fire Station #41
Location: Half Moon Bay, California
Project No.: E8940-04-01

Figure B6

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Unpublished reports, aerial photographs and maps on file with Geocon.