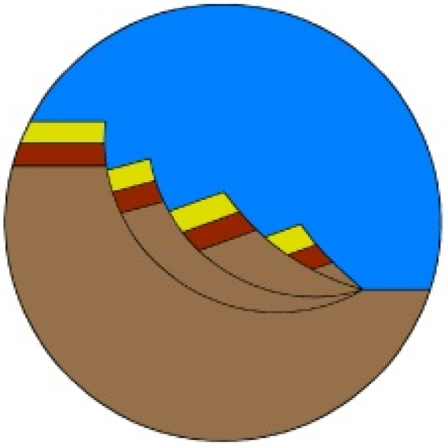
GEOTECHNICAL INVESTIGATION

515 Soquel Avenue
Santa Cruz, Santa Cruz County, California

Submitted to:

Shay Talbot
849 Almar Avenue, #C300
Santa Cruz, California 95060



Prepared by:

CMAG ENGINEERING, INC.

Project No. 11-106-SC
April 19, 2021

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April 19, 2021
Project No. 11-106-SC

Shay Talbot
849 Almar Avenue, #C300
Santa Cruz, California 95060

SUBJECT: **GEOTECHNICAL INVESTIGATION**
Proposed Multi-Unit Residential Buildings
513, 515, and 519 Soquel Avenue
Santa Cruz, Santa Cruz County, California
APN 010-012-29, 010-012-21, 010-012-30

Dear Mr. Talbot:

In accordance with your authorization, we have completed a geotechnical investigation for the subject project. This report summarizes the findings, conclusions, and recommendations from our field exploration, laboratory testing, and engineering analysis. It is a pleasure being associated with you on this project. If you have any questions, or if we may be of further assistance, please do not hesitate to contact our office.

Sincerely,

CMAG ENGINEERING, INC.

Reviewed by:

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Distribution: Addressee (4 Hard Copies; Electronic Copy)

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed multi-unit residential development at 515 Soquel Avenue, in Santa Cruz, Santa Cruz County, California.

The purpose of our investigation was to provide information regarding the surface and subsurface soil and bedrock conditions, and based on our findings, provide geotechnical recommendations for the design and construction of the proposed development. Conclusions and recommendations related to geotechnical hazards, site grading, drainage, foundations, retaining structures, and pavements are presented herein.

1.1 Terms of Reference

CMAG Engineering, Inc.'s (CMAG) scope of work for this phase of the project included site reconnaissance, subsurface exploration, soil and bedrock sampling, laboratory testing, engineering analyses, and preparation of this report.

The work was undertaken in accordance with CMAG's *Proposal for Geotechnical Services* dated March 23, 2011 and November 6, 2020.

The recommendations contained in this report are subject to the limitations presented in Section 8.0 of this report.

1.2 Site Location

The project site is located on the north side of Soquel Avenue, approximately 500 feet northeast of its intersection with Ocean Street in the City of Santa Cruz, Santa Cruz County, California. The site location is shown on the Site Location Map, Appendix A, Figure A-1.

1.3 Surface Conditions

The site is currently located on three separate parcels located on the north side of the intersection of May Avenue and Soquel Avenue. The site is situated on a dissected slope that descends to the southwest, which has been previously graded to achieve the existing grades. The parcels are currently developed with two buildings, asphalt paved parking and drive areas, and landscaping that includes mature trees. In addition to sloped terrain, site retaining walls support grade changes. Retaining walls have also been incorporated into one of the existing buildings.

2.0 PROJECT DESCRIPTION

It is our understanding that the project consists of the construction of three multi-unit residential buildings (Buildings A, B, and C), drive areas, parking, landscape areas, and associated improvements. The existing buildings and drive areas are to be demolished in association with the proposed development.

Proposed Buildings A and B consist of 4 story structures founded on a continuous podium supported by an underground parking area. Buildings A and B are located adjacent to Soquel Avenue, on APNs 010-012-29 and 30, covering the majority of the parcels. A small section of the building extends onto the eastern end of APN 010-012-21. At grade, the buildings are separated with a common area. The parking area is approximately 12 feet below the existing grade of the northeast portion of the site (APN 010-012-30). The overall square footage of Buildings A and B on the podium is 8,966 square feet and the underground parking / storage area is 13,021 square feet. The units consist of apartments, townhomes, studios, a commercial space on the first floor of Building B, and a community lounge on the first floor of Building A.

Building C consists of a one story 1,376 square foot residential structure with a covered drive area, at grade, beneath the building. Building C is located on APN 010-012-21, with the western property line adjacent to May Avenue.

The proposed grades do not differ significantly from the existing grades with the exception of the proposed underground parking area beneath Buildings A and B.

3.0 FIELD EXPLORATION AND LABORATORY TESTING PROGRAMS

Our field exploration program included drilling, logging, and interval sampling of 7 borings on March 30, 2011 and 1 boring on March 18, 2021. The borings were advanced to depths ranging from 6.5± feet to 34± feet below the existing grades. Details of the field exploration program, including the Boring Logs, Figures A-4 through A-12, are presented in Appendix A.

Representative samples obtained during the field investigation were taken to the laboratory for testing to determine physical and engineering properties. Details of the laboratory testing program are presented in Appendix B. Test results are presented on the Boring Logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS AND EARTH MATERIALS

4.1 General

The geologic map of Santa Cruz County (Brabb, 1997) depicts the subject property as underlain by the Purisima Formation bedrock (Tp; Pliocene and Upper Miocene). Alluvial deposits (Qal; Holocene) have been mapped to the west of the site and Lowest Emergent Coastal Terrace Deposits (Qcl; Pleistocene) have been mapped to the east of the site. See Figure A-2 in Appendix A for a geologic map of the area.

Eight borings were advanced on the project site. The subsurface conditions encountered were not consistent with the geologic mapping. We encountered Purisima Formation bedrock on the northern side of the site, however, fill and alluviual deposits were encountered on the southern side of the site, overlying the bedrock.

Complete subsurface profiles are presented on the Boring Logs, Appendix A, Figures A-5 through A-12. The boring locations are shown on the Boring Location Plan, Figure A-3. Two representative cross sections, Cross Section A-A’ and B-B’, have been constructed based on the results of our field exploration (Figures A-13 and A-14, in Appendix A).

4.2 Artificial Fill - af

Fill soils were encountered in Borings B-2, B-3, and B-4 varying in depth from 5± to 11.5± feet below the existing grades. The fill is located on the northeast side of the existing commercial building. The fill generally consisted of loose to very loose silty sands.

4.3 Alluvial Deposits - Qal

Alluvial deposits were encountered in all the borings with the exception of Borings B-1 and B-4, overlying the Purisima Formation Bedrock. As depicted on Cross Sections A-A’ and B-B’, the alluvial deposits consist of a wedge, increasing in thickness towards the west. Due to the large difference in the depth to bedrock over a relatively short distance, the contact between the alluvial deposits and bedrock has been labeled as approximate and uncertain on the cross sections. In addition, it is likely that the contact is not planar as depicted, yet irregular and stepped.

The alluvial deposits generally consist of interbedded sands, clays, and silts. In general, the cohesionless soils were loose to medium dense and the cohesive soils were stiff. Based on the results of our laboratory testing, the high plasticity clays have a very high expansion potential.

4.4 Purisima Formation Bedrock - Tp

Purisima Formation bedrock was encountered at depths varying from the surface to 26± feet below the existing grades. The bedrock generally consisted of medium dense to very dense, weakly to strongly cemented siltstone and fine grained sandstone. As shown on the cross sections, the subsurface bedrock profile steeply dips to the west.

4.5 Groundwater

Groundwater was encountered in Borings B-2 and B-7 at approximately 12 feet below existing grades during our field exploration on March 30, 2011. Groundwater was encountered at approximately 12 feet below grade in Boring B-8, advanced on March 18, 2021. The groundwater is perched on the underlying Purisima Formation bedrock.

It should be noted that groundwater conditions, perched or regional, may vary with location and may fluctuate with variations in rainfall, runoff, irrigation, and other changes to the conditions existing at the time our field investigation was performed.

5.0 GEOTECHNICAL HAZARDS

5.1 General

In our opinion, the geotechnical hazards that could potentially affect the proposed project are:

- Seismic shaking
- Collateral seismic hazards

5.2 Seismic Shaking

The hazard due to seismic shaking in California is high in many areas, indicative of the number of large earthquakes that have occurred historically. Intense seismic shaking may occur at the site during the design lifetime of the proposed structures from an earthquake along one of the local fault systems. Generally, the intensity of shaking will increase the closer the site is to the epicenter of an earthquake, however, seismic shaking is a complex phenomenon and may be modified by local topography and soil conditions. The transmission of earthquake vibrations from the ground into the structures may cause structural damage.

The City of Santa Cruz has adopted the seismic provisions set forth in the 2019 California Building Code (2019 CBC) to address seismic shaking. The seismic provisions in the 2019 CBC are minimum load requirements for the seismic design for the proposed structures. The provisions set forth in the 2019 CBC will not prevent structural and nonstructural damage from direct fault ground surface rupture, coseismic ground cracking, liquefaction and lateral spreading, seismically induced differential compaction, or seismically induced landsliding.

Table 1 has been constructed based on the 2019 CBC requirements for the seismic design of the proposed structures. The Site Class has been determined based on our field investigation and laboratory testing.

Table 1. Seismic Design Parameters - 2019 CBC

S_S	S_1	Site Class	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}	PGA_M
1.658g	0.635g	D	1.0	null*	1.658g	null*	1.105g	null*	0.765g

Note: *Refer to Section 11.4.8 in ASCE 7-16.

5.3 Collateral Seismic Hazards

In addition to seismic shaking, other seismic hazards that may have an adverse affect to the site and/or the structures are: fault ground surface rupture, coseismic ground cracking, seismically induced liquefaction and lateral spreading, seismically induced differential compaction, and seismically induced landsliding. It is our opinion that the potential for collateral seismic hazards to affect the site, and to damage the proposed structures is low with the exception of seismically induced liquefaction. Further discussions related to seismically induced liquefaction are presented below.

5.4 Seismically Induced Liquefaction

Seismically induced liquefaction tends to occur in loose, unconsolidated, noncohesive soils beneath the groundwater table. Liquefaction may cause the soil to settle uniformly or differentially. The magnitude of the liquefaction is a function of the severity of the seismic shaking, the relative density of the soil, the elevation of the groundwater table, and the thickness of the liquefiable soils. The alluvial soils which underlie the site potentially meet this criteria and we therefore performed a quantitative liquefaction analysis.

Based on the results of our field exploration, it is our opinion that the subsurface profiles in Borings B-2 and B-8 are reasonable to assess the liquefaction hazard for the site. Boring B-8 is located near the southwest end of Building A. The subsurface profile in Boring B-8 consists of approximately 26 feet of alluvial deposits overlying Purisima Formation bedrock. Boring B-2 is located near the northeast end of Building A. The subsurface profile in Boring B-2 consists of approximately 11.5 feet of fill overlying approximately 14.5 of alluvial deposits which in turn overlie Purisima

Formation bedrock.

Groundwater was observed in the boreholes during our drilling program. We have constructed an approximate groundwater profile based on the results of our field investigation (Figures A-13 and A-14).

We assumed a groundwater elevation of 12 feet below the existing grade in both Borings B-2 and B-8. The ground shaking parameter used for our analysis was determined using the *2014 National Seismic Hazard Maps* published by the U.S. Geological Survey (USGS) and *ASCE 7-16 Minimum Design Loads for Buildings and Other Structures* (2016) published by the American Society of Civil Engineers. A Maximum Considered Geometric Mean Peak Ground Acceleration (PGA_M), adjusted for Site Class effects, of 0.765g was determined based on the national maps and Section 11.8.3 of ASCE 7-16. A magnitude of 7.9 on the San Andreas Fault Zone was also used in our analysis.

Laboratory testing, consisting of particle size analyses were performed on samples considered representative of the potentially liquefiable soils encountered. Results of our particle size analyses are presented on the Boring Logs and in Appendix B.

A quantitative liquefaction analysis was performed using empirical predictions of earthquake-induced liquefaction potential. The analysis is based on a comparison of the in situ cyclic stress ratio (CSR) with that historically present in areas experiencing liquefaction for a given earthquake magnitude and recorded soil grain size distribution and penetration resistance (as expressed by SPT blows\ft). Our analysis is based on the method presented in the paper titled *Recent Advances In Soil Liquefaction Engineering: A Unified And Consistent Framework* (Seed et al. 2003).

Based on the conditions anticipated during the design seismic event, with a groundwater table at 12 feet below the existing grades and the subsurface profile in Borings B-2 and B-8, our liquefaction analysis determined that a portion of the alluvial soils are potentially liquefiable. Based on the recommended volumetric reconsolidation strains produced by Cetin et. al (2009), settlement of approximately 2 inches, due to volumetric reconsolidation strains, should be anticipated in the locations of Borings B-2 and B-8 during the design seismic event.

6.0 DISCUSSIONS AND CONCLUSIONS

The subsurface profile varies across the site. We encountered Purisima Formation bedrock at or near the ground surface on the northern side of the site, and fill and alluvial deposits overlying the bedrock on the southern side of the site. We constructed two representative cross sections for the subject site based on the results of our field investigation (Cross Section A-A’ and B-B’, Figures A-13 and A-14 in Appendix A). As shown on the cross sections, the subsurface bedrock profile steeply dips to the west, southwest. Due to the large difference in depth to the bedrock, over relatively short distances, the profile has been labeled as approximate and uncertain, and is likely not planar as depicted, yet irregular and stepped.

The fill is located on the northeast side of the existing commercial building and generally consists of loose to very loose silty sands. It is anticipated that the fill will be removed for the proposed development. The alluvial deposits generally consist of interbedded sands, clays, and silts. In general, the cohesionless soils are loose to medium dense and the cohesive soils are stiff. Based on the results of our laboratory testing, the high plasticity clays have a very high expansion potential. The bedrock generally consists of medium dense to very dense, weakly to strongly cemented, siltstone and fine grained sandstone.

Groundwater was encountered within the boreholes during our field exploration in both 2011 and 2021. The groundwater is perched on the underlying Purisima Formation bedrock. We have constructed an approximate groundwater profile based on the results of our field investigation (Figures A-13 and A-14). It is our opinion that the groundwater table may rise during the rainy season.

We analyzed the potential for seismically induced liquefaction within the underlying alluvial deposits on the south side of the site. Based on the results of our liquefaction analyses, under the conditions anticipated during the design seismic event, a portion of the deposits have a high potential for liquefaction. A settlement of approximately 2 inches, due to volumetric reconsolidation strains, should be anticipated for the south portion of the subject site during the design seismic event.

The elevation of the proposed parking garage for Buildings A and B is shown on the cross sections. The excavation for the northeast side of the building will be into bedrock and the southwest side of the building will be underlain by alluvial deposits. Based on the existing/proposed grades, retaining walls will be required to support the proposed grade changes. Due to the differential settlement characteristics, expansive properties, and differential seismic performance of the underlying alluvial deposits relative to the bedrock, the foundation supporting Buildings A and B should extend into the underlying bedrock. Deep foundations will be necessary for the southwest side of the building, as the bedrock extends below the proposed finished grades. Deep foundation members will be necessary for Building C. Caving of layers of the underlying alluvial deposits should be anticipated during the construction of deep foundations.

7.0 RECOMMENDATIONS

7.1 General

Based on the results of our field investigation, laboratory testing, and engineering analysis, it is our opinion, from the geotechnical standpoint, the subject site will be suitable for the proposed development provided the recommendations presented herein are implemented during grading and construction.

We recommend that the proposed multi-unit residential Buildings (A, B, and C) be founded into the underlying bedrock, with the foundation systems consisting of drilled, cast-in-place concrete shafts and grade beams with structurally supported slabs. The drilled, cast-in-place concrete shafts should be founded a minimum of 5 feet into the Purisima Formation bedrock. Drilled, cast-in-place concrete shaft recommendations are presented in Subsection 7.3. The structurally supported slab should be designed to span between grade beams with no soil support. The structural slab has the benefit that should settlement of the underlying soil occu due to differential settlement characteristics or due to seismically induced liquefaction, the resulting deformation will not adversely affect the slab.

The utilities on the southern side of the site should be designed to prevent breakage due to the anticipated seismically induced liquefaction settlement.

It is anticipated that retaining walls will be required to support the proposed grade changes for Buildings A and B. Retaining wall recommendations are provided in Subsection 7.4. Temporary cut slope recommendations are provided in Subsection 7.2.7.

We recommend that a backdrain and drainfield system be constructed per Subsection 7.2.9 for Buildings A and B. We recommend that a subdrain be constructed per Subsection 7.2.9 for Building C.

7.2 Site Grading

7.2.1 Site Clearing

Prior to grading, the areas to be developed for structures, pavements and other improvements, should be stripped of any vegetation and cleared of any surface or subsurface obstructions, including any existing foundations, utility lines, basements, septic tanks, pavements, stockpiled fills, and miscellaneous debris.

Surface vegetation and organically contaminated topsoil should be removed from areas to be graded. The required depth of stripping will vary with the time of year the work is done and should be observed by the Geotechnical Engineer. It is generally anticipated that the required depth of stripping will be 4 to 8 inches.

Holes resulting from the removal of buried obstructions that extend below finished site grades should be backfilled with compacted engineered fill compacted to the requirements of Subsection 7.2.2.

7.2.2 Preparation of On-Site Soils

Drilled, cast-in-place concrete shafts and grade beams with a structural slab, require no reworking of materials other than that necessary to rework materials disturbed during earthwork and construction.

In drive areas (including concrete, asphalt, and non-permeable pavers), the native soil should be overexcavated to a minimum of 1.5 feet below the bottom of the aggregate base course, or 1.5 feet below existing grade, whichever is greater. In addition, all existing fill must be removed to original grades. The exposed surface should then be scarified, moisture conditioned, and compacted to a minimum of 90 percent relative compaction. The material which was removed should then be replaced as engineered fill compacted to a minimum of 90 percent relative compaction. The upper 6 inches of subgrade and all aggregate base and subbase in drive areas shall be compacted to achieve a minimum relative compaction of 95 percent. This zone of reworking should extend laterally a minimum of 2 feet beyond the drive areas.

Beneath new fills, the native soil should be removed to a minimum of 1.5 feet below existing grades. In addition, all existing fill must be removed to original grades. The exposed surface should then be scarified, moisture conditioned, and compacted to a minimum of 90 percent relative compaction. The material which was removed should then be replaced as engineered fill compacted to a minimum of 90 percent relative compaction.

Granular on-site fill soils and alluvial soils may be considered for use as engineered fill. However, use of these soils will require separation from unsuitable clayey soils during excavation and blending and processing to achieve uniform fill material. Note: If this work is done during or soon after the rainy season, or in the spring, the soil may require significant drying prior to use as engineered fill. Regardless of the time of year, moisture conditioning the native soils to achieve moisture requirements should be anticipated. Moisture conditioning may include adding water or drying back the soil to achieve the required moisture. It is the contractors responsibility to adequately process the soil to achieve uniform moisture conditions of the material to be used as engineered fill. The soil should be verified by a representative of CMAG in the field during grading operations. All soils, both existing on-site and imported, to be used as fill, should contain less than 3 percent organics and be free of debris and gravel over 2.5 inches in maximum dimension.

Imported fill material should be approved by a representative of CMAG prior to importing. Soils having a significant expansion potential should not be used as imported fill. **The Geotechnical Engineer should be notified not less than 5 working days in advance of placing any fill or base course material proposed for import**. Each proposed source of import material should be sampled, tested, and approved by the Geotechnical Engineer prior to delivery of any soils imported for use on the site.

All fill should be compacted with heavy vibratory equipment. Fill should be compacted by mechanical means in uniform horizontal loose lifts not exceeding 8 inches in thickness. The relative compaction and required moisture content shall be based on the maximum dry density and optimum moisture content obtained in accordance with ASTM D1557. **The Geotechnical Engineer should observe the overexcavations, and placement of engineered fill.**

Any surface or subsurface obstruction, or questionable material encountered during grading, should be brought immediately to the attention of the Geotechnical Engineer for proper processing as required.

7.2.3 Cut and Fill Slopes

Cut and Fill slopes are not anticipated for the project at this time. Cut and fill slopes may affect the stability of the site, and should be analyzed for overall stability and suitability by the Geotechnical Engineer if project requirements change.

7.2.4 Utility Trenches

Bedding material should consist of sand with SE not less than 30 which may then be jetted.

Granular on-site fill soils and alluvial soils may be considered for use for trench backfill. See Section 7.2.2 for additional information on the use of on-site soils for use as engineered fill. Imported fill should be free of organic material and gravel over 2.5 inches in diameter. Backfill of all exterior and interior trenches should be placed in thin lifts and mechanically compacted to achieve a relative compaction of not less than 95 percent in paved areas and 90 percent in other areas per ASTM D1557. Care should be taken not to damage utility lines.

Utility trenches that are parallel to the sides of a building should be placed so that they do not extend below a line sloping down and away at an inclination of 2:1 H:V (horizontal to vertical) from the bottom outside edge of all footings.

A 3 foot concrete plug should be placed in each trench where it passes under the exterior footings. Anti-seep collars (trench dams) should also be placed in utility trenches on steep slopes to prevent migration of water and sand.

Trenches should be capped with 1.5± feet of impermeable material. Import material should be approved by the Geotechnical Engineer prior to its use.

Trenches must be shored as required by the local regulatory agency, the State Of California Division of Industrial Safety Construction Safety Orders, and Federal OSHA requirements.

7.2.5 Vibration During Compaction

The neighboring parcels are within close proximity to the proposed development The contractor should take all precautionary measures to minimize vibration on the site during grading operations. This may require that the engineered fill be placed in thin lifts using a static roller or hand operated equipment. It is the contractor's responsibility to ensure that the process in which the engineered fill is placed does not adversely affect the neighboring parcels.

7.2.6 Excavating Conditions

We anticipate that excavation of the on-site soils may be accomplished with standard earthmoving and trenching equipment. **Caving, with the cohesionless fill and alluvial deposits beneath the groundwater table should be anticipated.**

Difficult excavating and drilling conditions should be anticipated within the bedrock. The bedrock is cemented and varies from medium dense to very dense. Drilling equipment capable of excavating through rock should be anticipated for the construction of this project.

Groundwater may present a problem during construction. Temporary dewatering during the excavation for Building A and B should be anticipated. Geotextile, rock, or other means may be required to stabilize the base of excavations.

7.2.7 Temporary Excavation Slopes

Temporary cut slopes may be feasible to construct the proposed excavation for Buildings A and B. These excavations must comply with all applicable local, state and federal safety regulations and specifications, which should include reference to the State of California Trenching and Shoring Manual. This includes compliance with California Occupational Safety and Health Regulations (Cal/OSHA). These safety regulations are contained within the larger California Code of Regulations, Title 8 Industrial Relations (CCR Title 8). It is the Contractor's responsibility to maintain a safe work environment and to select the means, methods, and sequencing of all construction operations.

Heavy construction equipment, construction materials, excavated soils, and vehicular traffic may act as surcharge and therefore should not be allowed within a specified distance from the excavation. Recommendations for setback distances can be provided upon request.

For temporary excavations that are adequately dewatered, the on-site fill soils should be considered Type C Soils (CCR, Title 8, Section 1541.1). Type C Soils may be temporarily excavated to provide a maximum slope of 1.5:1 H:V (horizontal to vertical) for a maximum height of 20 vertical feet.

For temporary excavations that are adequately dewatered, the on-site alluvial soils should be considered Type B Soils (CCR, Title 8, Section 1541.1). Type B Soils may be temporarily excavated to provide a maximum slope of 1:1 H:V (horizontal to vertical) for a maximum height of 20 vertical feet.

For temporary excavations that are adequately dewatered, the on-site bedrock may be temporarily excavated to provide a maximum slope of 1:1.5 H:V (horizontal to vertical) for a maximum height of 20 vertical feet.

The excavations are considered temporary excavations only. The recommended slope configurations are based on dry conditions (dewatered and no significant precipitation). **A representative of our firm should be present during the excavation to observe the stability of the excavation slopes and to verify the slope angle.** A representative of our firm should visit the site a minimum of once a week after the excavation to observe the moisture condition of the soil/bedrock and the stability of the excavation slopes. If any tension cracks occur at the top of the excavation during construction, our firm should be notified immediately. All work within the excavation should stop until a representative of our firm is present at the site.

7.2.8 Surface Drainage

Pad drainage should be designed to collect and direct surface water away from structures to approved drainage facilities. A minimum gradient of 2± percent should be maintained and drainage should be directed toward approved swales or drainage facilities. Concentrations of surface water runoff should be handled by providing the necessary structures, paved ditches, catch basins, etc.

All roof eaves should be guttered with the outlets from the downspouts provided with adequate capacity to carry the storm water away from the structure to reduce the possibility of soil saturation and erosion.

Drainage patterns approved at the time of construction should be maintained throughout the life of the structures. The building and surface drainage facilities must not be altered nor any grading, filling, or excavation conducted in the area without prior review by the Geotechnical Engineer.

Irrigation activities at the site should be controlled and reasonable. Planter areas should not be sited adjacent to walls without implementing approved measures to contain irrigation water and prevent it from seeping into walls and under foundations and slabs-on-grade.

The finished ground surface should be planted with erosion resistant landscaping and ground cover and continually maintained to minimize surface erosion.

7.2.9 Subsurface Drainage

A backdrain and drainfield system should be constructed for Building A and B. The backdrain and drainfield may be integrated or constructed as separate systems. A subdrain should be constructed for Building C. A waterproofing expert should be consulted for their recommended moisture and vapor protection measures.

Backdrains should be constructed behind the retaining walls for Buildings A and B. Backdrains should consist of 4 inch diameter Schedule 40 PVC perforated pipe or equivalent, embedded in Caltrans Class 2 permeable drain rock. The drain should be a minimum of 18 inches in width and should extend to within 12 inches from the surface. The upper 12 inches should be capped with native soils or directly beneath the asphalt pavement section or concrete pavement. **Mirafi 180N** filter fabric should be placed between the native soil cap and the drain rock. The pipe should be 4± inches above the trench bottom; a gradient of 2± percent being provided to the pipe and trench bottom; discharging into suitably protected outlets. See Figure 1 for the standard detail for the backdrain.

A 12 inch thick drainfield should be placed beneath the foundation moisture barrier. The base of the excavation should be sloped at a minimum of 2 percent to the perforated pipes as described below. **Mirafi Filterweave 700**, or approved equivalent should be placed at the base of the excavation. The drainfield should incorporate 4 inch diameter Schedule 40, PVC pipe or equivalent, placed on 20 foot centers, embedded in Caltrans Class 2 permeable drain rock. The perforated pipe should be placed in 18 inch wide trenches that are a minimum of 1 foot below the base of the adjacent excavation. The pipes should be 4± inches above the base of the filter fabric; a gradient of 2± percent being provided to the pipe and bottom of the trenches; discharging into suitably protected outlets.

To alleviate the potential for perched groundwater during the rainy season to adversely affect Building C, we recommend a subdrain be constructed. The subdrain should be located on the northeast and south sides of the building. The subdrain should be a minimum of 3 feet below the lowest adjacent grade. The recommendations above for the backdrain, may be applied to the subdrain construction.

Perforations in subdrains are recommended as follows: 3/8 inch diameter, in 2 rows at the ends of a 120 degree arc, at 5 inch centers in each row, staggered between rows, placed downward.

An unobstructed outlet should be provided at the lower end of each segment of the perforated pipe. The outlet should consist of an unperforated pipe of the same diameter, connected to the perforated pipe and extended to a protected outlet at a lower elevation on a continuous gradient of at least 1 percent.

The backdrain and drainfield system should be observed by the Geotechnical Engineer after placement of filterfabric, bedding, and pipe, and prior to the placement of the remaining Caltrans Class 2 permeable. Subdrains should be observed by the Geotechnical Engineer after placement of bedding and pipe and prior to the placement of the remaining Caltrans Class 2 permeable.

7.3 Foundations

7.3.1 Drilled, Cast-In-Place Concrete Shafts

The drilled, cast-in-place concrete shafts for the proposed structures (Buildings A, B, and C) should have a minimum embedment depth of **5± feet below the bottom of the grade beams, or 5 feet into the underlying bedrock, whichever is greater.** For preliminary design purposes, Cross Sections A-A' and B-B' (Figures A-13 and A-14) may be used to determine the depth to the underlying bedrock. The minimum recommended shaft diameter is 18 inches. Shafts should be spaced no closer than 2.5 diameters, center to center. Grade beams should be founded a minimum of 18 inches below the lowest adjacent grade.

The allowable downward axial capacity for a drilled shaft extending into the underlying bedrock may be determined from the following equation:

$$q_{all} = 9\ B^2 + 2\ BD$$

The allowable upward axial capacity for a drilled shaft extending into the underlying bedrock may be determined from the following equation:

$$q_{all} = 1.4\ BD$$

Where q_{all} is the allowable axial load in kips, B is the diameter of the shaft in feet, and D is the depth into the underlying bedrock in feet. The downward capacity includes the weight of the shaft. The upward capacity does not include the weight of the shaft. The axial capacities above apply to a single shaft, as this is the anticipated configuration. The allowable capacities may be increased by 1/3 for seismic loading.

Under static conditions, a passive pressure of 230 psft/ft (equivalent fluid pressure) acting over a plane 1.5 times the shaft diameter may be assumed for design purposes within the alluvial deposits. A passive pressure of 400 psf/ft acting over a plane 3 times the shaft diameter, may be assumed for design purposes within the bedrock. Neglect passive pressure in the top 3 feet of soil.

For seismic design, A passive pressure of 530 psf/ft acting over a plane 3 times the shaft diameter, may be assumed for design purposes within the bedrock. Neglect passive pressure within the alluvial deposits.

The drilled excavations for the cast-in-place concrete shafts should be clean, dry, and free of debris or loose soil. The drilled excavations should not deviate more than 1 percent from vertical. **See Subsection 7.2.6 for anticipated excavation conditions.**

Based on the results of our field exploration, caving should be anticipated during the excavation of the drilled, cast-in-place concrete shafts within the cohesionless fill and alluvial soils beneath the groundwater table. If the contractor chooses to use casing, it must be pulled during the concrete pour. It must be pulled slowly with a minimum of 4 feet of casing remaining embedded within the concrete at all times. If the bottom of the holes are unable to be cleaned with conventional drilling and hand equipment, a bucket auger should be utilized to clean the bottom of the shafts and remove all loose slough.

Groundwater may present a problem during construction. If groundwater is encountered within the shafts and is unable to be pumped from the drilled excavation, a tremie will be required. The tremie must be placed to the bottom of the drilled excavation to remove all groundwater. The end of the tube must remain embedded a minimum of 4 feet into the concrete at all times. The concrete and steel design of the drilled, cast-in-place concrete shaft should be such that a tremie can be easily placed down the center of the excavation.

For drilled, cast-in-place concrete shafts depths in excess of 8 feet, concrete should be placed via a tremie. The end of the tube must remain embedded a minimum of 4 feet into the concrete at all times.

All shaft construction must be observed by the Geotechnical Engineer before steel reinforcement is placed and concrete is poured.

7.3.2 Concrete Slabs

We recommend that concrete slabs be designed as structural slabs (requiring no soil support). The soil may be assumed to support the slab during the curing process. We recommend that the subgrade be proof-rolled just prior to construction to provide a firm, relatively unyielding surface, especially if the surface has been loosened by the passage of construction traffic.

The concrete slab-on-grade for Building C should be underlain by a minimum 4 inch thick capillary break of clean crushed rock. It is recommended that neither Class II baserock nor sand be employed as the capillary break material. The concrete slab for Buildings A and B should be underlain by the drainfield system as outlined in Subsection 7.2.9.

Where moisture sensitive floor coverings are anticipated or vapor transmission may be a problem, a vapor retarder should be placed between the granular layer and the floor slab in order to reduce moisture condensation under the floor coverings. The vapor retarder should be specified by the slab designer. It should be noted that conventional slab-on-grade construction is not waterproof. Under-slab construction consisting of a capillary break and vapor retarder will not prevent moisture transmission through the slab-on-grade. CMAG does not practice in the field of moisture vapor transmission evaluation or mitigation. Where moisture sensitive floor coverings are to be installed, a waterproofing expert should be consulted for their recommended moisture and vapor protection measures.

7.3.3 Settlements

Total and differential settlements beneath drilled, cast-in-place concrete shaft foundations are expected to be within tolerable limits. Vertical movements are not expected to exceed 1 inch. Differential movements are expected to be within the normal range (½ inch) for the anticipated loads and spacings. These preliminary estimates should be reviewed by the Geotechnical Engineer when foundation plans for the proposed structures become available.

7.4 Retaining Structures

7.4.1 Foundations

Building retaining walls should be founded on drilled, cast-in-place concrete shafts per the recommendations of Subsections 7.3.1. Additional lateral resistance and bearing capacity for concrete cantilever retaining walls may be derived from footings placed on the underlying Purisima Formation bedrock. The allowable bearing capacity used should not exceed 4,000 psf. The allowable bearing capacity may be increased by one-third in the case of short duration loads, such as those induced by wind or seismic forces. A passive pressure of 400 psf/ft (equivalent fluid pressure) may be assumed for design purposes within the bedrock. Passive pressures may be increased by one-third for seismic loading. A friction coefficient of 0.4, between the bedrock rough concrete may be assumed for design purposes Where both friction and the passive resistance are utilized for sliding resistance, either of the values indicated should be reduced by one-third.

Recommendations for site retaining wall foundations can be provided upon request. The foundation recommendations provided for site retaining walls are dependent on the proposed location and configuration.

7.4.2 Lateral Pressure Due to Earthquake Motions

For design purposes, the lateral force on retaining walls due to earthquake motions is $6H^2$ lbs/horizontal foot, acting at a point 1/3H above the wall base, where H is the height of the wall in feet.

7.4.3 Removal of Expansive Clayey Soils

In locations where the on-site expansive clayey alluvial deposits are encountered behind retaining walls, we recommend that they be removed to a minimum of 3 feet, horizontally, behind the wall.

7.4.4 Lateral Earth Pressures

The lateral earth pressures presented in Table 2 are recommended for the design of retaining structures with a backdrain and backfill consisting of the on-site granular soils.

Table 2. Lateral Earth Pressures

Soil Profile (H:V)	Equivalent Fluid Pressure (psf/ft)	
	Active Pressure	At-Rest Pressure
Level	39	59
6:1	40	69
3:1	46	78
2:1	58	86

Pressure due to any surcharge loads from adjacent footings, traffic, etc., should be analyzed separately. Pressures due to these loading can be supplied upon receipt of the appropriate plans and loads. Refer to Figure 2.

7.4.5 Backfill

Backfill should be placed under engineering control. Backfill should be compacted to a minimum of 90 percent relative compaction per Subsection 7.2.2, however, precautions should be taken to ensure that heavy compaction equipment is not used immediately adjacent to walls, so as to prevent undue pressures against, and movement of, the walls.

It is recommended that granular, or relatively low expansivity, backfill be utilized, for a width equal to approximately 1/3 times the wall height, and not less than 1.5 feet, subject to review during construction.

The granular backfill should be capped with at least 12 inches of relatively impermeable material.

The use of water-stops/impermeable barriers and appropriate waterproofing should be considered for any basement construction, and for building walls which retain earth.

7.4.6 Backfill Drainage

Backdrains should be provided in the backfill, or weepholes/weepslits should be provided in retaining walls. (It is recommended that backdrains be provided for walls over 4± feet high, for retaining walls which form part of a building structure, and where any staining or efflorescence due to dripping from weepholes/weepslits would be aesthetically unacceptable.) Backdrains recommendations are provided in Subsection 7.2.9.

7.5 Plan Review

The recommendations presented in this report are based on preliminary design information for the proposed project and on the findings of our geotechnical investigation. When completed, the Grading Plans, Foundation Plans and design loads should be reviewed by CMAG prior to submitting the plans and contract bidding. Additional field exploration and laboratory testing may be required upon review of the final project design plans.

7.6 Observation and Testing

Field observation and testing must be provided by a representative of CMAG to enable them to form an opinion regarding the adequacy of the site preparation, the adequacy of fill materials, and the extent to which the earthwork is performed in accordance with the geotechnical conditions present, the requirements of the regulating agencies, the project specifications, and the recommendations presented in this report. Any earthwork performed in connection with the subject project without the full knowledge of, and not under the direct observation of CMAG will render the recommendations of this report invalid.

CMAG should be notified **at least 5 working days** prior to any site clearing or other earthwork operations on the subject project in order to observe the stripping and disposal of unsuitable materials and to ensure coordination with the grading contractor. During this period, a preconstruction meeting should be held on the site to discuss project specifications, observation and testing requirements and responsibilities, and scheduling.

8.0 LIMITATIONS

The recommendations contained in this report are based on our field explorations, laboratory testing, and our understanding of the proposed construction. The subsurface data used in the preparation of this report was obtained from the borings drilled during our field investigation. Variation in soil, geologic, and groundwater conditions can vary significantly between sample locations. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by the Project Geotechnical Engineer and the Geologist, and revised recommendations be provided as required. In addition, if the scope of the proposed construction changes from the described in this report, our firm should also be notified.

Our investigation was performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report.

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 that it is ensured that the Contractor and Subcontractors implement such recommendations in the field. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

This firm does not practice or consult in the field of safety engineering. We do not direct the Contractor's operations, and we are not responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the Contractor. The Contractor should notify the Owner if he considers any of the recommended actions presented herein to be unsafe.

The findings of this report are considered valid as of the present date. However, changes in the conditions of a site can occur with the passage of time, whether they be due to natural events or to human activities on this or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this report may become invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

The scope of our services mutually agreed upon did not include any environmental assessment or study for the presence of hazardous to toxic materials in the soil, surface water, or air, on or below or around the site. CMAG is not a mold prevention consultant; none of our services performed in connection with the proposed project are for the purpose of mold prevention. Proper implementation of the recommendations conveyed in our reports will not itself be sufficient to prevent mold from growing in or on the structures involved.

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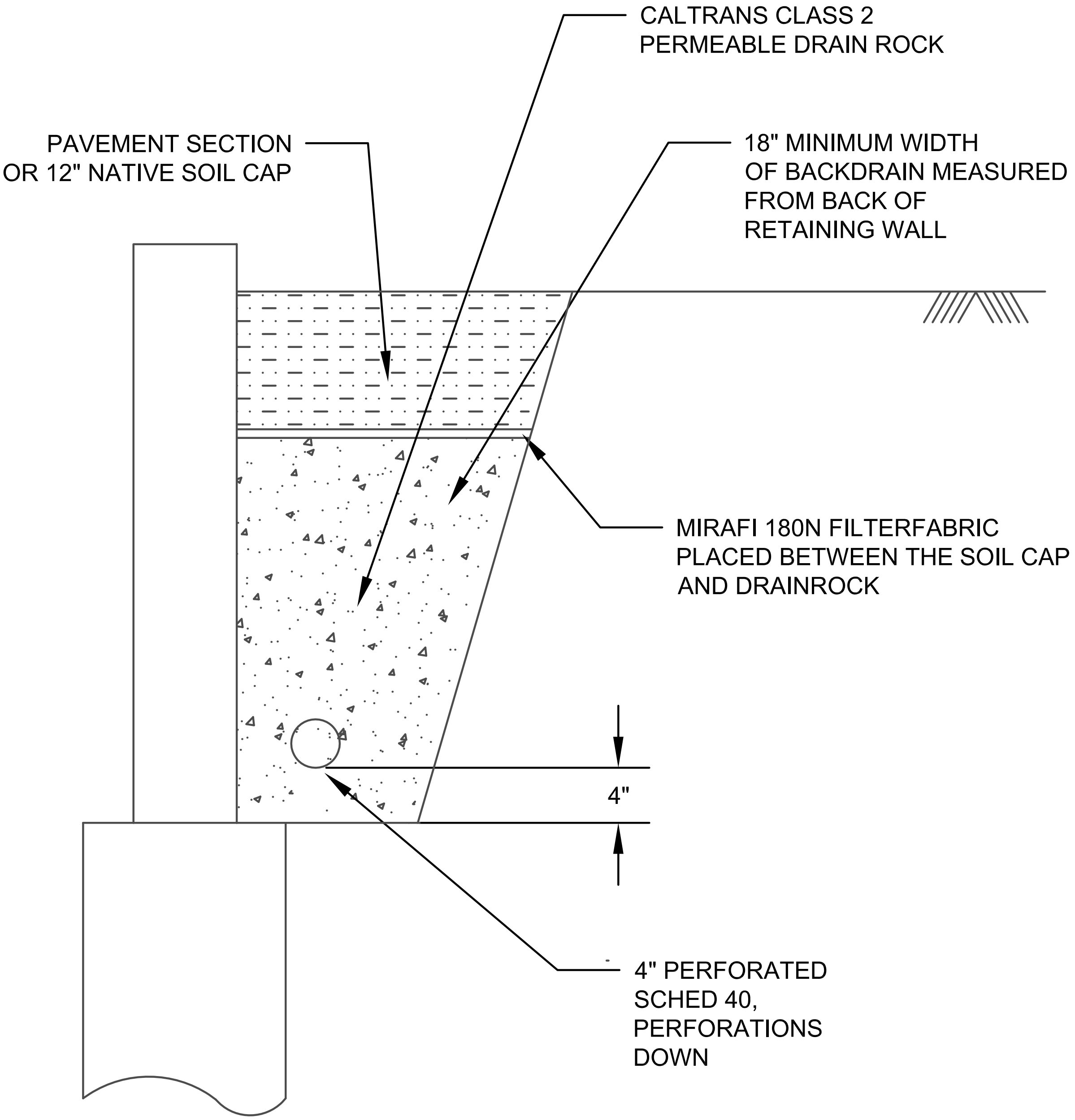
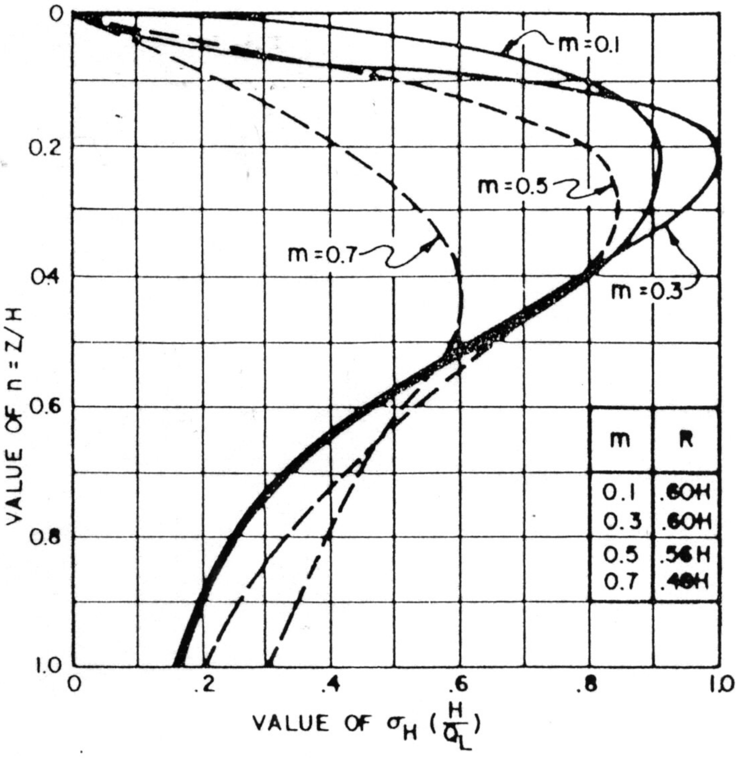


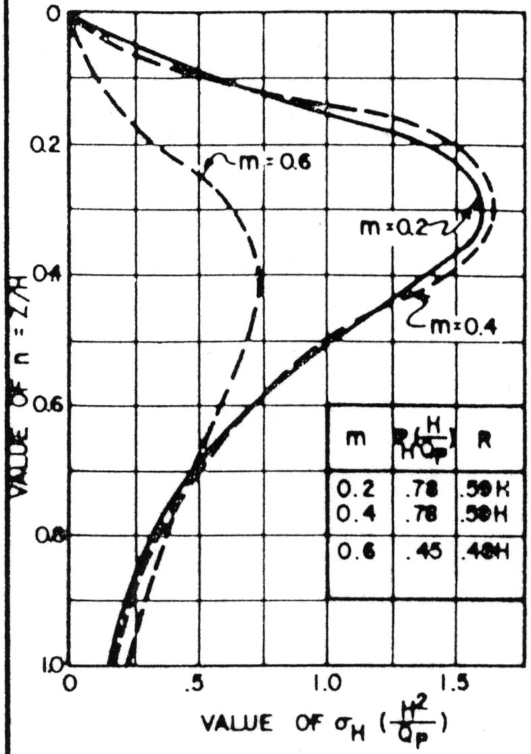
NOTES:

1. DRAWING IS NOT TO SCALE
2. 2± PERCENT TO PIPE AND TRENCH BOTTOM
3. PERFORATED SCHED 40 PVC PIPE, OR APPROVED EQUIVALENT, CONNECTED TO CLOSED CONDUITS THAT DISCHARGE TO AN APPROVED LOCATION
4. INSTALL CLEAN OUTS AT APPROVED LOCATIONS

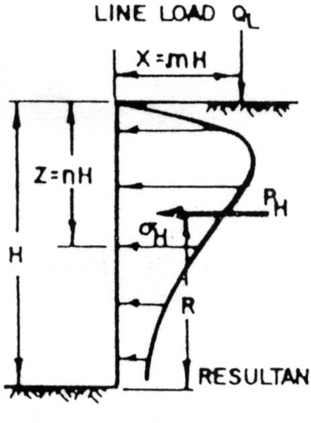
CMAG ENGINEERING	TYPICAL BACKDRAIN DETAIL	FIGURE 1



m	R
0.1	.60H
0.3	.60H
0.5	.56H
0.7	.48H



m	$P_H(\frac{H}{Q_p})$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H



FOR m ≤ 0.4:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{0.20n}{(0.16+n^2)^2}$$

$$P_H = 0.55Q_L$$

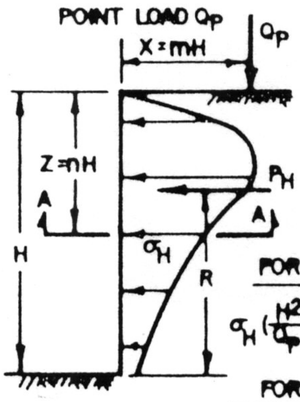
FOR m > 0.4:

$$\sigma_H\left(\frac{H}{Q_L}\right) = \frac{1.28m^2n}{(m^2+n^2)^2}$$

RESULTANT $P_H = \frac{0.64Q_L}{(m^2+1)}$

PRESSURES FROM LINE LOAD Q_L

(BOUSSINESQ EQUATION MODIFIED BY EXPERIMENT)



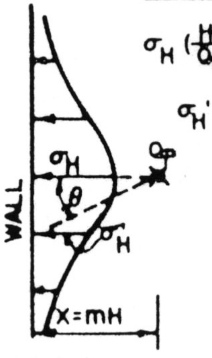
FOR m ≤ 0.4:

$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{0.28n^2}{(0.16+n^2)^3}$$

FOR m > 0.4:

$$\sigma_H\left(\frac{H^2}{Q_p}\right) = \frac{1.77m^2n^2}{(m^2+n^2)^3}$$

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



SECTION A-A

PRESSURES FROM POINT LOAD Q_p

(BOUSSINESQ EQUATION
MODIFIED BY EXPERIMENT)

CMAG ENGINEERING	SURCHARGE PRESSURE DIAGRAM	FIGURE 2

APPENDIX A

FIELD EXPLORATION PROGRAM

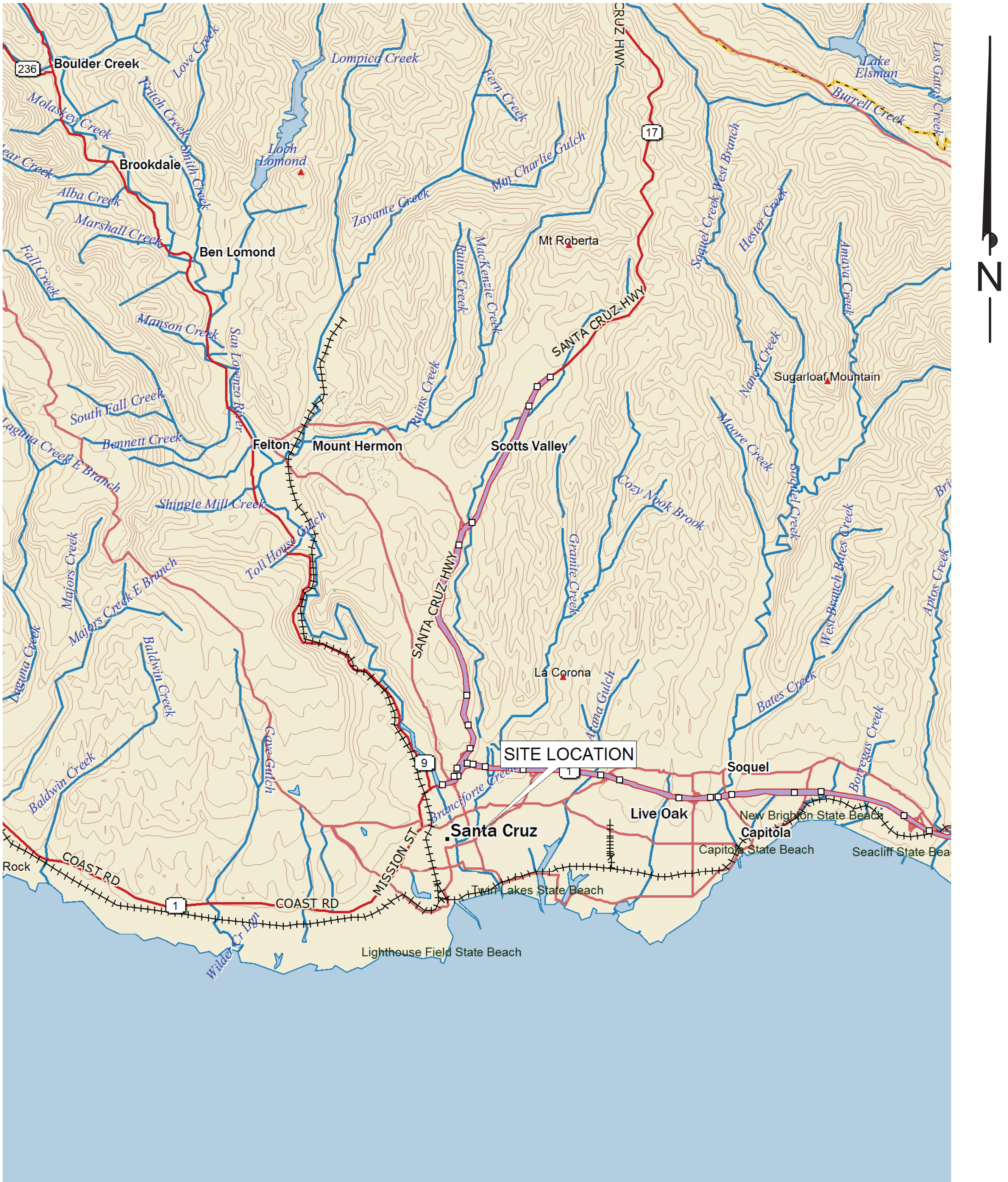
FIELD EXPLORATION PROCEDURES

Subsurface conditions were explored by drilling 8 borings to depths between 6.5± and 34± feet below the existing grades. Borings B-1 through B-7 were drilled with a truck mounted drill rig equipped with 4 inch diameter solid stem augers. Boring B-8 was advanced with a track mounted drill rig equipped with 6 inch diameter solid stem augers. The Key to The Logs and the Logs of the Borings are included in Appendix A, Figures A-4 through A-12. The approximate location of the borings are shown on the Site Map and Boring Location Plan, Figure A-3.

The earth materials encountered in the borings were continuously logged in the field by a representative of CMAG. Bulk and relatively undisturbed samples for identification and laboratory testing were obtained in the field. These samples were classified based on field observations and laboratory tests. The classification is in accordance with the Unified Soil Classification System (Figure A-3).

Representative soil samples were obtained by means of a drive sampler, the hammer weight and drop being 140 lb and 30 inches, respectively. These samples were recovered using a 3 inch outside diameter Modified California Sampler or a 2 inch outside diameter Terzaghi Sampler. The number of blows required to drive the samplers 12 inches are indicated on the Boring Logs. The penetration test data for the Terzaghi driven samples has been presented as N_{60} values. The N_{60} values are also indicated on the Boring Logs.

Two representative cross sections were obtained for the subject site. See Cross Section A-A' and B-B', Figures A-13 and A-14. For an explanation of the symbols and units on the cross sections, see Section 4.0 of the report.



SCALE: 1:125,000

1 0 1 2

MILE

BASEMAP: DeLorme Topo USA®

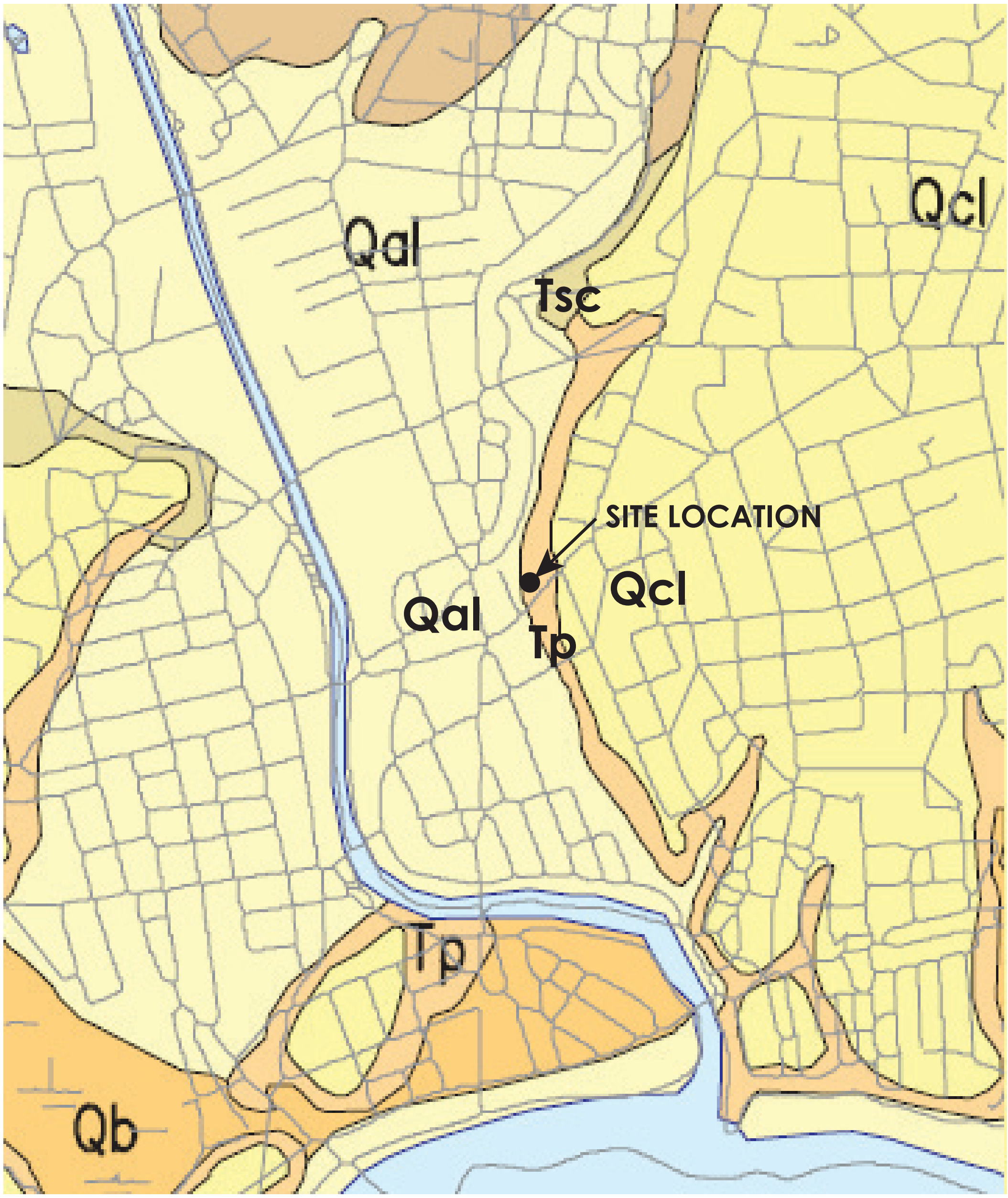
CMAG ENGINEERING

SITE LOCATION MAP

515 Soquel Avenue

FIGURE

A-1



EXPLANATION

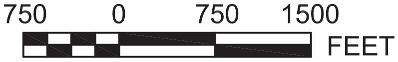
UNITS

Qal	Alluvial Deposits (Holocene)
Qb	Basin Deposits (Holocene)
Qcl	Lowest Emergent Coastal Terrace Deposits (Pleistocene)
Tp	Purisima Formation (Pliocene and upper Miocene)
Tsc	Santa Cruz Mudstone (upper Miocene)

SYMBOLS

╲ GEOLOGIC CONTACT

SCALE: 1 INCH = 1500 FEET



BASEMAP: Brabb, E.E. (1997). *Geologic Map of Santa Cruz County, California*: A digital Database:
U.S. Geological Survey Open File Report 97-489-2373, Scale 1:62,500.

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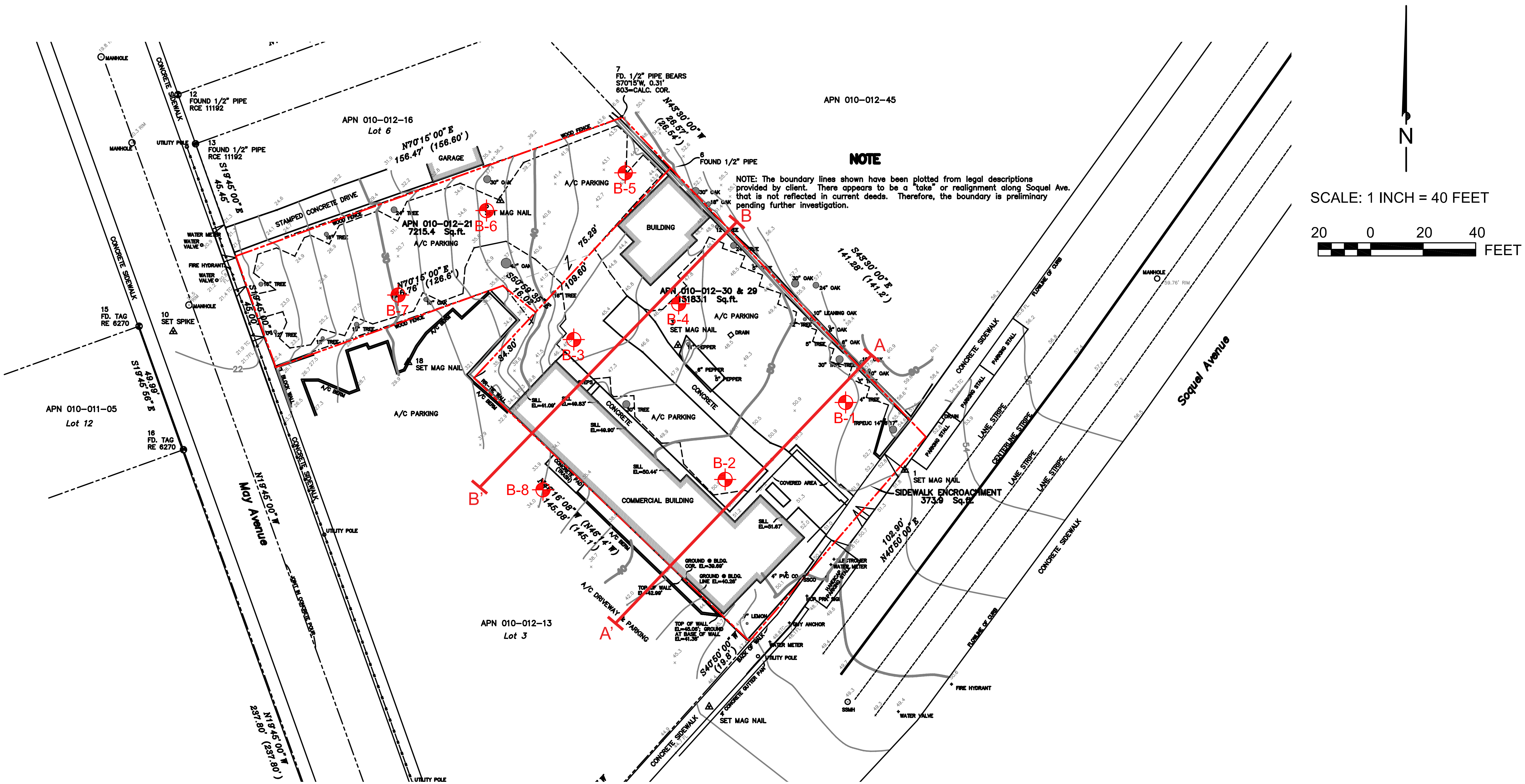
LOCAL GEOLOGIC MAP

515 Soquel Avenue

FIGURE

A-2



NOTE: The boundary lines shown have been plotted from legal descriptions provided by client. There appears to be a "take" or realignment along Soquel Ave. that is not reflected in current deeds. Therefore, the boundary is preliminary pending further investigation.

BASEMAP: EDMUNDSON & ASSOCIATES LAND SURVEYING (4/13/11). *Topographic Survey Map of Assessor's Parcel Nos. 010-012-21, -29, -30, Santa Cruz County, California*. Original Scale: 1" = 20'. Job #11016

EXPLANATION OF SYMBOLS

A ⊢—⊣ A'	LOCATION OF CROSS SECTION A-A'
⊕ B-1	APPROXIMATE LOCATION OF BORING B-1

CMAG ENGINEERING

SITE MAP AND BORING LOCATION PLAN

515 Soquel Avenue

FIGURE

A-3

KEY TO LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS More than half of the material is larger than the No. 200 sieve	GRAVELS More than half of the coarse fraction is larger than the No. 4 sieve	CLEAN GRAVELS (Less than 5% fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS More than half of the coarse fraction is smaller than the No. 4 sieve	CLEAN SANDS (Less than 5% fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		SAND WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS More than half of the material is smaller than the No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50		ML	Inorganic silts and very fine sands, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50		MH	Inorganic silts, micaceous or diatomacaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

GRAIN SIZE LIMITS

SILT AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. 12 in.

US STANDARD SIEVE SIZE

RELATIVE DENSITY	
SAND AND GRAVEL	BLOWS/FT*
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY	
SILT AND CLAY	BLOWS/FT*
VERY SOFT	0 - 2
SOFT	2 - 4
FIRM	4 - 8
STIFF	8 - 16
VERY STIFF	16 - 32
HARD	OVER 32

MOISTURE CONDITION
DRY
MOIST
WET

BEDROCK
(GROUP SYMBOL) Brackets Denote Bedrock

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 inch I.D.) split spoon (ASTM D-1586).

CMAG ENGINEERING

FIGURE
A-4

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-1
Project:	515 Soquel Avenue Santa Cruz County, California	Location:	See Figure A-3, Boring Location Plan
		Elevation:	52± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem Auger, 140lb. Safety Hammer
Logged By:	ALG		

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~2" A/C, ~1' Baserock					
	(ML)	⧅		**Tp:** Light Olive Brown SILTSTONE. Dense, Moist. (Sandy Silt, Sand - Fine Grained). Strongly Cemented.	102				
5		◫		Material Consistent - Dark Greenish Gray.	63	42		36.2	Sulfate
10	(ML)	◫		Dark Greenish Gray and Light Yellowish Brown SILTSTONE. Dense, Moist. (Sandy Silt, Sand - Fine Grained). Moderately Cemented. Increase in Sand Content.	52	38		58.0	
15	(ML-SM)	◫		Dark Greenish Gray SILTSTONE to SANDSTONE. Dense, Moist. (Sandy Silty to Silty Sand, Sand - Fine Grained). Moderately Cemented.	74	59		50.3	
	(ML)	◫		Dark Greenish Gray SILTSTONE. Very Dense, Moist. (Sandy Silt, Sand - Fine Grained). Moderately Cemented.	70	62		49.1	
20				Boring Terminated at 20± ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.					
25									
30									
35									

CMAG ENGINEERING

FIGURE
A-5

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-2
Project:	515 Soquel Avenue	Location:	See Figure A-3, Boring Location Plan
	Santa Cruz County, California	Elevation:	51± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem
Logged By:	ALG		Auger, 140lb. Safety Hammer

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
		◫		~2" A/C, ~1.5' Baserock					
	ML			**af:** Black Sandy SILT. Very Loose, Moist, Non Plastic. Sand - FG to CG.	4	3		15.3	
5	SM	◫		Yellowish Brown Silty SAND w/ Light Olive Brown Siltstone Gravels. Very Loose, Moist, Non Plastic. Sand - Fine to Coarse Grained.	5	3		20.7	
	SM	◫		Black Silty SAND. Loose, Moist to Wet, Non Plastic. Sand - Fine to Coarse Grained.	6	4		23.4	
10									
	▽								
	SC	◫		**Qal:** Brown Clayey SAND. Medium Dense, Wet, Plastic. Sand - Fine to Coarse Grained.	20	16		19.7	FC = 49.2%
15									
	SC	◫		Yellowish Brown and Brown Clayey SAND. Medium Dense, Wet, Plastic to Non Plastic. Sand - Fine to Coarse Grained.	14	12		23.2	Sulfate FC = 47.2%
20									
	CH	◫		Yellowish Brown Sandy Fat CLAY. Stiff, Moist, Plastic. Sand - Fine Grained.	11	10		44.4	
25									
	(ML)	◫		**Tp:** Olive Brown and Dark Yellowish Brown SILTSTONE. Medium Dense, Moist. (Sandy Silt, Sand - Fine Grained). Weakly to Moderately Cemented.	28	25		72.2	
30									
	(ML)	◫		Dark Greenish Gray SILTSTONE. Very Dense, Moist. (Sandy Silt, Sand - Fine Grained). Strongly Cemented.	60	56		54.6	
35				Boring Terminated at 33± ft. Groundwater Encountered at 12± ft., Boring Backfilled With Cuttings.					

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FIGURE
A-6

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-3
Project:	515 Soquel Avenue	Location:	See Figure A-3, Boring Location Plan
	Santa Cruz County, California	Elevation:	45± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem
Logged By:	ALG		Auger, 140lb. Safety Hammer

⧄ 2" Ring Sample

⧅ 2.5" Ring Sample

☒ Bulk Sample

◫ Terzaghi Split Spoon Sample

▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~2" A/C					
	SC-SM			**af:** Yellowish Brown and Dark Yellowish Brown Clayey SAND to					
		⧅		Silty SAND. Very Loose to Loose, Moist, Non Plastic.	6		104.8	18.6	
		◫		Sand - Fine to Medium Grained.	8	5		19.9	
5									
	SC			**Qal:** Black Clayey SAND. Loose, Wet, Plastic to Non Plastic.					
10	CH	⧅		Light Olive Brown and Yellowish Brown Sandy Fat CLAY. Stiff, Moist,	11		101.6	21.9	
		◫		Plastic. Sand - Fine to Coarse Grained.	21	16		21.8	
15	CH	◫		Light Olive Brown and Yellowish Brown Sandy Fat CLAY. Very Stiff,	25	21		28.6	
				Moist, Plastic. Sand - Fine to Coarse Grained.					
				Boring Terminated at 16.5± ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.					
20									
25									
30									
35									

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FIGURE A-7

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-4
Project:	515 Soquel Avenue Santa Cruz County, California	Location:	See Figure A-3, Boring Location Plan
		Elevation:	47± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem Auger, 140lb. Safety Hammer
Logged By:	ALG		

◩ 2" Ring Sample ◪ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~4" A/C					
	SC-SM	◫		**af:** Black and Yellowish Brown Silty SAND and Clayey SAND w/ Siltstone Gravels. Loose, Moist, Non Plastic. Sand - Fine to Coarse Grained.	10	7		15.9	
5	(ML)	◫		**Tp:** Olive Brown SILTSTONE. Loose, Moist. (Sandy Silt, Sand - Fine Grained). Weakly Cemented.	13	9		61.2	
10				Boring Terminated at 6.5± ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.					
15									
20									
25									
30									
35									

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-5
Project:	515 Soquel Avenue	Location:	See Figure A-3, Boring Location Plan
	Santa Cruz County, California	Elevation:	43± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem
Logged By:	ALG		Auger, 140lb. Safety Hammer

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ⊠ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
	CL			~3" A/C, 4" of Baserock.					
		◫		**Qal:** Very Dark Grayish Brown Sandy Lean CLAY. Stiff, Moist, Plastic. Sand - Fine to Coarse Grained.	15	10			Sulfate
								45.0	
5	(ML- SM)	◫		**Tp:** Light Olive Brown SILTSTONE to SANDSTONE. Medium Dense, Moist. (Silty Sand to Sandy Silt, Sand - Fine Grained). Weakly Cemented.	36	25		48.7	
10				Boring Terminated at 6.5± ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.					
15									
20									
25									
30									
35									

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FIGURE
A-9

LOG OF EXPLORATORY BORING

Project No.: 11-106-SC
Project: 515 Soquel Avenue
Santa Cruz County, California
Date: March 30, 2011
Logged By: ALG
Boring: B-6
Location: See Figure A-3, Boring Location Plan
Elevation: 37± ft.
Method of Drilling: Truck Mounted Drill Rig, 4in. Solid Stem Auger, 140lb. Safety Hammer

⧄ 2" Ring Sample
⧅ 2.5" Ring Sample
⊠ Bulk Sample
◫ Terzaghi Split Spoon Sample
▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~4" A/C					
5	SM	◫		**Qal:** Black Silty SAND. Very Loose, Wet, Non Plastic. Sand - Fine to Coarse Grained.	3	2		22.3	
	(SM)	◫		**Tp:** Light Olive Brown and Dark Yellowish Brown SANDSTONE. Medium Dense, Moist. (Silty Sand, Sand - Fine Grained). Weakly Cemented.	34	24		15.3	
10				Boring Terminated at 8.5± ft. Groundwater Not Encountered. Boring Backfilled With Cuttings.					
15									
20									
25									
30									
35									

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FIGURE
A-10

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-7
Project:	515 Soquel Avenue	Location:	See Figure A-3, Boring Location Plan
	Santa Cruz County, California	Elevation:	31± ft.
Date:	March 30, 2011	Method of Drilling:	Truck Mounted Drill Rig, 4in. Solid Stem
Logged By:	ALG		Auger, 140lb. Safety Hammer

⧄ 2" Ring Sample ⧅ 2.5" Ring Sample ☒ Bulk Sample

◫ Terzaghi Split Spoon Sample ▽ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~3" A/C					
	SM-SC	⧅		**Qal:** Black Silty and Clayey SAND. Very Loose, Wet, Non Plastic to Plastic.	7	2	102.6	20.1	
	SC-CL	◫		Brown Clayey SAND to Sandy Lean CLAY. Medium Dense, Moist, Plastic.	17	11		13.5	
				Sand - Fine to Coarse Grained.					FC = 50.9%
5									
	ML			Olive Brown and Dark Yellowish Brown Sandy SILT.					
		◫		Stiff, Moist to Wet, Plastic to Non Plastic. Sand - Fine Grained.	17	12		25.2	FC = 70.6%
10									
	▽								
	ML	◫		Olive Brown Sandy SILT. Firm, Wet, Plastic. Sand - Fine Grained.	10	8		59.9	
15				Boring Terminated at 13.5± ft. Groundwater Encountered at 12± ft. Boring Backfilled With Cuttings.					
20									
25									
30									
35									

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FIGURE
A-11

LOG OF EXPLORATORY BORING

Project No.:	11-106-SC	Boring:	B-8
Project:	515 Soquel Avenue	Location:	See Figure A-3, Boring Location Plan
	Santa Cruz County, California	Elevation:	34± ft.
Date:	March 18, 2021	Method of Drilling:	Track Mounted Drill Rig, 6in. Solid Stem
Logged By:	ALG		Auger, 140lb. Automatic Trip Hammer

2" Ring Sample
2.5" Ring Sample
Bulk Sample
Terzaghi Split Spoon Sample
∇ Static Water Table

Depth (ft.)	Soil Type	Undisturbed	Bulk	Description	Blows / Foot	N_{60}	Dry Density (pcf)	Moisture Content (%)	Other Tests
				~2" A/C, ~4" Baserock					
	CL			**Qal:** Black Lean CLAY Grading to a Clayey SAND. Firm, Moist to Wet,			108.3	17.5	q_u = 991 psf
	SC			Plastic to Non Plastic. Sand - Fine to Coarse Grained.	11			16.2	
	CH			Grayish Brown Sandy Fat CLAY. Stiff, Moist, Plastic. Sand - Fine Grained.	10	10		24.4	EI = 130
5									
							94.5	27.5	q_u = 5,897 psf
	CH			Light Olive Brown with Dark Yellowish Brown Sandy Fat CLAY with Sand.	18		91.8	28.5	Consol
				Very Stiff Grading to Stiff, Moist, Plastic. Sand - Fine Grained.	8	9		31.8	
10									
	∇								
	CH			Material Consistent - Stiff.			97.6	31.7	q_u =2,543 psf
					24			20.8	
15	SM			Yellowish Brown Silty SAND. Medium Dense, Wet, Non Plastic.	21	25		25.9	FC = 14.6%
				Sand - Fine to Medium Grained.					
	SP			Yellowish Brown Poorly Graded SAND. Medium Dense, Wet, Non Plastic.				27.3	FC = 3.8%
				Sand - Fine to Medium Grained.	13	17			
20	ML			Lower 6" - Grayish Brown Sandy SILT. Medium Dense, Wet, Non Plastic.				27.9	FC = 68.8%
				Sand - Fine Grained.					
	SM-GM			Dark Gray Silty SAND with Gravel to Silty GRAVEL with Sand. Very Dense,	100+	100+		11.9	FC = 11.9%
				Wet, Non Plastic. Sand - Fine to Coarse Grained. Gravel - up to 1.5",					
25				Subangular to Angular. Schist Gravels.					
	(ML)			**Tp:** Olive Brown and Dark Yellowish Brown SILTSTONE. Very Dense,					
				Moist. (Sandy Silt, Sand - Fine Grained). Strongly Cemented.	100+	100+		58.6	
30									
	(ML)			Dark Greenish Gray SILTSTONE. Very Dense, Moist. (Sandy Silt,					
				Sand - Fine Grained). Strongly Cemented.	100+	100+		61.3	
				Boring Terminated at 34± ft.					
35				Groundwater Encountered at 12± ft., Boring Backfilled With Cuttings.					

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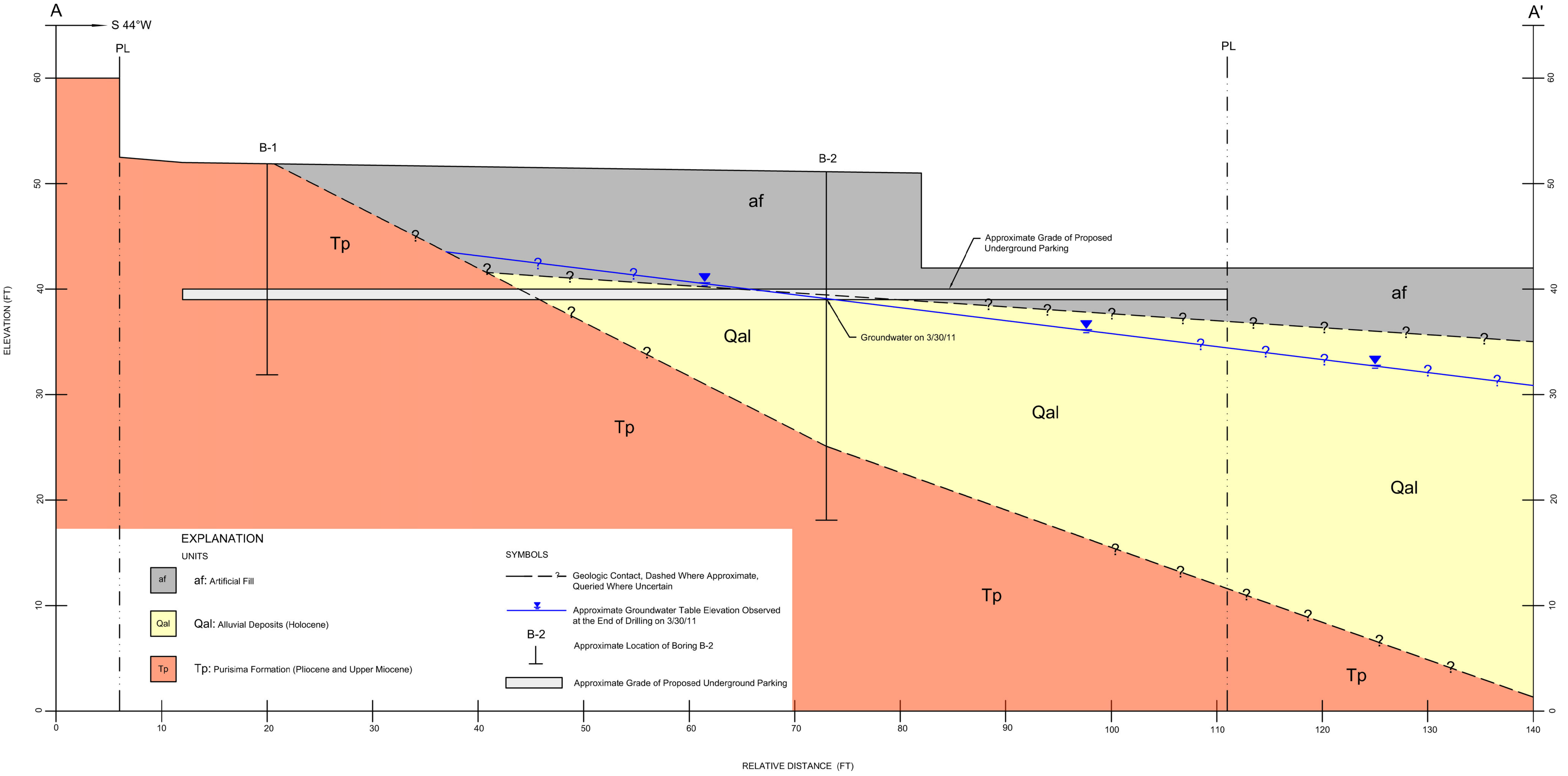
FIGURE
A-12

CROSS SECTION A-A'

SCALE: 1" = 10' H = V



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CROSS SECTION A-A'

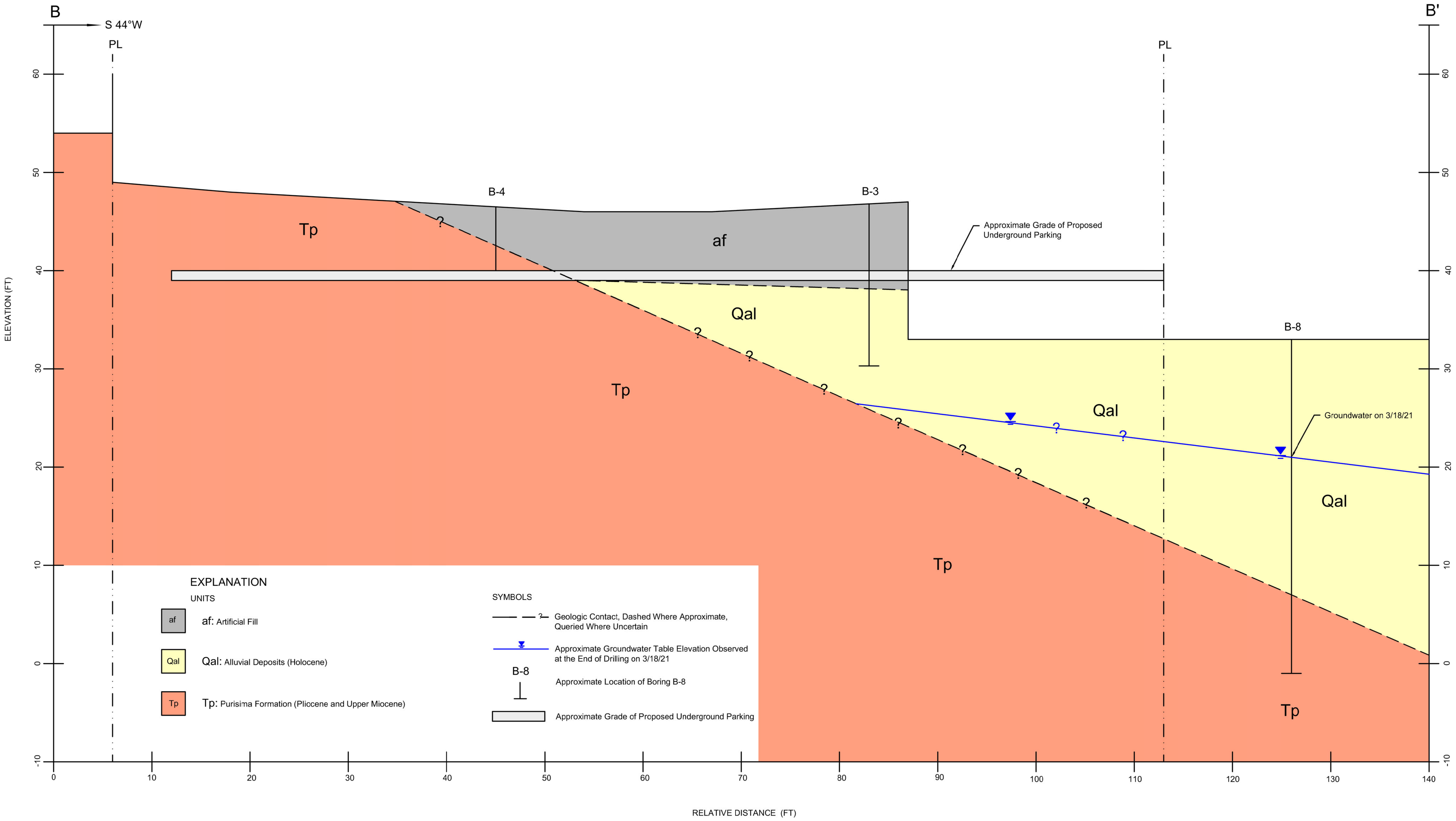
515 Soquel Avenue

FIGURE

A-13

CROSS SECTION B-B'

SCALE: 1" = 10' H = V



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CROSS SECTION B-B'

515 Soquel Avenue

FIGURE

A-14

APPENDIX B

LABORATORY TESTING PROGRAM

LABORATORY TESTING PROCEDURES

Classification

Earth materials were classified according to the Unified Soil Classification System in accordance with ASTM D 2487 and D 2488. See Figure A-4. Moisture content and dry density determinations were made for representative, relatively undisturbed samples in accordance with ASTM D 2216. Results of moisture-density determinations, together with classifications, are shown on the Boring Logs, Figures A-5 through A-12.

Unconfined Compression

Unconfined compression tests were performed on representative samples of the on-site soils in accordance with ASTM D 2166. The test results are presented on the Boring Logs and on Figures B-1 through B-3.

Consolidation

A one dimensional consolidation test using incremental loadings was performed in accordance with ASTM D 2435 on a representative, relatively undisturbed sample of the underlying soils. The sample was saturated prior to the test to simulate possible adverse field conditions. A saturating device was used which permitted the sample to absorb moisture while preventing volume change. The test results are presented on Figure B-4.

Particle Size Distribution

Particle size distribution tests were performed on representative samples of the underlying soils to determine the particle size distribution in accordance with ASTM D 422. The test results are presented on Figures B-5 through B-12.

Expansion Index

An expansion index test was performed on a representative remolded samples of the on-site soils in accordance with the ASTM D 4829. The test results are presented in Table B-1.

Table B-1. Expansion Index Test Results

Test Location	Soil Type	Expansion Index	Expansion Potential
B-8 at 3.5 to 5 Feet	CH	130	Very High

Soluble Sulfates

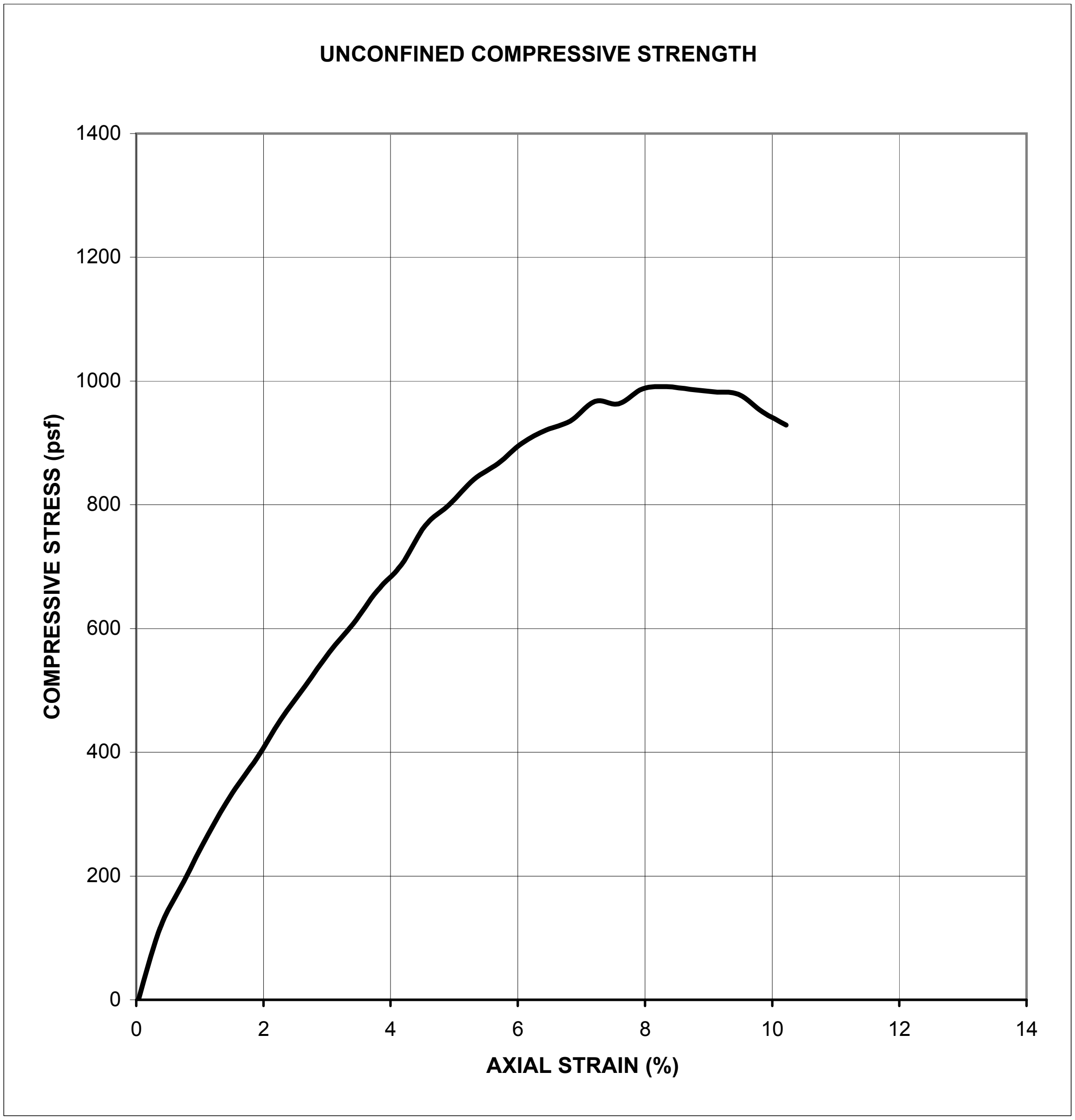
The soluble sulfate content was determined for samples considered representative of the on-site soils likely to come into contact with concrete in accordance with Caltrans 417. The test results are presented in Table B-2.

Table B-2. Soluble Sulfate Test Results (Caltrans 417)

Test Location	Soil Type	Sulfate Content (%)	Sulfate Exposure Class
B-1 at 3 Feet	(ML)	.059	Negligible
B-2 at 18 Feet	SC	.0074	Negligible
B-5 at 1 Feet	CL	.0025	Negligible

BORING:	B-8	**UNDISTURBED**	UCS
DEPTH (ft):	2		
SOIL TYPE (USCS):	SC-CL		q_u = 991 psf

MOISTURE: **SATURATED**



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UNCONFINED COMPRESSIVE STRENGTH RESULTS

515 Soquel Avenue

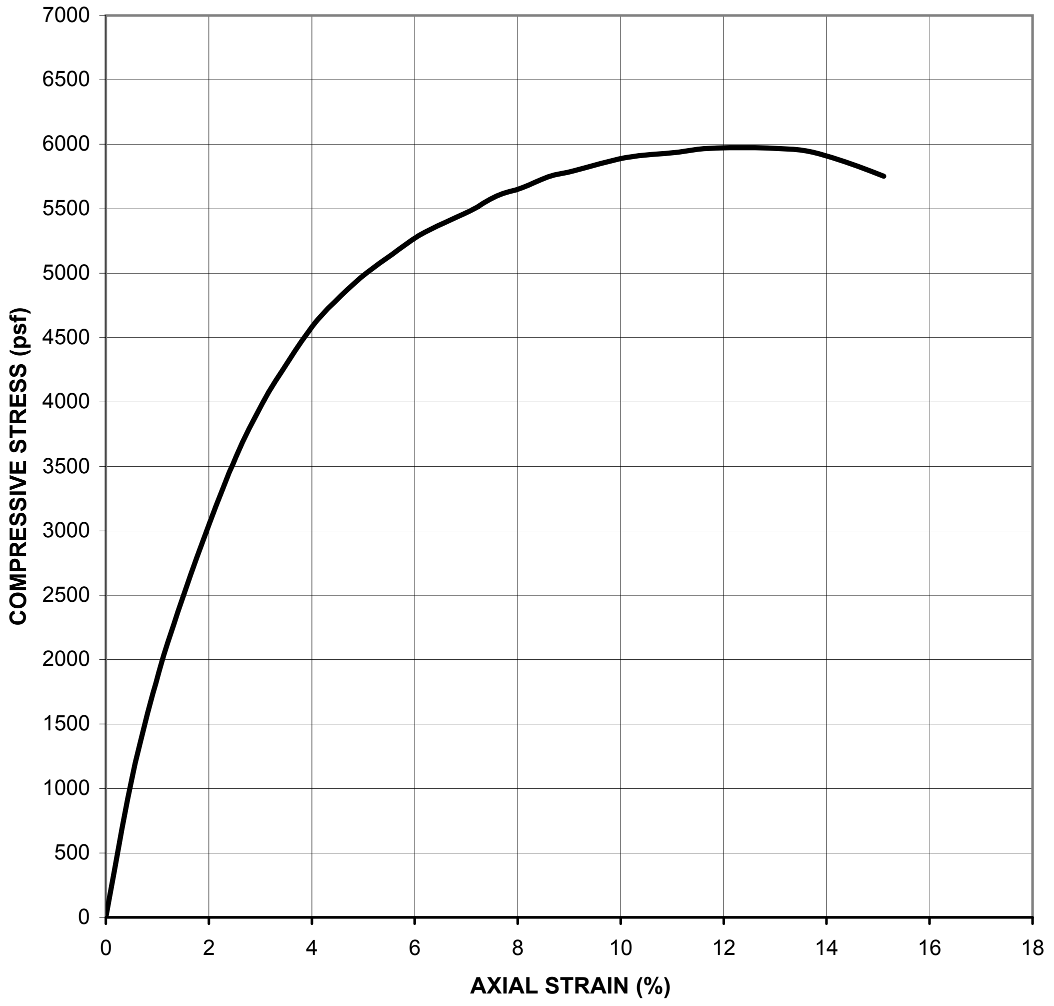
FIGURE

B-1

BORING:	B-8	**UNDISTURBED**	UCS
DEPTH (ft):	7		
SOIL TYPE (USCS):	CH		q_u = 5,897 psf

MOISTURE: **SATURATED**

UNCONFINED COMPRESSIVE STRENGTH



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UNCONFINED COMPRESSIVE STRENGTH RESULTS

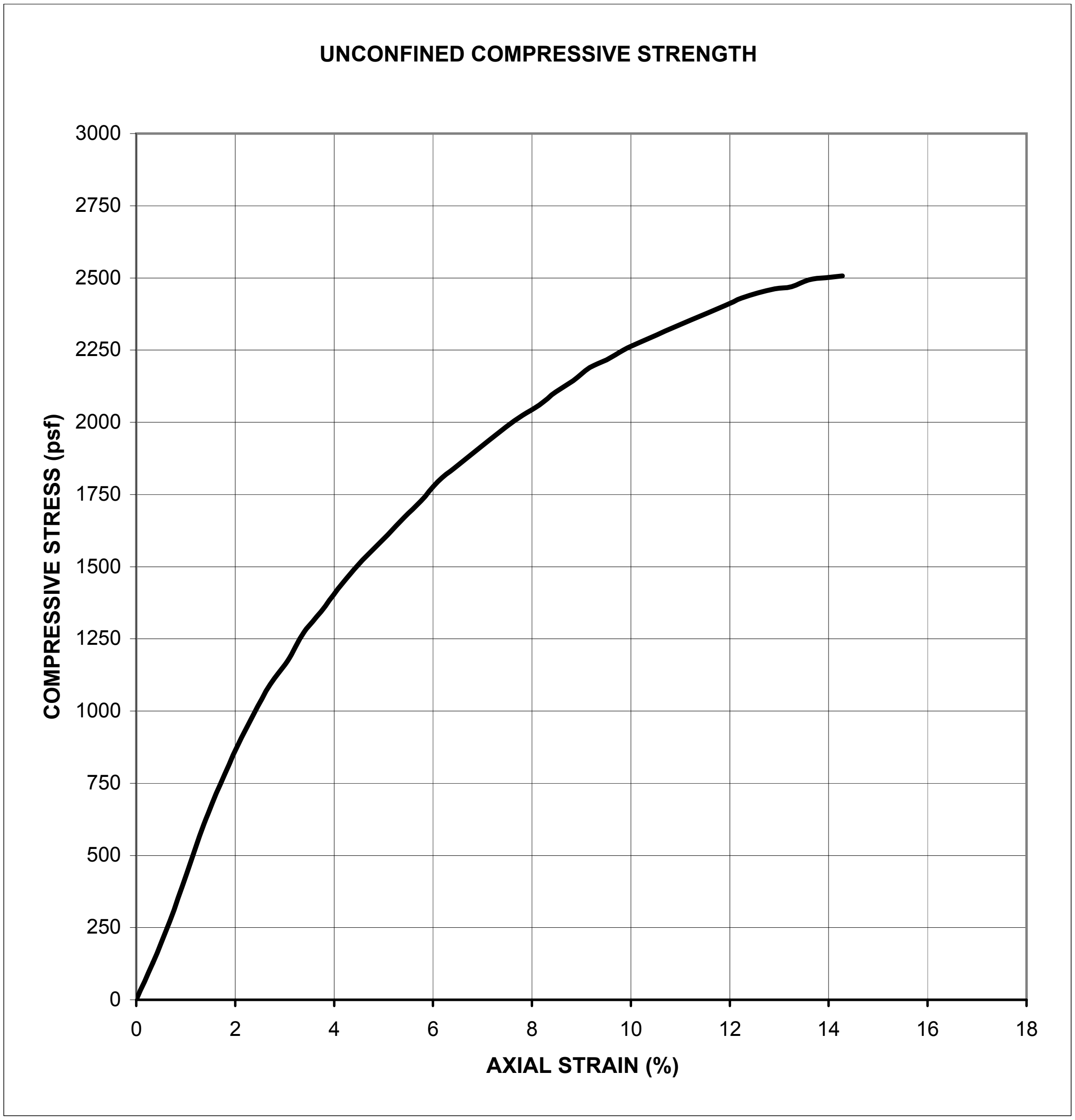
515 Soquel Avenue

FIGURE

B-2

BORING:	B-8	**UNDISTURBED**	UCS
DEPTH (ft):	13		
SOIL TYPE (USCS):	CH		q_u = 2,543 psf

MOISTURE: **SATURATED**



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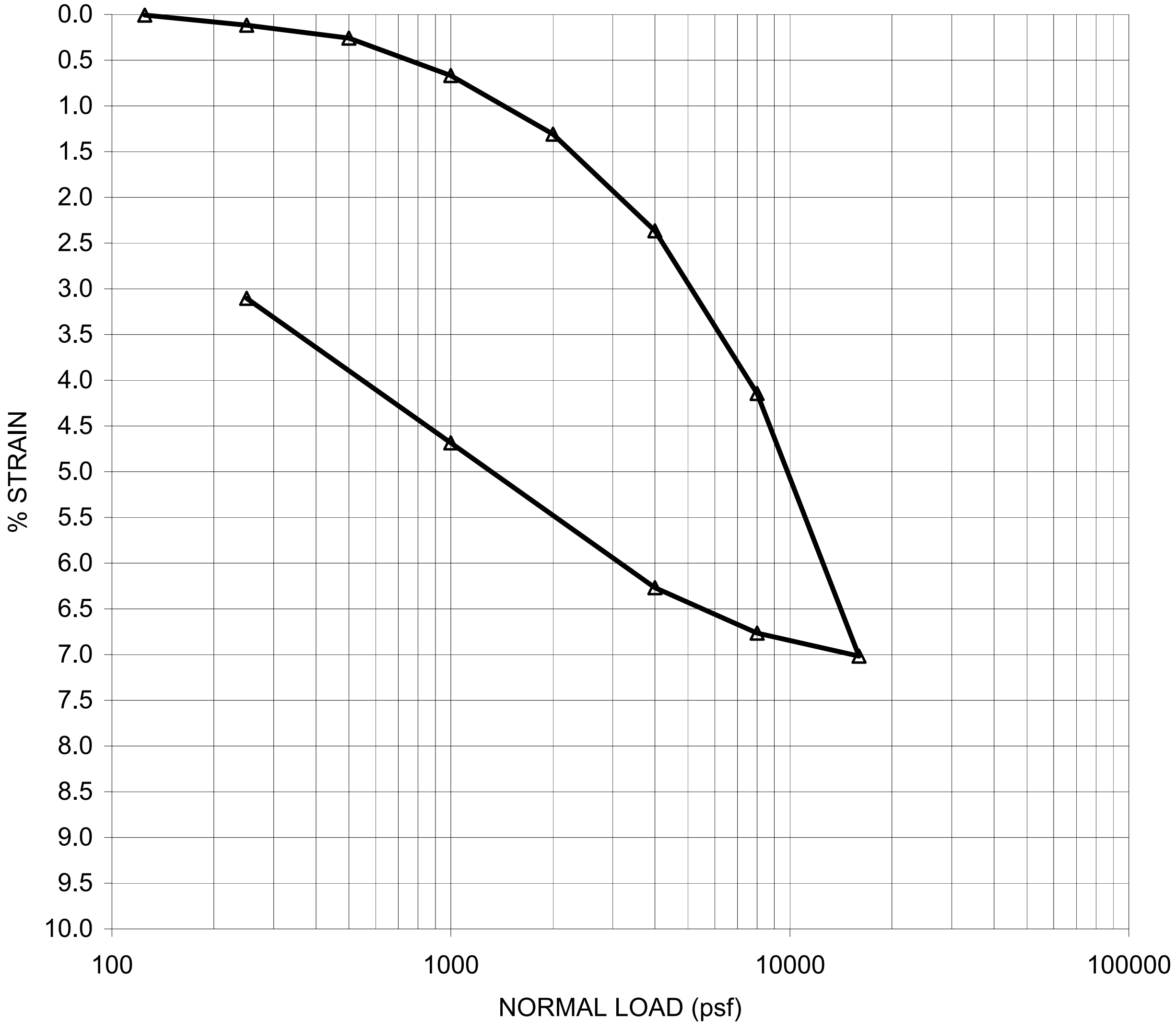
UNCONFINED COMPRESSIVE STRENGTH RESULTS

515 Soquel Avenue

FIGURE

B-3

BORING:	B-8		
DEPTH (ft):	8	━━━━━━	SATURATED
SOIL TYPE (USCS):	CH	FIELD MOISTURE:	28.5%
		FINAL MOISTURE:	32.0%



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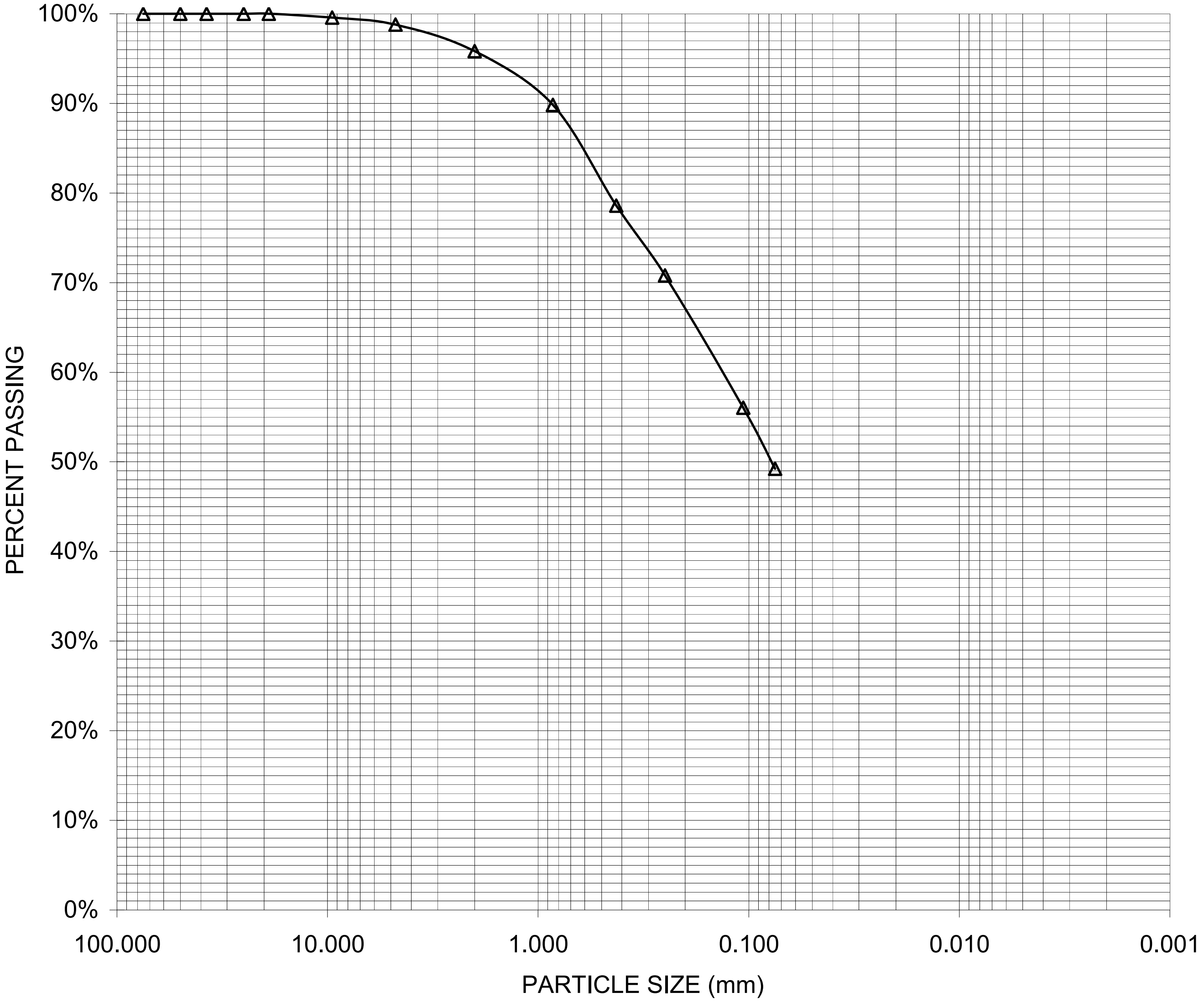
CONSOLIDATION TEST RESULTS

515 Soquel Avenue

FIGURE

B-4

BORING:	B-2	PERCENT	PERCENT
DEPTH (ft):	12	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	SC	98.8%	49.2%



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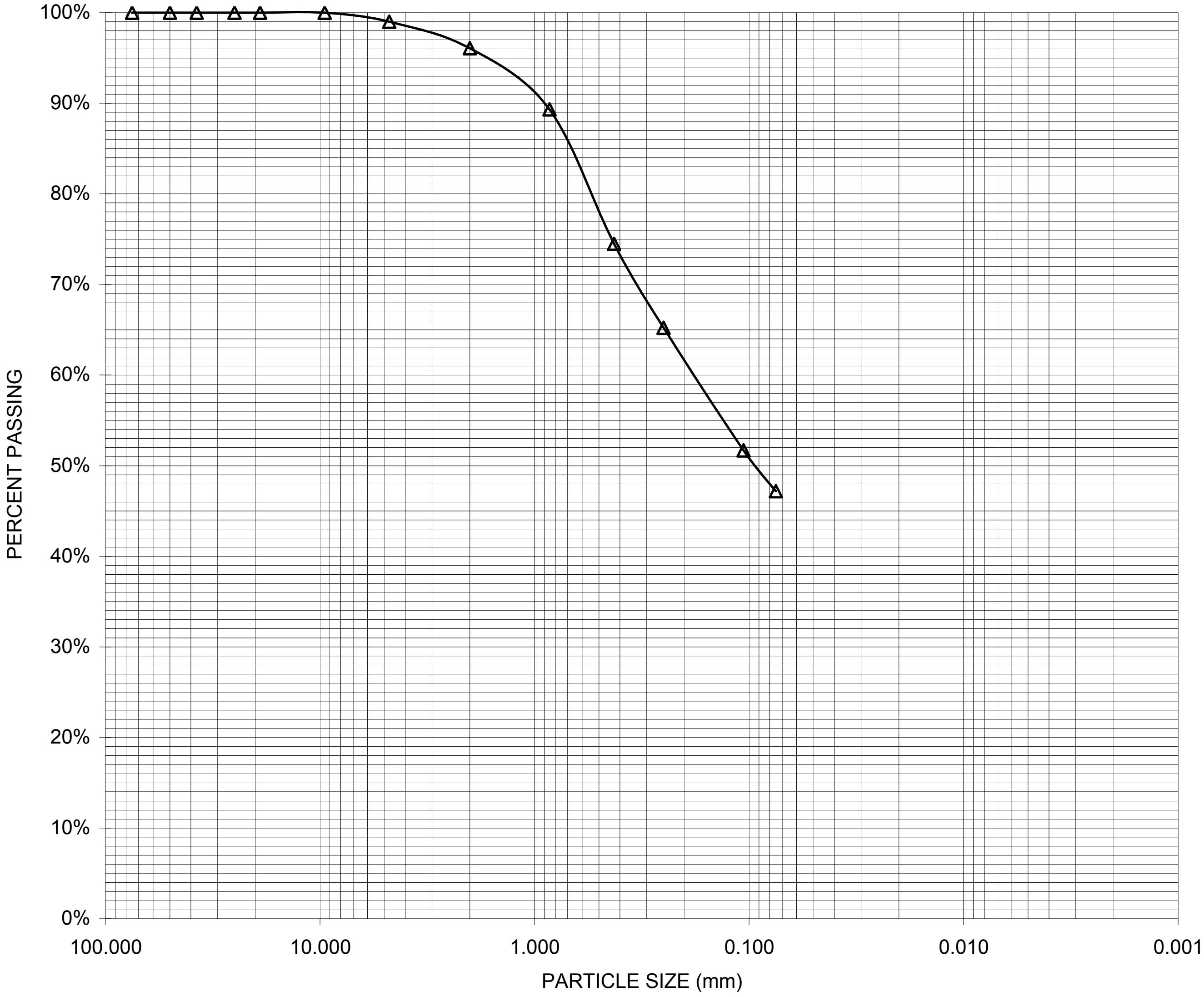
PARTICLE SIZE DISTRIBUTION

515 Soquel Avenue

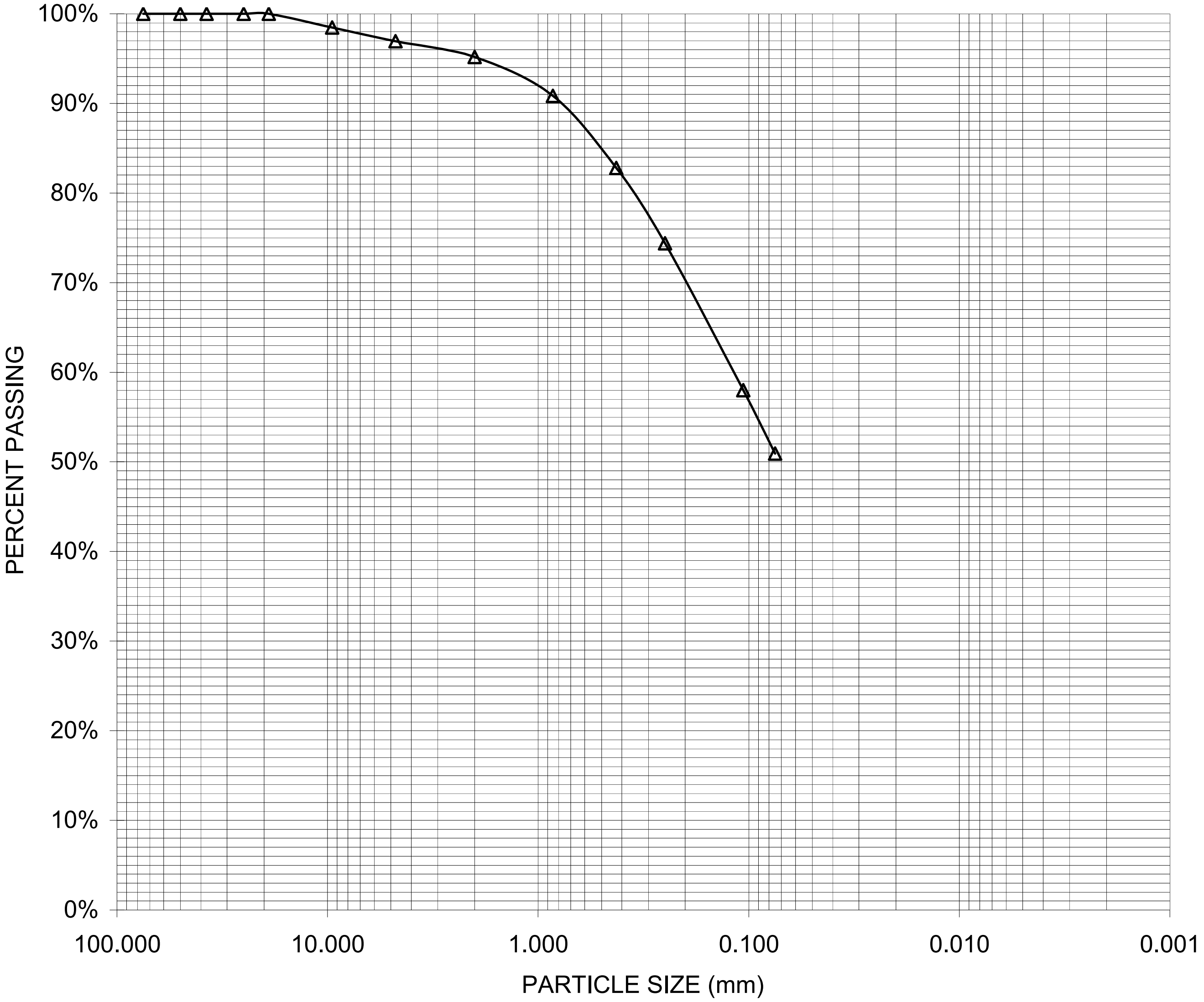
FIGURE B-5

BORING:	B-2	PERCENT	PERCENT
DEPTH (ft):	18	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	SC	99.0%	47.2%



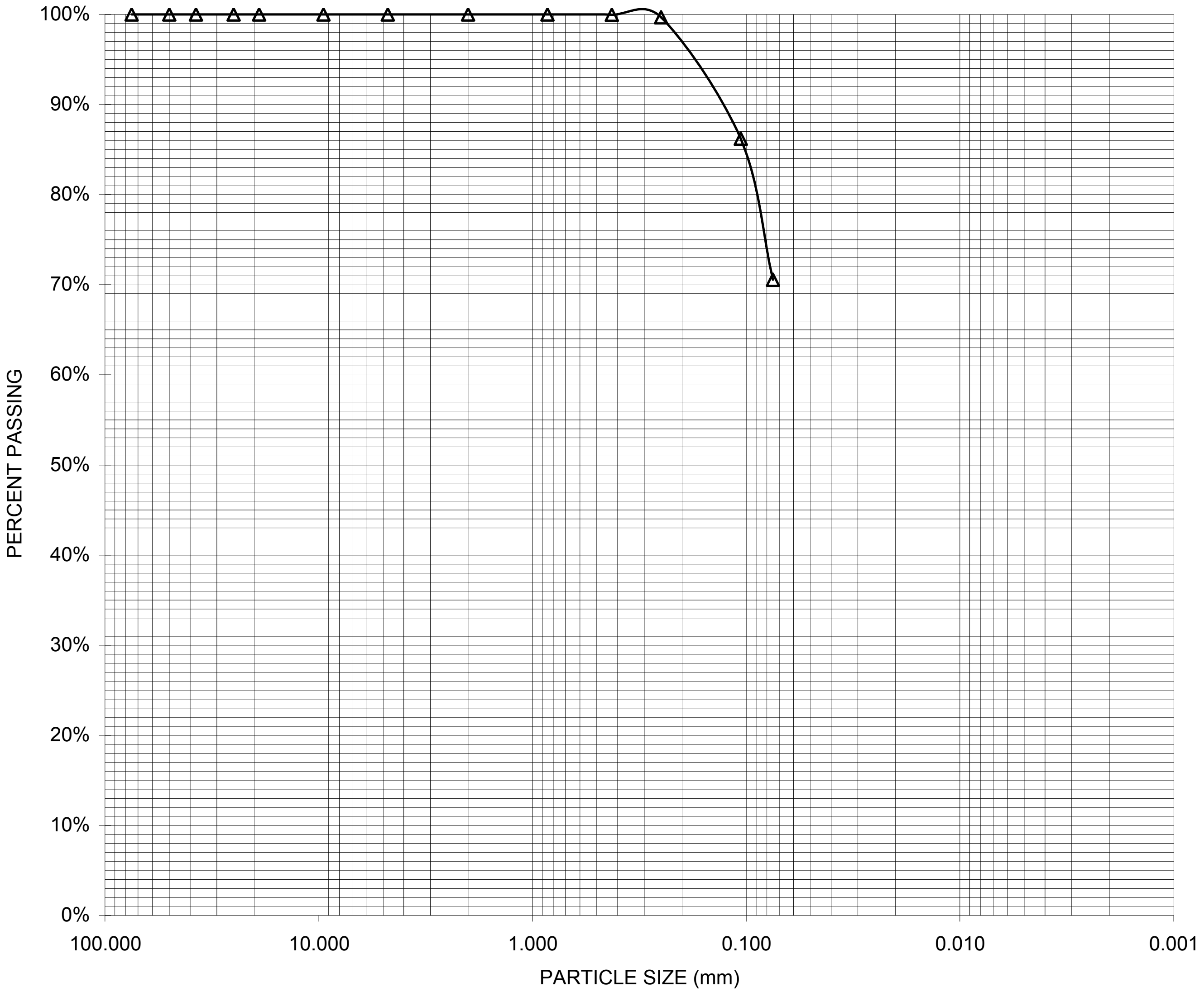
CMAG ENGINEERING	**PARTICLE SIZE DISTRIBUTION**	FIGURE
	515 Soquel Avenue	B-6

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B-7		
DEPTH (ft):	2.5		
SOIL TYPE (USCS):	SC - CL	97.0%	50.9%



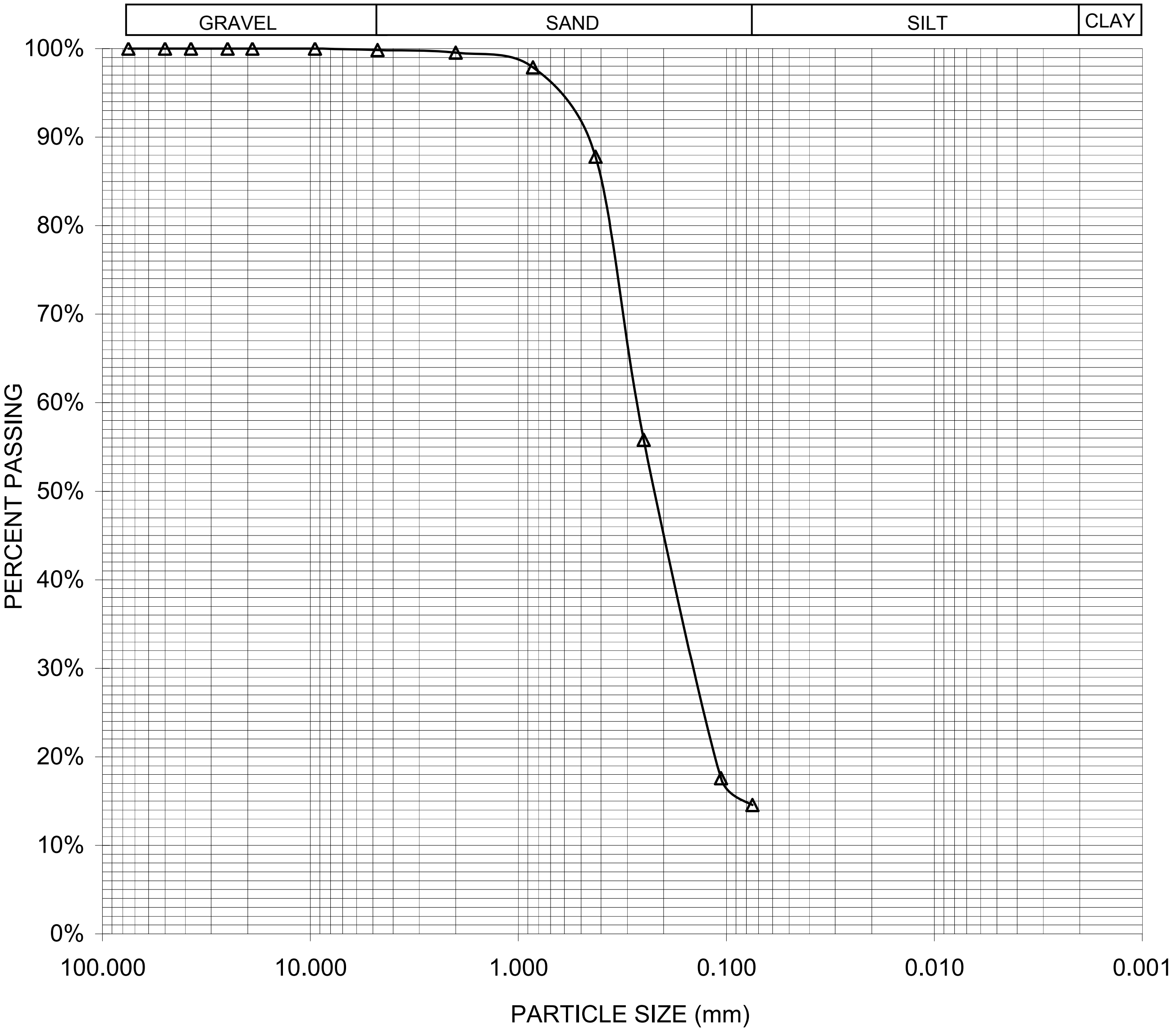
CMAG ENGINEERING	PARTICLE SIZE DISTRIBUTION	FIGURE
	515 Soquel Avenue	B-7

		PERCENT PASSING No. 4	PERCENT PASSING No. 200
BORING:	B-7		
DEPTH (ft):	7		
SOIL TYPE (USCS):	ML	100.0%	70.6%



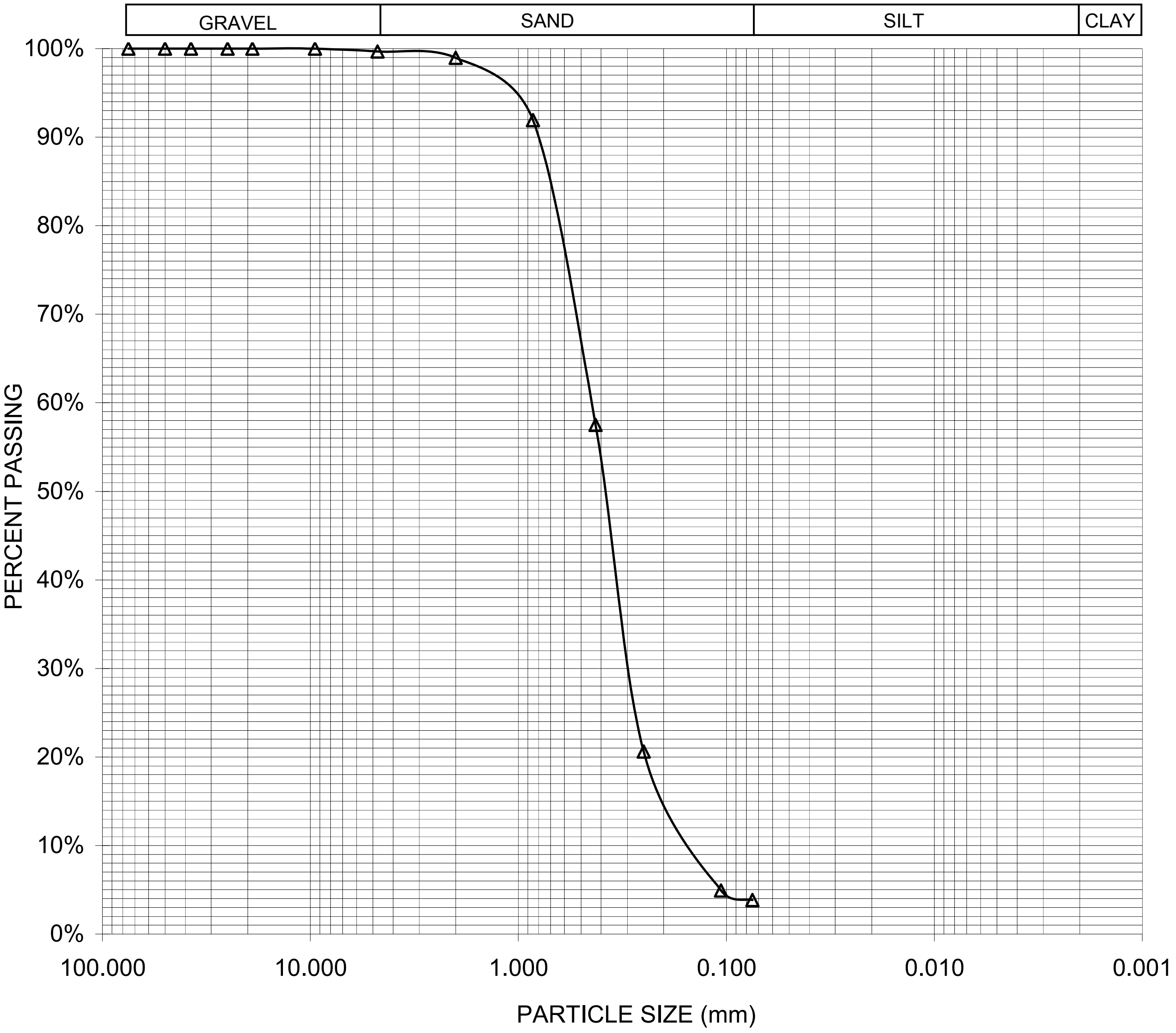
CMAG ENGINEERING	**PARTICLE SIZE DISTRIBUTION**	FIGURE
	515 Soquel Avenue	B-8

BORING:	B-8	PERCENT	PERCENT
DEPTH (ft):	14.5	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	SM	99.8%	14.6%



CMAG ENGINEERING	**PARTICLE SIZE DISTRIBUTION**	FIGURE
	515 Soquel Avenue	B-9

BORING:	B-8	PERCENT	PERCENT
DEPTH (ft):	18	PASSING No. 4	PASSING No. 200
SOIL TYPE (USCS):	SP	99.7%	3.8%



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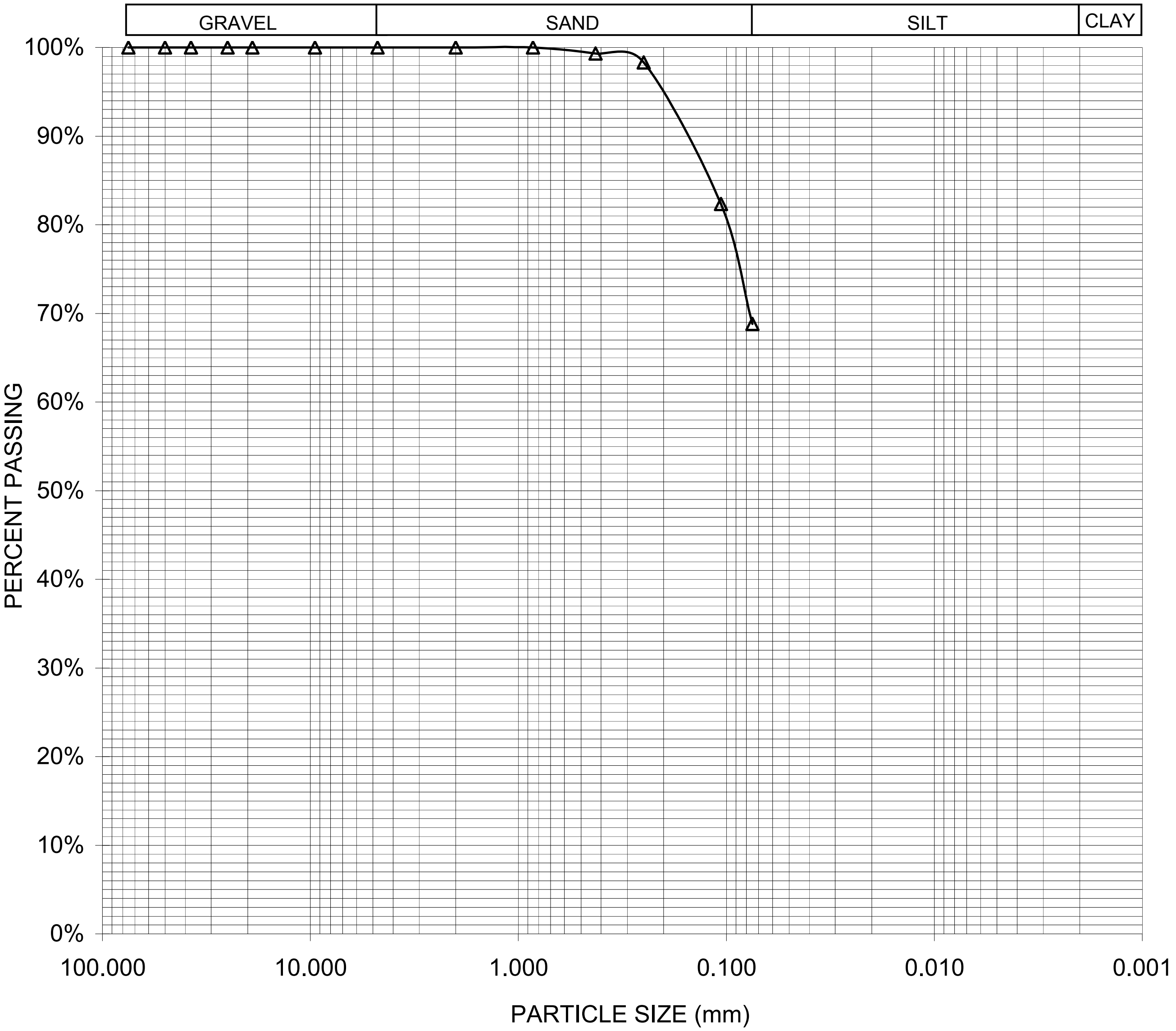
PARTICLE SIZE DISTRIBUTION

515 Soquel Avenue

FIGURE

B-10

BORING:	B-8	PERCENT PASSING No. 4	PERCENT PASSING No. 200
DEPTH (ft):	19		
SOIL TYPE (USCS):	ML	100.0%	68.8%



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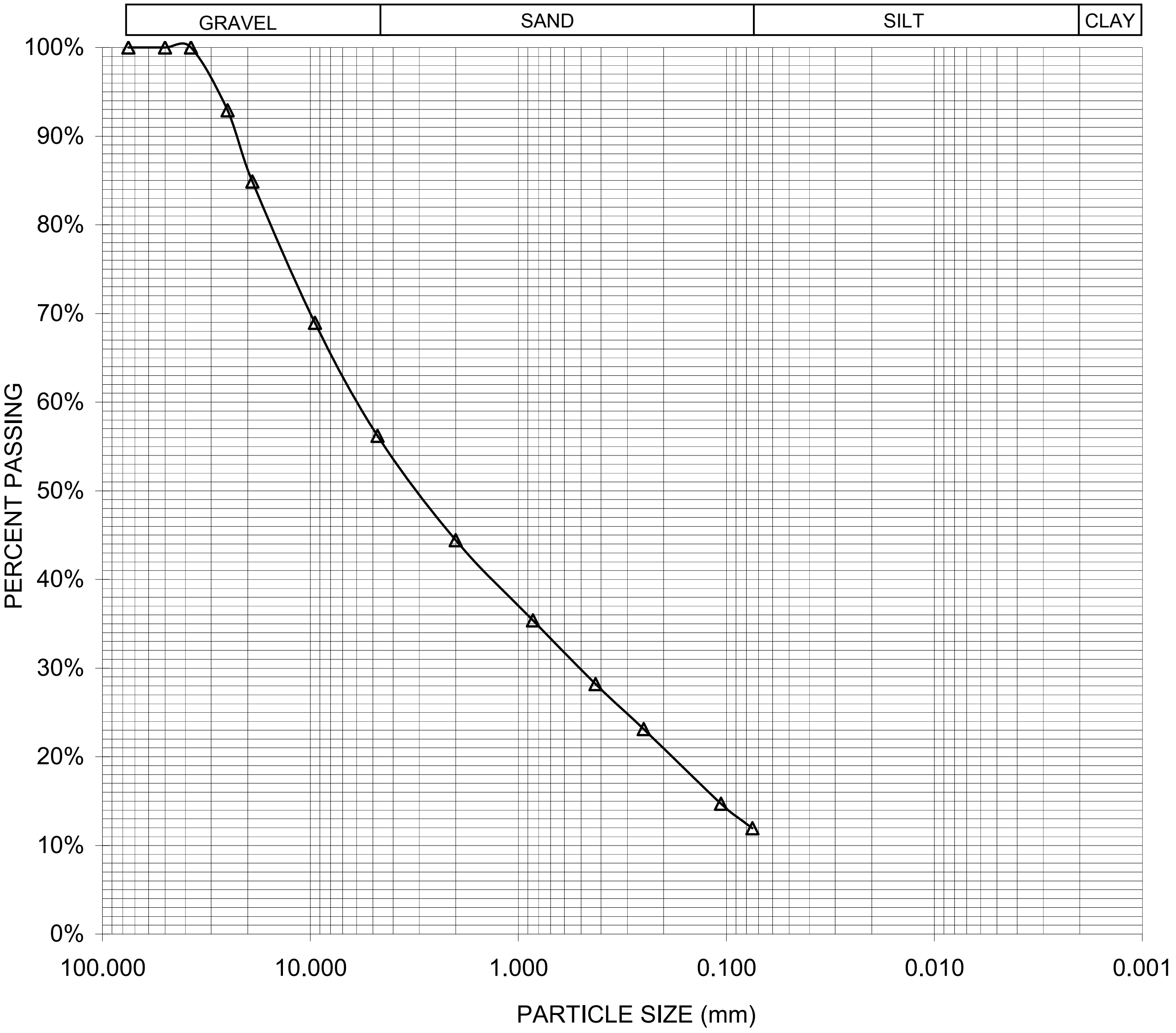
PARTICLE SIZE DISTRIBUTION

515 Soquel Avenue

FIGURE

B-11

BORING:	B-8	PERCENT PASSING No. 4	PERCENT PASSING No. 200
DEPTH (ft):	23		
SOIL TYPE (USCS):	SM - GM	56.2%	11.9%



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PARTICLE SIZE DISTRIBUTION

515 Soquel Avenue

FIGURE

B-12