COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT
701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123
KATHLEEN MOLLOY, PLANNING DIRECTOR

8 January 2019

Boos Development West, LLC
2020 L Street, Suite 245
Sacramento, CA 95811

Subject: Review of the Geotechnical Engineering Investigation – Proposed CVS Pharmacy at 1505/1515 Commercial Way dated 15 January 2018 by Moore Twining Associates – Project No. G10838.03

Project Site: 1505/1515 Commercial Way
APN 025-071-20
Application No. B-181177

Dear Applicant:

The purpose of this letter is to inform you the Planning Department has accepted the subject report. The following items shall be required:

1. All project design and construction shall comply with the recommendations of the report.

2. Final plans shall reference the subject report by title, author and date. Final Plans should also include a statement that the project shall conform to the report's recommendations.

3. After plans are prepared that are acceptable to all reviewing agencies, please submit a completed Soils (Geotechnical) Engineer Plan Review Form to Environmental Planning. The author of the soils report shall sign and stamp the completed form. Please note that the plan review form must reference the final plan set by last revision date.

Electronic copies of all forms required to be completed by the Geotechnical Engineer may be found on our website: www.sccoplanning.com, under "Environmental", "Geology & Soils", and "Assistance & Forms".

After building permit issuance the soils engineer *must remain involved with the project* during construction. Please review the Notice to Permits Holders (attached).

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Please note that this determination may be appealed within 14 calendar days of the date of service. Additional information regarding the appeals process may be found online at: http://www.sccoplanning.com/html/devrev/plnappeal_bldg.htm

Please contact the undersigned at (831) 454-3168 or rick.parks@santacruzcounty.us if we can be of any further assistance.

Sincerely,

Rick Parks, GE 2603
Civil Engineer – Environmental Planning

Cc: Environmental Planning, Attn: Leah MacCarter
Planning Department, Attn: Annette Olson
Owner, Plymouth-Grant LLC
MTA, Attn: Read Andersen, GE

Attachments: Notice to Permit Holders

NOTICE TO PERMIT HOLDERS WHEN A SOILS REPORT HAS BEEN PREPARED, REVIEWED AND ACCEPTED FOR THE PROJECT

After issuance of the building permit, the County requires your soils engineer to be involved during construction. Several letters or reports are required to be submitted to the County at various times during construction. They are as follows:

1. **When a project has engineered fills and / or grading**, a letter from your soils engineer must be submitted to the Environmental Planning section of the Planning Department prior to foundations being excavated. This letter must state that the grading has been completed in conformance with the recommendations of the soils report. Compaction reports or a summary thereof must be submitted.

2. **Prior to placing concrete for foundations**, a letter from the soils engineer must be submitted to the building inspector and to Environmental Planning stating that the soils engineer has observed the foundation excavation and that it meets the recommendations of the soils report.

3. **At the completion of construction,** a *Soils (Geotechnical) Engineer Final Inspection Form* from your soils engineer is required to be submitted to Environmental Planning that includes copies of all observations and the tests the soils engineer has made during construction and is stamped and signed, certifying that the project was constructed in conformance with the recommendations of the soils report.

   If the *Final Inspection Form* identifies any portions of the project that were not observed by the soils engineer, you may be required to perform destructive testing in order for your permit to obtain a final inspection. The soils engineer then must complete and initial an *Exceptions Addendum Form* that certifies that the features not observed will not pose a life safety risk to occupants.

MOORE TWINING ASSOCIATES, INC.

GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED CVS PHARMACY

1505 and 1515 COMMERCIAL WAY

SANTA CRUZ, CALIFORNIA

Project Number: G10838.03

For:

Boos Development West, LLC
701 Park Center Drive, Suite 200
Santa Ana, CA 92705

January 15, 2018

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MOORE TWINING
ASSOCIATES, INC.

January 15, 2018 G10825.01

Ms. Leanna Swenson
Boos Development West, LLC
2020 L Street, Suite 245
Sacramento, CA 95811

Subject: **Geotechnical Engineering Investigation**
Proposed CVS Pharmacy
1505 and 1515 Commercial Way
Santa Cruz, California

Dear Ms. Swenson:

We are pleased to submit this geotechnical engineering investigation report prepared for the proposed CVS Pharmacy to be located at the subject property.

The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations. It is recommended that those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by Moore Twining Associates, Inc. (Moore Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement; however, the client should provide these documents for our review prior to their issuance for construction bidding purposes.

In addition, it is recommended that Moore Twining be retained to provide inspection and testing services for the excavation, earthwork, pavement, and foundation phases of construction. These services are necessary to determine if the subsurface conditions are consistent with those used in the analyses and formulation of recommendations for this investigation, and if the construction complies with our recommendations. These services are not, however, part of this current contractual agreement. A representative with our firm will contact you in the near future regarding these services.

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We appreciate the opportunity to be of service to Boos Development West, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Allen H. Harker

Allen H. Harker
Professional Geologist

EXECUTIVE SUMMARY

This report presents the results of a geotechnical engineering investigation for a CVS Pharmacy building to be located at 1505 and 1515 Commercial Way in Santa Cruz, California.

The proposed CVS Pharmacy store will be approximately 13,111 square feet and have a second floor mezzanine that will occupy 1,712 square feet. Appurtenant construction is anticipated to include various underground utility service lines, an asphalt concrete paved parking lot, a concrete paved trash enclosure, retaining walls along the eastern and western property boundaries, a monument sign, a transformer and landscaped areas.

At the time of our field exploration, the site was occupied by a vacant lot, two commercial buildings with associated parking, and an alley. The eastern portion of the site was occupied by a Decor retail furniture building (1515 Commercial Way) and the southwestern portion of the site was occupied by a building used for storage of furniture (1505 Commercial Way).

On December 13 through 15, 2017, ten (10) borings were drilled at the subject site. The near surface soils encountered in the borings conducted for this investigation generally consisted of clayey sands extending to depths of about 2 to 10 feet BSG or lean clays, lean clays with sand or sandy lean clays that extended to depths of about 2½ to 8½ feet BSG. The near surface clayey sands were underlain by lean clays, clayey sands, silty sands extending to the maximum depth explored, about 26½ feet BSG. The near surface lean clays, lean clays with sand or sandy lean clays were underlain by silty sands and poorly graded sands extending to the maximum depth explored, about 50 feet BSG. The silty sands were generally dense to very dense below a depth of about 20 feet BSG. One of the near surface soil samples encountered exhibited weak cementation where silty sands were encountered at a depth of about 2½ feet.

Fil soils were encountered in boring B-2 drilled in the alle on the east side of the site (northeast corner of the proposed CVS Pharmacy). The fill soils consisted of loose clayey sands with brick debris and asphalt debris extending to a depth of 5 feet BSG. Fill soils are anticipated in other portions of the site due to prior site grading.

Groundwater was encountered in some of the borings during our December 2017 field exploration. Groundwater was generally encountered during drilling at depths ranging from about 14½ feet to 23¾ feet BSG. About ½ hour to 1 hour after completion of the borings that encountered groundwater, groundwater stabilized at depths ranging from about 16½ to 23¾ feet BSG. It should be noted that perched water was encountered at a depth of 4½ feet BSG in boring B-2 near the bottom of the clayey sand fill soils and top of the native clay soils encountered in this borehole.

Based on our field and laboratory investigation, the near surface soils tested possess a medium expansion potential and high compressibility characteristics.

In order to reduce the potential for excessive static settlement, over-excavation of the existing fill soils and near surface native soils is recommended to support new foundations on engineered fill. In addition, over-excavation will be required to remove soils disturbed from removal of surface and subsurface improvements and all fill soils that are encountered.

EXECUTIVE SUMMARY (Continued)

Seismic settlements of about ⅔ inch total and ½ inch differential in 40 feet were estimated.

The potential for surface fault rupture at the site is considered low.

Chemical testing of the near surface soil samples indicated the soils exhibit a "highly corrosive"corrosion potential. Chemical analyses also indicated a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

This Executive Summary should not be used for design or construction and should be reviewed in conjunction with the attached report.

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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED CVS PHARMACY

1505 and 1515 COMMERCIAL WAY

SANTA CRUZ, CALIFORNIA

Project Number: G10838.03

1.0 INTRODUCTION

This report presents the results of a geotechnical engineering investigation for the proposed CVS Pharmacy and associated site improvements to be located at 1505 and 1515 Commercial Way in Santa Cruz, California. Moore Twining Associates, Inc. (Moore Twining) was authorized by Boos Development West, LLC to perform this geotechnical engineering investigation.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, site description, and anticipated construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The report appendices contain the drawings (Appendix A), the logs of borings (Appendix B), and the results of laboratory tests (Appendix C).

The Geotechnical Engineering Division of Moore Twining, headquartered in Fresno, California, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The intent of this investigation is to satisfy the requirements of the 2016 California Building Code (CBC), and the Boos Development West (BDW) Geotechnical Investigation Requirements, as related to geotechnical investigations. The purpose of the investigation was to conduct an exploration program, evaluate the data collected during the field investigation and laboratory testing, and provide geotechnical engineering recommendations for project design.

2.1.1 Evaluation of the near surface soils within the zone of influence of the proposed foundations with regard to the anticipated foundation loads;

2.1.2 Recommendations for 2016 California Building Code seismic coefficients and earthquake spectral response acceleration values;

2.1.3 Geotechnical parameters for use in design of foundations and slabs-on-grade, (e.g., soil bearing capacity, settlement, lateral resistance);

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2.1.4 Recommendations for site preparation including placement, moisture conditioning, and compaction of engineered fill soils;

2.1.5 Recommendations for temporary excavations, trench excavation, and trench backfill;

2.1.6 Evaluation of the potential for liquefaction and seismic settlement;

2.1.7 Recommendations for slab-on-grade floors and exterior concrete flatwork;

2.1.8 Recommendations for asphalt concrete and Portland cement concrete pavements; and

2.1.9 Conclusions regarding soil corrosion potential.

This report is provided specifically for the proposed improvements described in the Anticipated Construction section of this report. This investigation did not include a geologic/seismic hazards evaluation, flood plain investigation, compaction tests, percolation tests, environmental investigation, or environmental audit.

2.2 <u>**Scope**</u>**:** Our proposal, reference MTP 4417-1266, dated December 7, 2017, outlined the scope of our services. The actions undertaken during the investigation are summarized as follows.

2.2.1 A Site Plan (SK-1), dated March 29, 2017, prepared by Kimley-Horn and Associates, Inc., was reviewed. The plan was used for preparation of the Test Boring Location Map (Drawing No. 2 in Appendix A) and is referred to herein as the site plan.

2.2.2 The BDW Geotechnical Investigation Requirements included in our Agreement for Geotech Consultant Services, dated January 9, 2013, was reviewed.

2.2.3 A visual site reconnaissance and subsurface exploration were conducted.

2.2.4 Various satellite images of the site from 1993 to 2016 from online sources, were reviewed. In addition, various aerial photographs from 1931 to 2012 were reviewed from Environmental Data Resources, Inc. (EDR).

2.2.5 A report entitled, "Draft Phase I Environmental Site Assessment, Proposed CVS Pharmacy CS No. 105634, 1505 & 1515 Commercial Way, Santa Cruz, California,", dated December 18, 2017, prepared by Moore Twining's Environmental Division, was reviewed.

2.2.6 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils encountered.

2.2.7 Ms. Leanna Swenson (Boos Development West, LLC) and Ms. Melissa Mahar (Appenrodt Commercial) were consulted prior to the investigation.

2.2.8 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and the engineering properties of the soils encountered.

2.2.9 This report was prepared to present the purpose and scope, background information, field exploration procedures, findings, evaluation, conclusions, and recommendations.

3.0 BACKGROUND INFORMATION

The site description, site history, previous studies, and the anticipated construction are summarized in the following subsections.

3.1 Site Description: The subject site is addressed as 1505 and 1515 Commercial Way in Santa Cruz, California. According to the site plan, the site comprises approximately 1.23 acres. The site is bounded by Soquel Drive to the northwest, by an existing gas station to the west, by Commercial Way to the south, and by existing commercial development to the east.

At the time of our field exploration, the site was occupied by a vacant lot which had been partially excavated, two commercial buildings with associated parking, and an alley between Commercial Way and Soquel Drive. The eastern portion of the site was occupied by a Decor Furniture building (1515 Commercial Way) and the southwestern portion of the site was occupied by a building used for storage of furniture (1505 Commercial Way). A depressed loading dock was noted near the northeast corner of the Decor Furniture building. Some bushes and a tree were noted around the exterior of the Decor Furniture building and some bushes were noted near the driveway from Soquel Drive.

Based on our review of the "Draft Phase I Environmental Site Assessment, Proposed CVS Pharmacy CS No. 105634, 1505 & 1515 Commercial Way, Santa Cruz, California,", dated December 18, 2017, prepared by Moore Twining's Environmental Division: "The building located in the southwestern portion of the site comprised approximately 2,480 square feet in plan dimension and was a slab-on-grade, corrugated metal constructed warehouse structure. The building included a second story, wood constructed loft in the southwest corner of the building's interior. The building located in the eastern portion of the site comprised approximately 13,150 square feet in plan dimension and was a slab-on-grade masonry constructed commercial building. A mezzanine and covered wooden deck area were located along the western interior wall and a wooden loft was located along the northern interior wall."

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Several of the utilities for the commercial building on the eastern side of the site were noted along the western side of the building, above ground, along wood supports. Other utilities at the site were noted to be underground and their locations were painted on the ground surface. Overhead utilities were also noted on the south side of the site.

A vacant, unpaved lot was located in the northwestern portion of the site and had been recently excavated as part of recent environmental related remediation activities that removed some of the near surface soils to depths of about 2 to 3 feet below site grade (this is discussed further in the Site History and Previous Studies section of this report). The site grades in the area of the vacant lot were uneven as different areas were excavated to different depths. In addition, a broken clay pipe was noted to daylight in a vertical sidewall of an excavation which was conducted as part of soil remediation. Scattered grasses and weeds were noted on small portions of the vacant lot. The western portion of the vacant lot was covered by gravel/base material. A small diameter black drain pipe was noted protruding from the ground surface in the southern portion of the vacant lot. Some erosional gullies about 6 to 12 inches deep were noted in the western-central portion of the vacant lot.

The site is surrounded by a concrete masonry retaining wall and chain link fence on the west side of the site, a chain link fence on the north side of the vacant lot, landscaped areas and an asphalt concrete paved driveway on the north side of the Decor Furniture building, a masonry screen wall and an auto center building on the east side of the site, an asphalt concrete paved driveway on the south side of the site. In the northern half of the property, a masonry screen wall and chain link fence separates the Decor Furniture property (eastern side of the site) from the vacant lot in the western portion of the site.

The concrete masonry unit wall on the west side of the Decor Furniture building exhibited a significant crack which was offset by about 1 inch.

The asphalt concrete pavements on the south side of the Decor Furniture building (eastern side of the site), adjacent to Commercial Way, exhibited severe alligator cracking and complete deterioration of the pavement in some areas, exposing the subgrade soils. The asphalt concrete pavements on the south side of the of the smaller building in the western portion of the site generally did not exhibit any significant cracks except in the southeastern portion of the driveway entrance which exhibited alligator cracking and complete deterioration of the pavement areas in some areas, exposing the subgrade soils. The asphalt concrete pavements on the north side of the Decor Furniture building, adjacent to Soquel Drive, appeared to have been patched in some areas and exhibited alligator cracking in several areas, including previously patched areas.

Based on our review of a satellite image of the site, the elevation of the site appears to slope gently down to the west. Site grades appear to range from about 100 feet above mean sea level (AMSL) in the western part of the site to about 108 feet AMSL in the eastern part of the site. However, due to the excavation of about 2 to 3 feet of soil throughout the vacant dirt lot area on the west side of the site, some of the existing elevations may now be as low as about 97 to 98 feet AMSL.

3.2 **<u>Previous Studies and Site History</u>:** Site history information is noted from a report entitled, Draft Phase I Environmental Site Assessment, Proposed CVS Pharmacy CS No. 105634, 1505 & 1515 Commercial Way, Santa Cruz, California,", dated December 18, 2017, prepared by Moore Twining's Environmental Division.

Aerial photographs were reviewed from 1931, 1940, 1943, 1956, 1968, 1974, 1982, 1993, 2005, 2009, 2010 and 2012. Based on our review of the 1931 through 1943 aerial photographs, it is difficult to tell if any structures existed at the site, but it appears that the site was vacant land until at least 1943. Based on our review of Moore Twining's Phase I Environmental Site Assessment (ESA) report, the report indicated: "The site was developed with commercial buildings sometime around 1956. The western portion of the site was occupied by an auto wrecking yard from at least 1968 until sometime around 1992. The western portion of the site was then occupied by an equipment storage yard for Lewis Plaster Company until sometime around 2014. The eastern portion of the site was occupied by a rehabilitation facility and child development facility from 1977 until sometime in the early 1990s. The use of this portion of the site prior to 1977 was not identified. The eastern portion of the site has been occupied by a furniture store since the mid-1990s."

No previous geotechnical engineering, geological, or compaction test reports conducted for this site were provided for review. If these reports become available, the reports should be provided for review and consideration for this project.

3.3 **<u>Anticipated Construction</u>:** A CVS Pharmacy is planned in the southeastern portion of the site. The CVS Pharmacy building will be approximately 13,111 square feet and will include a mezzanine that will occupy 1,712 square feet. Driveway entrances are shown on the site plan to be planned from both Soquel Drive on the north side of the site and Commercial Way on the south side of the site. Appurtenant construction is anticipated to include various underground utility service lines, an asphalt concrete paved parking lot, a concrete paved trash enclosure, retaining walls along the eastern and western property boundaries, a monument sign, a transformer and landscaped areas.

It is anticipated that the proposed CVS building construction will consist of a single-story structure with a mezzanine and with concrete slab-on-grade floors and exterior CMU walls with interior steel columns and a steel-frame supported roof system. According to the CVS Geo-technical Investigation Requirements, a minimum soil bearing pressure of 2,000 pounds per square foot is required for the foundation design and the following structure loads are anticipated: interior column loads of 120 kips, exterior column loads of 100 kips, load bearing wall loading of 3.5 kips per lineal foot and a floor slab loading of 150 pounds per square foot.

The site plan indicates that the area of the CVS Pharmacy has an existing elevation of -2.00± and a proposed elevation of 3.00±. Thus, we assume that about 5 feet of fill is planned to achieve the finished floor elevation. However, the current site grades vary across the site. Thus, it is anticipated the depth of fill will vary.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing programs conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, conducting standard penetration tests, and soil sampling.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Barry Smith of Moore Twining during the field exploration that took place between December 13 and 15, 2017. The features noted are described in the background information section of this report.

4.1.2 Drilling Test Borings: Based on the presence of the existing buildings at the site, the investigation was planned so as to drill all the borings outside the existing buildings. Thus, some of the borings had to be moved slightly due to the presence of the existing buildings. The locations of the test borings were agreed upon with Ms. Leanna Swenson (Boos Development West, LLC) and considered the location of the existing structures size of the proposed structure, and type of construction. The depths of the borings were conducted in accordance with the BDW Geotechnical Investigation Requirements, dated January 9, 2013 and considered the estimated depth of influence of the anticipated foundation loads, and the subsurface soil conditions encountered.

On December 13 through 15, 2017, ten (10) borings were drilled at the subject site. Five (5) borings (B-2, B-4, B-5, B-8 and B-9) were drilled in the proposed building footprint to depths of about 25 to 48⅔ feet below site grade (BSG) and a sixth boring was drilled using a hand auger due to access restrictions for the drill rig to a depth of about 6½ feet BSG. Two (2) borings (B-1 and B-7) were drilled about 50 feet north and west of the proposed CVS Pharmacy building to depths of about 10 to 13 feet BSG. Two (2) borings (B-3 and B-6) were drilled in the areas of the proposed driveway entrances near Soquel Drive and Commercial Way to depths of about 10 to 11½ feet BSG.

The test borings were drilled using a truck-mounted CME-75 drill rig equipped with 6-⅝ inch outside diameter (O.D.) hollow-stem augers. Boring B-10 was advanced with a hand auger equipped with a 4-inch diameter auger.

During the drilling of the test borings, bulk and relatively undisturbed samples of soil were obtained for laboratory testing. The test borings were drilled under the direction of a Moore Twining staff engineer. The soils encountered in the test borings were logged during drilling by a representative of our firm. The field soil classification was in accordance with the Unified Soil Classification System consisted of particle size, color, and other distinguishing features of the soil.

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The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and up to an hour following completion of the borings.

Test boring locations were determined with reference to the existing site features shown on the site plan. The borings were backfilled with cuttings and topped with rapid setting concrete where borings were drilled in the Portland cement concrete pavement areas. The approximate locations of the borings are shown on Drawing No. 2 in Appendix A of this report.

4.1.3 Soil Sampling: Standard penetration tests were conducted in the test borings, and both disturbed and relatively undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜-inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil with the drill rig or with hand sampling equipment. The soil was retained in brass rings, 2.5 inches O.D. and 1-inch in height. The lower 6-inch portion of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory. Soil samples obtained were taken to Moore Twining's laboratory for classification and testing.

4.2 Laboratory Testing: The laboratory testing was programmed to determine selected physical and engineering properties of selected samples of the soils obtained during drilling. The tests were conducted on disturbed and relatively undisturbed samples considered representative of the subsurface soils encountered.

The results of laboratory tests are summarized in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 Surface Conditions: At the time of our field exploration, the site was occupied by two buildings, asphalt and concrete pavements, and landscaped areas. Various underground utilities are located throughout the site. Additional information regarding the existing site conditions is noted in the Background Information section of this report.

5.2 Portland Cement Concrete Pavements: The Portland cement concrete pavement sections encountered in the borings drilled in Portland cement concrete pavement areas are summarized in Table No. 1 below.

Table No. 1
Portland Cement Concrete Pavement Section Thicknesses Encountered

Boring Number	Portland Cement Concrete Thickness[1] (inches)	Aggregate Base Thickness[2] (inches)	Subgrade Soil
B-1	4.0	6	Clayey Sand
B-2	4.0	4	Clayey Sand Fill
B-3	7.1	2.5	Lean Clay
B-8	4.0	3	Sandy Lean Clay

[1] - Portland cement concrete thickness was measured on four sides to the nearest ¼ inch and averaged to the nearest tenth of an inch.
[2] - Aggregate base thickness was measured to the nearest ½ inch.

5.3 Soil Profile: Based on our review of the Geologic Map of Santa Cruz County, California, dated 1997, prepared by U.S. Geological Survey, the site is mapped as being underlain by Pleistocene-age lowest emergent coastal terrace deposits, which are described as follows: "Semiconsolidated, generally well-sorted sand with a few thin, relatively continuous layers of gravel. Deposited in nearshore high-energy marine environment."

Below the Portland cement concrete pavements or landscaped areas, the near surface soils encountered in the borings conducted for this investigation generally consisted of clayey sands extending to depths of about 2 to 10 feet BSG or lean clays, lean clays with sand or sandy lean clays that extended to depths of about 2½ to 8½ feet BSG. The near surface clayey sands were underlain by lean clays, clayey sands, and silty sands extending to the maximum depth explored, about 26½ feet BSG. The near surface lean clays, lean clays with sand or sandy lean clays were underlain by silty sands and poorly graded sands extending to the maximum depth explored, about 50 feet BSG. The silty sands were generally dense to very dense below a depth of 20 feet BSG. One of the near surface soil samples encountered exhibited weak cementation where silty sands were encountered at a depth of about 2½ feet.

Fil soils were encountered in boring B-2 drilled in the alleyway on the east side of the site (northeast corner of the proposed CVS Pharmacy). The fill soils consisted of loose clayey sands with brick debris and asphalt debris extending to a depth of 5 feet BSG.

Fill soils were also noted in the sidewalls of some of the excavated areas. In addition, a clay pipe was exposed in the sidewall of an excavation which was conducted for recent soil remediation.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring are presented in the logs of borings in Appendix B. The stratification lines in the logs represent the approximate boundary soil types; the actual in-situ transition may be gradual.

5.4 **Soil Engineering Properties:** The following is a description of the engineering properties of the soil as determined from our field exploration and laboratory testing.

Clayey Sands Fill Soils: The clayey sand fill soils encountered were described as loose, as determined by a Standard Penetration Test (SPT), N-value, of 5 blows per foot. The moisture content of a sample tested was about 8 percent.

Native Clayey Sands: The native clayey sands encountered were described as very loose to medium dense, as determined by Standard Penetration Test (SPT), N-values, ranging from 2 to 21 blows per foot. The moisture content of the samples tested ranged from about 8 to 16 percent. The results of testing of one (1) relatively undisturbed sample indicated a dry density of 106.1 pounds per cubic foot.

Sandy Lean Clays, Lean Clay with Sand and Lean Clays: The sandy lean clays, lean clays with sand and lean clays encountered were described as soft to very stiff, as determined by Standard Penetration Test (SPT), N-values, ranging from 2 to 18 blows per foot, and as indicated by equivalent Standard Penetration Test (SPT), N-values, ranging from 14 to 19 blows per foot, which were estimated by driving a California Modified split barrel sampler. The moisture content of the samples tested ranged from about 8 to 13 percent. The results of testing of two (2) relatively undisturbed samples both indicated dry densities of 109.2 pounds per cubic foot. A sieve analysis conducted on a sample from boring B-2 encountered from a depth of 10 to 11.5 feet BSG indicated 45.8 percent sand and 54.2 percent fines (silt and clay). An Atterberg Limits test conducted on the same sample indicated a liquid limit of 47 and a plasticity index of 24. A sieve analysis conducted on a sample from boring B-4 encountered from a depth of 1 to 4 feet BSG indicated 48.4 percent sand and 51.6 percent fines (silt and clay). An Atterberg Limits test conducted on the same sample indicated a liquid limit of 49 and a plasticity index of 26. An expansion index test conducted on the same sample indicated an expansion index value of 73. A consolidation test conducted on a sample collected from depths of 2 to 3½ feet BSG from boring B-4 indicated high compressibility characteristics (8.8 percent consolidation under a load of 16 kips per square foot). Another consolidation test conducted on a sample collected from depths of 5 to 6½ feet BSG from boring B-5 indicated high compressibility characteristics (9.1 percent consolidation under a load of 16 kips per square foot). A direct shear test conducted on a sample collected at depths of 2 to 3½ feet BSG from boring B-4 indicated an internal angle of friction of 19 degrees and 270 pounds per square foot of cohesion.

Silty Sands: The silty sands encountered were described as medium dense to very dense, as indicated by Standard Penetration Test (SPT), N-values, ranging from 12 to greater than 50 blows per foot. The moisture content of two samples tested were both about 10 percent. The results of testing of one (1) relatively undisturbed sample and exhibiting weak cementation indicated a dry density of 111.7 pounds per cubic foot. A sieve analysis conducted on a sample collected from depths of about 21½ to 23 feet BSG from boring B-2 indicated 0.3 percent gravel, 71.0 percent sand and 28.7 percent fines (silt and clay). A sieve analysis conducted on a sample collected from depths of about 8½ to 10 feet BSG from boring B-4 indicated 0.5 percent gravel, 68.0 percent sand and 31.5 percent fines (silt and clay). A sieve analysis conducted on a sample collected from depths of about 15 to 16½ feet BSG from boring B-5 indicated 0.4 percent gravel, 67.0 percent sand and 32.6 percent fines (silt and clay). A consolidation test conducted on a sample collected from depths of 5 to 6½ feet BSG from boring B-5 indicated high compressibility characteristics (10.9 percent consolidation under a load of 16 kips per square foot). Upon inundation, the sample exhibited slight swell potential (1.0 percent swell when wetted under a load of 0.5 kips per square foot).

R-value: The result of two (2) R-value tests conducted on near surface samples of lean clay or a mixture of clayey sand and lean clay obtained from borings B-3 and B-6 both indicated a R-value of 20.

Chemical Tests: Chemical tests performed on near surface soil samples collected from boring B-4 and boring B-8 indicated pH values of 6.8 and 6.6; minimum resistivity values of 1,801 and 1,934 ohms-centimeter; 0.0024 and 0.0019 percent by weight concentrations of sulfate; and 0.00091 and 0.0010 percent by weight concentrations of chloride, respectively.

5.5 Groundwater Conditions: Groundwater was encountered in some of the borings during our December 2017 field exploration. Groundwater was generally encountered during drilling at depths ranging from about 14½ feet to 23¾ feet BSG. About ½ hour to 1 hour after completion of the borings that encountered groundwater, groundwater stabilized at depths ranging from about 16½ to 23¾ feet BSG. It should be noted that perched water was encountered at a depth of 4½ feet BSG in boring B-2 near the bottom of the clayey sand fill soils and top of the native clay soils encountered in this borehole.

Based on our review of water well data from the California Department of Water Resources website, a well located about ¼ mile east-southeast of the site indicated that groundwater ranged from about 26 to 32 feet BSG between 1980 and 1982.

A groundwater monitoring report for a site located on the west side of the subject site was reviewed from the California State Water Resources Control Board GeoTracker website. The report entitled, "Third and Fourth Quarter 2012, Groundwater Monitoring Report, 76 Service Station No. 6193, 1500 Soquel Drive, Santa Cruz, California," prepared by Stantec Consulting Services, Inc., dated December 20, 2012, indicated the depth to groundwater from data collected between the years 2010 and 2012 from multiple wells generally ranged from depths of about 15 to 16 feet BSG.

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It should be recognized; however, that groundwater elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for project design and preparation of construction specifications are summarized in the following subsections. The evaluation was based upon the subsurface soil conditions encountered during this investigation and our understanding of the proposed construction. The conclusions obtained from the results of our evaluations are described in the Conclusions section of this report.

6.1 Existing Surface and Subsurface Improvements: At the time of our field exploration, the site included two structures, asphalt concrete and Portland cement concrete pavements, concrete flatwork, underground utilities, and a vacant area (western side of the site) that had been recently excavated to depths of about 2 to 3 feet BSG for soil remediation. Thus, the site grades in the area of the vacant lot were uneven as different areas were excavated to different depths.

As part of the site preparation, the existing surface and subsurface improvements (buildings, foundations, pavements, underground utilities) and associated fill soils will need to be removed. In addition, all soils disturbed from removal of the existing surface and subsurface improvements and all fill soils (anticipated below buildings, pavements, and associated with utility trenches) should be removed during site preparation. During our field exploration, fill soils were noted in boring B-2 that included brick and asphalt debris extending to a depth of about 5 feet BSG. Fill soils are also anticipated in other areas of the site due to grading for prior site development. As part of the site preparation, the existing fill soils will need to be removed and compacted as engineered fill. Excavations resulting from removal of surface and subsurface improvements should be backfilled with engineered fill in accordance with the recommendations of this report.

Asphalt concrete material and other debris generated during demolition at the site should not be incorporated into the soils for use as fill below the building.

In addition, we understand some environmental conditions are present which will require special procedures for excavation and grading. All earthwork activities should be conducted in accordance with the recommendations of the project environmental consultant, applicable documents such as soil management plans and the requirements of the governing agency.

6.2 **<u>Expansive Soils</u>:** In evaluation of the potential for expansive soils, expansion index testing was performed on a representative sample of the near surface soils encountered. The expansion index testing was performed in accordance with ASTM D4829. The sample was tested and classified by expansion potential in accordance with Table 1 of ASTM D4829 and the results are summarized in Appendix C of this report. The results of the expansion index testing indicated the near surface soils are expansive with a medium expansion potential based on an expansion index value of 73. Due to the expansive soils conditions, this report recommends that the interior slab-on-grade and all slabs attached to the building be underlain by at least 6 inches of aggregate base soils over 12 inches of imported, non-expansive granular fill soils. As an alternative, aggregate base may be substituted for the imported, non-expansive granular fill soils.

6.3 **<u>Static Settlement and Bearing Capacity of Shallow Foundations:</u>** The potential for excessive total and differential static settlement of foundations and slabs-on-grade is a geotechnical engineering concern that was evaluated for this project. The increases in effective stress to underlying soils which can occur from new foundations and structures, placement of fill, withdrawal of groundwater, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structures and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Due to the compressibility of the near surface soils, the presence of undocumented fill, and the potential for disturbance of the near surface soils from demolition and removal of the existing improvements, over-excavation and compaction of the near surface soils is recommended to support the new foundations on engineered fill in order to limit the static settlement to 1 inch total and ½ inch differential. Provided the site preparation recommendations of this report are followed, a net allowable soil bearing pressure of 2,500 pounds per square foot, for dead-plus-live loads, may be used for design.

The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill and weight of the footing may be neglected.

A structural engineer experienced in foundation and slab-on-grade design should determine the thickness, reinforcement, design details and concrete specifications for the proposed building foundations and slabs-on-grade based on the anticipated settlements estimated in this report.

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6.4 <u>Seismic Ground Rupture and Design Parameters</u>: The project site is not located in an Alquist-Priolo Earthquake Fault Zone. Based on our review, the San Andreas Fault is the closest active fault to the site. This fault is mapped about 9 miles northeast of the site. The potential for surface fault rupture at the site is considered low.

It is assumed that the 2016 CBC will be used for structural design, and that seismic site coefficients are needed for design.

Based on the 2016 CBC, a Site Class D represents the on-site soil conditions with standard penetration resistance, N-values averaging between 15 and 50 blows per foot in the upper 100 feet below site grade.

A table providing the recommended seismic coefficients and earthquake spectral response acceleration values for the project site is included in the Foundation Recommendations section of this report. A Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M) of 0.500g was determined for the site using the Ground Motion Parameter Calculator provided by the United States Geological Survey (http://earthquake.usgs.gov/designmaps/us/application.php). A Maximum Considered Earthquake magnitude of 7.5 was determined for the site based on deaggregation analysis (United States Geological Survey deaggregation website (https://earthquake.usgs.gov/hazards/interactive/).

6.5 <u>Liquefaction and Seismic Settlement</u>: Liquefaction and seismic settlement are conditions that can occur under seismic shaking from earthquake events. Liquefaction describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movement of the soil mass, combined with loss of bearing, can result. Saturated, loose, granular soils, higher intensity earthquakes, and particularly long duration of ground shaking are the requisite conditions for liquefaction. One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils; however, seismic settlements are typically largest where liquefaction occurs (saturated soils).

Groundwater was encountered as shallow as about 14½ feet BSG during the December 2017 field exploration. Based on the groundwater encountered at the site and our review of other groundwater data (see section 5.5 of this report for discussion), a groundwater depth of 14½ feet BSG was used for our liquefaction analysis.

The analysis was conducted using the computer program LiquefyPro, developed by CivilTech Software. A horizontal ground acceleration of 0.5g, a maximum considered earthquake of 7.5 and a groundwater depth of 14½ feet were used in the analysis of the soils. Soil parameters, such as wet unit weight, N-values and fines content were input from the boring data for the soil layers encountered throughout the depths explored. The analysis was conducted based on the soil conditions encountered in boring B-4 that extended to a depth of 48⅔ feet BSG and B-5 that extended to a depth of 26½ feet BSG.

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Based on our analyses, a medium dense silty sand layer encountered from depths of 15 to 20 feet BSG in boring B-5 is potentially liquefiable. Seismic settlements of about ⅔ inch total and ½ inch differential were estimated.

6.6 **Asphaltic Concrete (AC) Pavements:** Recommendations for asphaltic concrete pavement structural sections are presented in the "Recommendations" section of this report for proposed asphaltic concrete (AC) pavements. The structural sections were designed using the gravel equivalent method in accordance with the California Department of Transportation Highway Design Manual. The analysis was based on traffic index values ranging from 5.0 to 7.0. A traffic index of 6.5 appears to correspond with the CVS criteria of a maximum of 60,000 ESALs. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. If traffic loading is anticipated to be greater than assumed, the pavement sections should be re-evaluated.

It should be noted that if pavements are constructed prior to the construction of the structures, the additional construction truck traffic should be considered in the selection of the traffic index value. If more frequent or heavier traffic is anticipated and higher Traffic Index values are needed, Moore Twining should be contacted to provide additional pavement section designs.

Based on the results of the laboratory testing, an R-value of 20 was used as a basis for the pavement section thickness recommendations.

6.7 **Portland Cement Concrete (PCC) Pavements:** Recommendations for Portland cement concrete (PCC) pavement structural sections are presented in the "Recommendations" section of this report. The PCC pavement sections are based upon the amount and type of traffic loads being considered and the characteristics of the subgrade soils which will support the pavement. The measure of the amount and type of traffic loads are based upon an index of equivalent axle loads (EAL).

In accordance with the CVS Geo-Technical Investigation Requirements, the PCC pavement sections were designed for a life of 20 years, a load safety factor of 1.1, equivalent single axle loading of 18,000 pounds, and a maximum load of 60,000 ESAL. A modulus of subgrade reaction, K-value, for the pavement section, of 160 psi/in was used for the pavement design considering the R-value recommended to be used for design and considering that the pavement will be underlain by a minimum of 6 inches of aggregate base.

6.8 **Soil Corrosion:** The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. Corrosion is a naturally occurring process whereby the surface of a metallic structure is oxidized or reduced to a corrosion product such as iron oxide (i.e., rust). The metallic surface is attacked through the migration of ions and loses its original strength by the thinning of the member.

Soils make up a complex environment for potential metallic corrosion. The corrosion potential of a soil depends on numerous factors including soil resistivity, texture, acidity, field moisture and chemical concentrations. In order to evaluate the potential for corrosion of metallic objects in contact with the onsite soils, chemical testing of soil samples was performed by Moore Twining as part of this report. The test results are included in Appendix C of this report. Conclusions regarding the corrosion potential of the soils tested are included in the Conclusions section of this report based on the National Association of Corrosion Engineers (NACE) corrosion severity ratings listed in the Table No. 2, below.

Table No. 2
Association of Corrosion Engineers (NACE) Corrosion Severity Ratings

Soil Resistivity (ohm cm)	Corrosion Potential Rating
>20,000	Essentially non-corrosive
10,000 - 20,000	Mildly corrosive
5,000 - 10,000	Moderately corrosive
3,000 - 5,000	Corrosive
1,000 - 3,000	Highly corrosive
<1,000	Extremely corrosive

The results of soil sample analyses indicate that the near-surface soils exhibit a "highly corrosive" corrosion potential to buried metal objects. Appropriate corrosion protection should be provided for buried improvements based on the "highly corrosive" corrosion potential of the soils tested. If piping or concrete are placed in contact with imported soils, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Moore Twining does not provide corrosion engineering services.

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6.9 **Sulfate Attack of Concrete:** Degradation of concrete in contact with soils due to sulfate attack involves complex physical and chemical processes. When sulfate attack occurs, these processes can reduce the durability of concrete by altering the chemical and microstructural nature of the cement paste. Sulfate attack is dependent on a variety of conditions including concrete quality, exposure to sulfates in soil/groundwater and environmental factors. The standard practice for geotechnical engineers in evaluation of the soils anticipated to be in contact with concrete is to perform testing to determine the sulfates present in the soils. The test results are then compared with the provisions of ACI 318, section 4.3 to provide guidelines for concrete exposed to sulfate-containing solutions. Common methods used to resist the potential for degradation of concrete due to sulfate attack from soils include, but are not limited to the use of sulfate-resisting cements, air-entrainment and reduced water to cement ratios. The test results are included in Appendix C of this report. Conclusions regarding the sulfate test results are included in the Conclusions section of this report.

7.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the anticipated construction, the following general conclusions are presented.

7.1 The site is considered suitable for the proposed construction with regard to support of the proposed improvements, provided the recommendations contained in this report are followed. It should be noted that the recommended design consultation and observation of clearing, and earthwork activities by Moore Twining are integral to this conclusion.

7.2 Below the Portland cement concrete pavements or landscaped areas, the near surface soils encountered in the borings conducted for this investigation generally consisted of clayey sands extending to depths of about 2 to 10 feet BSG or lean clays, lean clays with sand or sandy lean clays that extended to depths of about 2½ to 8½ feet BSG. The near surface clayey sands were underlain by lean clays, clayey sands, silty sands extending to the maximum depth explored, about 26½ feet BSG. The near surface lean clays, lean clays with sand or sandy lean clays were underlain by silty sands and poorly graded sands extending to the maximum depth explored, about 50 feet BSG. The silty sands were generally dense to very dense below a depth of 20 feet BSG. One of the near surface soil samples encountered exhibited weak cementation where silty sands were encountered at a depth of about 2½ feet.

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Fil soils were encountered in boring B-2 drilled in the alleyway on the east side of the site (northeast corner of the proposed CVS Pharmacy). The fill soils consisted of loose clayey sands with brick debris and asphalt debris extending to a depth of 5 feet BSG. Fill soils are anticipated in other portions of the site due to grading for prior site development.

7.3 Based on our field and laboratory investigation, the near native surface soils tested possess a medium expansion potential and high compressibility characteristics.

7.4 In order to limit the differential static settlement of new foundations to ½ inch, over-excavation and compaction of the near surface soils are recommended to remove and compact the existing undocumented fill soils and near surface native soils to support the proposed foundations on engineered fill.

7.5 Groundwater was encountered in some of the borings during our December 2017 field exploration. Groundwater was generally encountered during drilling at depths ranging from about 14½ feet to 23¾ feet BSG. About ½ hour to 1 hour after completion of the borings that encountered groundwater, groundwater stabilized at depths ranging from about 16½ to 23¾ feet BSG. It should also be noted that perched water was encountered at a depth of 4½ feet BSG in boring B-2 near the bottom of the fill soils and top of the native clay soils encountered in this borehole.

7.6 Based on our liquefaction analyses, a medium dense silty sand layer encountered from depths of 15 to 20 feet BSG from boring B-5 is susceptible to liquefaction. Seismic settlements of about ⅔ inch total and ½ inch differential in 40 feet were estimated.

7.7 Chemical testing of the near surface soil samples indicated the soils exhibit a "highly corrosive" corrosion potential.

7.8 Chemical analyses indicated a "negligible" potential for sulfate attack on concrete placed in contact with the near surface soils.

7.9 The potential for surface fault rupture at the site is considered low.

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8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, the following recommendations are presented for use in the project design and construction. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and construction monitoring by Moore Twining are integral to the proper application of the recommendations. The Contractor is required to comply with the requirements and recommendations presented in this report.

Where the requirements of a governing agency, utility agency or pipe manufacturer differ from the recommendations of this report, the more stringent recommendations should be applied to the project.

8.1 General

8.1.1 The CVS Geo-technical Investigation Requirements indicate maximum column loads of about 120 kips and maximum perimeter wall loads of 3.5 kips per linear foot and a floor slab load of 150 pounds per square foot for a CVS Pharmacy building. When the actual foundation loads are known, this information should be provided to Moore Twining for review to confirm the recommendations for site preparation are appropriate. In the event the foundation loads are different than assumed, the recommendations in this report may need to be revised.

8.1.2 All earthwork activities should be conducted in accordance with the recommendations of the project environmental consultant, applicable documents such as soil management plans and the requirements of the governing agency.

8.1.3 A preconstruction meeting including, as a minimum, the owner, developer, general contractor, earthwork contractor, foundation and paving subcontractors, and Moore Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss project requirements and scheduling.

8.1.4 The Contractor(s) bidding on this project should determine if the information included in the construction documents are sufficient for accurate bid purposes. If the data are not sufficient, the Contractor should conduct, or retain a qualified geotechnical engineer to conduct, supplemental studies and collect information as required to prepare accurate bids.

8.1.5 If wet, unstable soil conditions are experienced, methods such as aeration, mixing wet soils with drier soils, chemical (i.e., lime) treatment of the soil, or over-excavation and placement of a bridge lift of aggregate base and a geotextile stabilization fabric such as Mirafi 600X, may be required to achieve a stable soil condition. The actual method employed to stabilize the bottom of the excavation or pavement subgrade should be selected at the time of construction.

8.1.6 Appropriate construction methods and equipment, such as low vibration equipment, should be used adjacent to the existing improvements so as not to damage existing improvements which are to remain.

8.2 Site Grades and Drainage

8.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations and floor slabs - both during and after construction. Adjacent exterior finished grades should be sloped a minimum of two percent for a distance of at least five feet away from the structure, or as necessary to preclude ponding of water adjacent to foundations, whichever is more stringent. Adjacent exterior grades which are paved should be sloped at least 1 percent away from the foundations for a distance of at least five feet from the building foundations.

8.2.2 It is recommended that landscape planted areas, etc. not be placed adjacent to the building foundations and/or interior slabs-on-grade. Trees should be setback from the proposed structure at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

8.2.3 Landscaping after construction should direct rainfall and irrigation runoff away from the structure and should establish positive drainage of water away from the structure. Care should be taken to maintain a leak-free sprinkler system.

8.2.4 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the aggregate base and reducing the life of the pavements.

8.2.5 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements are recommended.

8.2.6 Rain gutters and roof drains should be provided, and connected directly to the site storm drain system.

8.2.7 Due to the low permeability and expansive nature of the near surface soils, and the shallow groundwater conditions at the site (14½ feet BSG), infiltration of storm water at the site is not recommended for this site.

8.3 Site Preparation

8.3.1 All surface topsoil, vegetation and organics should be removed from all work areas. The general depth of stripping should be sufficiently deep to remove the root systems and organic top soils. The actual depth of stripping should be reviewed by Moore Twining at the time of construction.

8.3.2 The root systems of all trees and bushes to be removed should be removed in their entirety. It is anticipated that roots, root balls, and loose soils and voids resulting from tree removal operations will extend to depths of about 3 to 4 feet BSG. All roots larger than ¼ inch in diameter and any accumulation of organic matter that will result in an organic content more than 3 percent by weight should be removed and not used as engineered fill. The areas occupied by trees should be excavated to a minimum depth of 12 inches below the excavations required to remove the tree, root ball, and roots. The bottom of the excavation should be scarified to a minimum depth of 8 inches and compacted as engineered fill prior to backfilling operations.

8.3.3 As part of site preparation, all existing underground utilities, foundations, subsurface structures, and associated fills should be excavated and removed from the site and all soils disturbed from the demolition and removal of these improvements should be over-excavated to expose undisturbed soils. Trench backfill soils should be excavated from within a zone extending from 1 foot below the pipe at a 1H to 1V slope to the ground surface. Utilities to be removed should be completely removed and disposed of off-site. Excavations to remove existing improvements should extend to at least 12 inches below the bottom of the improvements to be removed or to the depth required to remove all soils disturbed from demolition, whichever is greater. After over-excavation, prior to backfill, the bottom of the excavation should be scarified to a depth of 8 inches, moisture conditioned, and compacted as engineered fill.

8.3.4 After stripping and removal of existing surface and subsurface improvements, the building area and all new foundations should be over-excavated to at least 4 feet below preconstruction site grades, to at least 12 inches below the bottom of the existing improvements to be removed, to the depth required to remove all fill soils (encountered to a depth of 5 feet BSG in boring B-2), and to at least 18 inches below the bottom of the footings, whichever is greater. The over-excavation limits should include the entire building footprint, all foundations and adjacent walkways, and a minimum of 5 feet beyond the foundations, or 5 feet beyond walkways adjacent to the building, whichever is further. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the excavation should be scarified 8 inches in depth, moisture conditioned one (1) to four (4) percent above optimum moisture content and compacted as engineered fill.

8.3.5 The plans should depict the minimum limits of over-excavation for the building pad as described in section 8.3.4.

8.3.6 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the over-excavation and compaction conform to the site preparation recommendations presented in this report. Moore Twining is not responsible for surveying and measuring to verify the horizontal and vertical extent of over-excavation and compaction. The contractor should verify in writing to the owner and Moore Twining that the horizontal and vertical over-excavation limits were completed in conformance with the recommendations of this report, the project plans, and the specifications (the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from Moore Twining or excavating for foundations.

8.3.7 Following stripping and removal of existing surface and subsurface improvements, exterior slabs-on-grade which are not located adjacent to the building (i.e., outside the building pad preparation limits), pavements and areas to receive fill outside the building pad over-excavation limits should be prepared by over-excavation to a minimum of 12 inches below the resulting ground surface, to the depth required to remove existing fill soils, to the bottom of the aggregate base, to at least 12 inches below the bottom of improvements to be removed, and to the depth required to remove all fill soils, whichever is greater. Over-excavation should extend horizontally a minimum of 3 feet beyond exterior slabs on grade and pavements, or up to

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the existing improvements to remain, whichever occurs first. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the over-excavation should be scarified to a minimum depth of 12 inches, moisture conditioned to between one (1) and four (4) percent above optimum moisture content and compacted as engineered fill.

8.3.8 Structural loads for miscellaneous, lightly loaded foundations (such as retaining walls, sound walls, screen walls, monument signs, etc.) should be evaluated on a case by case basis to present supplemental recommendations for site preparation and foundation design. In lieu of a case by case evaluation, the areas of miscellaneous foundations should be over-excavated to at least 24 inches below preconstruction site grades, to at least 12 inches below subsurface structures to be removed, to the depth required to remove all existing fill soils, and to the bottom of foundations, whichever is greater. After approval of the over-excavation by Moore Twining Associates, Inc., the bottom of the over-excavation should be scarified to a depth of 8 inches, moisture conditioned to one (1) to four (4) percent above optimum moisture content and compacted as engineered fill. The over-excavation should extend a minimum of 3 feet beyond the limits of the foundations on all sides, or to property lines, or to improvements to remain, whichever occurs first.

8.3.9 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

8.3.10 The contractor should locate all on-site water wells (if any). All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Moore Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters) should have their casings removed to a depth of at least 8 feet below preconstruction site grades or finished pad grades, whichever is deeper. In parking lot or landscape areas, the casings should be removed to a depth of at least 5 feet below site grades or finished grades. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill.

8.3.11 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

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8.3.12 Final grading shall produce a building pad ready to receive a slab-on-grade which is smooth, planar, and resistant to rutting. The finished pad (before aggregate base is placed) shall not depress more than one-half (½) inch under the wheels of a fully loaded water truck, or equivalent loading. If depressions more than one-half (½) inch occur, the contractor shall perform remedial grading to achieve this requirement at no cost to the owner.

8.3.13 The Contractor should be responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. (if any) that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor should be responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal.

8.4 Engineered Fill

8.4.1 The near surface soils encountered with an expansion index of less than 80 are considered suitable for use as engineered fill below depths of 18 inches below interior concrete slabs on grade and below depths of 12 inches below exterior concrete slabs on grade and Portland cement concrete pavements, provided that the soils are free of debris, do not contain material greater than 6 inches in maximum dimension, and are moisture conditioned in accordance with the recommendations of this report. During site preparation, debris and unsuitable materials encountered should be removed from the soils to be used as engineered fill. Interior concrete slabs on grade and exterior concrete slabs on grade directly adjacent to the building should be supported on a minimum of 6 inches of non-recycled Class 2 aggregate base over 12 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. Exterior slabs-on-grade which are not located adjacent to the building and Portland cement concrete pavements should be underlain by 6 inches of Class 2 aggregate base over 6 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. As an alternative, Class 2 aggregate base may be substituted for the imported, non-expansive granular fill soils.

8.4.2 If soils other than those considered in this report are encountered, Moore Twining should be notified to provide alternate recommendations.

8.4.3 The compactability of the native soils is dependent upon the moisture contents, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, it is recommended that they be evaluated by the contractor during preparation of bids and construction of the project.

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8.4.4 Import fill soil used for the building pad preparation (if any) should be non-recycled, have a very low expansion potential and be granular in nature with the following acceptance criteria recommended.

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 - 100
Percent Passing No. 200 Sieve	10 - 40
Expansion Index (ASTM D4829)	Less than 15
Plasticity Index (ASTM D4318)	Less than 12
Organics	Less than 3 percent by weight
Sulfates	< 0.05 percent by weight
Resistivity	> 5,000 ohms-cm

Prior to importing fill, the import material shall be certified by the Contractor and the supplier (to the satisfaction of the Owner) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. The Contractor shall pay for the environmental testing required to determine compliance with the requirements of this report. This certification shall consist of, as a minimum, recent analytical data specific to the source of the import material including proper chain-of-custody documentation. Moore Twining will sample and test the material after the environmental certification submittal is approved to verify that the proposed material complies with the geotechnical engineering recommendations of this report. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested for the geotechnical properties.

8.4.5 Onsite soils used as engineered fill and final utility trench backfill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to at least one (1) and four (4) percent above optimum moisture content, and compacted to a dry density of at least 90 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.6 On-site silty sand soils and imported, granular engineered fill, bedding sand and initial utility trench backfill should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to optimum to three (3)

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percent above optimum moisture content, and compacted to a dry density of at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557, with exception that the upper 12 inches of subgrade below the aggregate base for pavements should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.7 Utility trenches should be a minimum of 24 inches in width to allow for in-place density testing by traditional (nuclear density test) methods and the backfill should be compacted in accordance with the recommendations for engineered fill in Sections 8.4.5 and 8.4.6 of this report.

8.4.8 In-place density testing should be conducted in accordance with ASTM D 6938 (nuclear methods) at the minimum frequency listed in Table No. 3, below.

Table No. 3
Minimum In-place Density Test Frequency

Area	Minimum Test Frequency
Mass Fills or Subgrade for Building Pad	1 test per 5,000 square feet per compacted lift, but not less than 3 tests per building pad per lift
Pavement Subgrade and Aggregate Base	1 test per 10,000 square feet per compacted lift
Utility Lines	1 test per 150 feet per compacted lift

8.4.9 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill, all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Moore Twining. If the contractor elects to use crushed rock (and if approved by Moore Twining), the contractor will be responsible for slurry cut off walls at the locations directed by Moore Twining. Materials such as crushed rock should be placed in thin (less than 8 inches) lifts and each lift should be compacted with a minimum of three (3) passes with a vibratory compactor.

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8.4.10 Aggregate base below the building slab should comply with State of California Department of Transportation requirements for a non-recycled Class 2 aggregate base, with exception that the aggregate base used below the building slab should not contain recycled materials. Aggregate base should be compacted to a minimum relative compaction of 95 percent. Prior to importing the aggregate base material, the contractor should submit documentation demonstrating that the material meets all the quality requirements (i.e., gradation, R-value, sand equivalent, durability, etc.) for the applicable aggregate base. Documentation should be provided to the Owner, Architect and Moore Twining and reviewed and approved prior to delivery of the aggregate base to the site.

8.5 Conventional Shallow Spread Foundations and Concrete Slabs on Grade

8.5.1 A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations and slabs on grade based on the estimated settlements. The following should be anticipated for design: 1) a total static settlement and heave of 1 inch; 2) a differential static settlement and heave of ½ inch in 40 feet; 3) a total seismic settlement of ⅔ inch, and 4) a differential seismic settlement of ½ inch in 40 feet.

8.5.2 Building foundations supported on engineered fill soils prepared as recommended in the Site Preparation section of this report may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. This value may be increased by one-third for short duration wind or seismic loads.

8.5.3 All perimeter footings for the new building should have a minimum depth of 24 inches below the lowest adjacent grade. All interior foundations should have a minimum depth of 18 inches below the bottom of the floor slab. All footings for the new building should have a minimum width of 15 inches, regardless of load.

8.5.4 The foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

8.5.5 Structural loads for miscellaneous, lightly loaded non-building foundations (such as retaining walls, sound walls, screen walls, monument signs, etc.) should be supported on subgrade soils prepared as recommended in the Site Preparation section of this report. Spread and continuous footings for miscellaneous foundations extending a minimum depth of 18 inches below

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grade may be designed for a maximum net allowable soil bearing pressure of 2,500 pounds per square foot for dead-plus-live loads. These values may be increased by one-third for short duration wind or seismic loads. The weight of the footing and the soil backfill may be ignored in design.

8.5.6 The values in Table No. 4 were developed using the Ground Motion Parameter Calculator provided by the United States Geological Survey (http://earthquake.usgs.gov/) in accordance with the 2016 CBC, a site latitude of 36.98777 degrees, and a longitude of -121.98381 degrees.

Table No. 4
Seismic Factors

Seismic Factor	2016 CBC Value
Site Class	D
Maximum Considered Earthquake (geometric mean) peak ground acceleration adjusted for site effects (PGA_M)	0.5
Mapped Maximum Considered Earthquake (geometric mean) peak ground acceleration ASCE 7-10 (PGA)	0.5
Spectral Response At Short Period (0.2 Second), Ss	1.500
Spectral Response At 1-Second Period, S_1	0.600
Site Coefficient (based on Spectral Response At Short Period), Fa	1.0
Site Coefficient (based on spectral response at 1-second period) Fv	1.5
Maximum considered earthquake spectral response acceleration for short period, S_{MS}	1.500
Maximum considered earthquake spectral response acceleration at 1 second, S_{M1}	0.900
Five percent damped design spectral response accelerations for short period, S_{DS}	1.000
Five percent damped design spectral response accelerations at 1-second period, S_{D1}	0.600

8.5.7 The prepared soils exposed in foundation excavations should be periodically moistened to maintain the moisture content in the onsite clayey soils at a minimum of one (1) percent above optimum until concrete is placed. It should be noted that the contractor should take precautions not to allow the exposed soils to dry, including weekends and holidays.

8.5.8 Foundation excavations should be observed by Moore Twining prior to the placement of steel reinforcement and concrete to verify conformance with the intent of the recommendations of this report. The Contractor is responsible for proper notification to Moore Twining and receipt of written confirmation of this observation prior to placement of steel reinforcement.

8.5.9 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.25 can be used for design. In areas where slabs are underlain by a synthetic moisture vapor membrane, an allowable coefficient of friction of 0.10 can be used for design.

8.5.10 The allowable passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. The upper 12 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.5.11 Site lighting and pylon signs may be supported on a drilled-cast-in-hole reinforced concrete foundation (pier). An allowable skin friction of 150 pounds per square foot may be used to resist axial loads. Lateral load resistance may be estimated using the 2016 CBC non-constrained procedure (Section 1807.3.2.1). The allowable passive resistance of the native soils may be assumed to be equal to the pressure developed by a fluid with a density of 200 pounds per cubic foot to a maximum of 2,000 pounds per square foot. The passive pressure may be assumed to act over twice the pier diameter. The upper 12 inches of subgrade soils in landscape areas should be neglected in determining the total passive resistance.

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8.6 Frictional Coefficient and Earth Pressures

8.6.1 The bottom surface area of concrete footings in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.25, can be used for design.

8.6.2 The allowable passive resistance of the engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. The upper 12 inches of subgrade in landscape areas should be neglected in determining the total passive resistance.

8.6.3 The active and at-rest pressures of imported, non-expansive engineered fill meeting the requirements of Section 8.9.1 of this report may be assumed to be equal to the pressures developed by a fluid with a density of 45 and 68 pounds per cubic foot, respectively. These pressures assume level ground surface and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.6.4 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

8.6.5 The wall designer should determine if seismic increments should be used or not. If seismic increments are required, contact Moore Twining for recommendations for seismic geotechnical design considerations for the retaining structures.

8.6.6 The above earth pressures assume that the backfill soils will be drained. Therefore, all retaining walls should incorporate the use of a drain, a filter fabric encased gravel section and a geo-composite system, to prevent hydrostatic pressures from acting on the walls. Recommendations for drainage of walls are included in Section 8.7 of this report. Drainage should be directed to perforated pipes running parallel to the walls which can carry drainage from behind the walls to the on-site drainage system. Clean-outs should be incorporated into the design.

8.7 Retaining Walls

8.7.1 Retaining walls should be backfilled with on-site or imported, granular backfill (See Section 8.4.4 of this report) placed within the zone extending from a distance of 1 foot laterally from the bottom of the wall footing at a 1 horizontal to 1 vertical gradient to the surface. This requirement should be detailed on the construction drawings.

8.7.2 All retaining wall backfill should be compacted as engineered fill.

8.7.3 Retaining walls may be subject to lateral loading from pressures exerted from slabs-on-grade, and pavement traffic loads, adjacent to the walls. In addition to earth pressures, lateral loads due to slabs-on-grade, footings, or traffic above the base of the walls should be included in design of the walls. The designer should take into consideration the allowable settlements for the improvements to be supported by the retaining wall.

8.7.4 Retaining walls should be constructed with a drain system including, as a minimum, drain pipes surrounded by at least 1 cubic foot of ¾-inch open graded rock fully encapsulated by geotextile filter fabric (140N or equivalent). Drain pipes should be located near the wall to adequately reduce the potential for hydrostatic pressures behind the wall. Drainage should be directed to pipes which gravity drain to closed pipes of the storm drain system. Drain pipe outlet invert elevations should be sufficient (a bypass should be constructed if necessary) to preclude hydrostatic surcharge to the wall in the event the storm drain system does not function properly. Clean out and inspection points should be incorporated into the drain system. Drainage should be directed to the site storm drain system. The drainage system should be detailed on the plans.

8.7.5 Segmented wall design (mechanically stabilized walls) should be conducted by a California licensed geotechnical engineer familiar with segmented wall design and having successfully designed at least three walls at sites with similar soil conditions. However, none of the data included in this report should be used for mechanically stabilized earth wall design. A design level geotechnical report should be conducted to provide wall design parameters. If the designer uses the data in this report for wall design, the designer assumes the sole risk for this data. The wall designer should perform sufficient observations of the wall construction to certify that the wall was constructed in accordance with the design plans and specifications.

8.7.6 It is recommended to use lighter hand operated or walk behind compaction equipment in the zone equal to one wall height behind the wall to reduce the potential for damage to the wall during construction. Heavier compaction equipment could cause loads in excess of design loads which could result in cracking, excessive rotation, or failure of a retaining structure. The Contractor is responsible for damage to the wall caused by improper compaction methods behind the wall.

8.7.7 If retaining walls are to be finished with dry wall, plaster, decorative stone, etc., waterproofing measures should be applied to walls. Waterproofing systems should be designed by a qualified professional.

8.8 Interior Concrete Slabs on Grade and Moisture Vapor Retarder

The recommendations provided herein are intended only for design of interior concrete slabs-on-grade, and their proposed uses, which do not include construction loading. The building contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

8.8.1 The concrete slabs on grade should be reinforced for the anticipated temperature and shrinkage stresses, settlement and swell. A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade as well as any reinforcement for temperature and shrinkage stresses based on the settlements noted in this report.

8.8.2 The subgrade soils should be prepared as recommended in the "Site Preparation" section of this report. Upon completion of the over-excavation and compaction of subgrade soils, the concrete slabs on grade should be supported on 6 inches of non-recycled aggregate base over 12 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. As an alternative, Class 2 aggregate base may be substituted for the imported, non-expansive granular fill soils.

8.8.3 The moisture content of the clay subgrade soils below the non-expansive fill should be verified to be at least one (1) percent above optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

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8.8.4 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarder when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 should be used where floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The vapor retarder should overlay the 6 inches of compacted aggregate base and 12 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. It should be noted that placing the PCC slab directly on the vapor barrier will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the engineered fill soils and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with the current ASTM C 755, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to the current ASTM E 154 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor barrier selection and installation conform to the current ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R), Addendum, Vapor Retarder Location and current ASTM E 1643, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements. It should be noted that the recommendations presented in this report are not intended to achieve a specific vapor emission rate.

8.8.5 The slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

8.8.6 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape continuous at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be caulked per manufacturer's recommendations.

8.8.7 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per the manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.

8.8.8 The moisture retarding membrane is not required beneath exposed concrete floors, such as exposed warehouses floors, provided that moisture intrusion into the structure is permissible for the design life of the structure.

8.8.9 Additional measures to reduce moisture migration should be implemented for floors that will receive moisture sensitive coverings. These include: 1) constructing a less pervious concrete floor slab by maintaining a low water-cement ratio of 0.52 lb./lb. or less in the concrete for slabs-on-grade, 2) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture barrier, 3) placing concrete walkways or pavements adjacent to the structure, 4) providing adequate drainage away from the structure, 5) moist cure the slabs for at least 7 days, and 6) locating lawns, irrigated landscape areas, and flower beds away from the structure.

8.8.10 To reduce the potential for damaging slabs during construction, the following recommendations are presented: 1) design for a differential slab movement of ½ inch relative to perimeter foundations; 2) provide an aggregate base layer below the slabs; and 3) the suitability of the loads from construction equipment which will operate on slabs or pavements should be evaluated by the contractor prior to loading the slab.

8.8.11 If construction traffic will be traveling over the aggregate base material, or the aggregate base will be used as a working surface, the contractor should determine an adequate aggregate base section thickness for the type and methods of construction proposed for the project. The proposed compacted subgrade can experience instability under construction traffic resulting in heaving and depressions in the subgrade. Often the aggregate base can reduce the potential for instability under the construction traffic.

8.8.12 The Contractor shall test the moisture vapor transmission through the slab, the pH, internal relative humidity, etc., at a frequency and method as specified by the flooring manufacturer or as required by the plans and specifications, whichever is most stringent. The results of vapor transmission tests, pH tests, internal relative humidity tests, ambient building conditions, etc. should be within floor manufacturer's and adhesive manufacturer's specifications at the time the floor is placed. It is recommended that the floor manufacturer and subcontractor review and approve the test data prior to floor covering installation.

8.8.13 Backfill the zone above the top of footings at interior column locations, building perimeters, and below the bottom of slabs with an approved backfill as recommended herein for the area below interior slabs-on-grade. This procedure should provide more uniform support for the slabs which may reduce the potential for cracking.

8.9 <u>Exterior Slabs-On-Grade</u>

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, rather lightly loaded sidewalks, curbs, and planters, etc.

8.9.1 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with the recommendations presented in this report for interior slabs-on-grade. Moore Twining can provide alternative design recommendations for exterior slabs, if requested.

8.9.2 Subgrade soils for exterior slabs should be prepared as recommended in the "Site Preparation" section of this report. Exterior slabs on grade directly adjacent to the building should be supported on 6 inches of aggregate base over 12 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. The exterior slabs on grade should be supported on 6 inches of aggregate base over 6 inches of imported, non-expansive, granular fill soils over the prepared subgrade soils. As an alternative, Class 2 aggregate base may be substituted for the imported, non-expansive granular fill soils.

8.9.3 The moisture content of the clay subgrade soils below the non-expansive fill should be verified to be at least one (1) percent above optimum moisture content within 48 hours of placement of the aggregate base. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

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8.9.4 The exterior slabs-on-grade adjacent to landscape areas should be designed with thickened edges which extend to the bottom of the aggregate base and imported, non-expansive granular fill layer.

8.9.5 Since exterior sidewalks, curbs, etc. are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing concrete walks and finish work over dry or slightly moist subgrade should be avoided. It is recommended that the general contractor notify Moore Twining to conduct in-place moisture and density tests prior to placing concrete flatwork. Written test results indicating passing density and moisture tests (minimum of two percent over optimum for the clay subgrade soils) should be in the general contractor's possession prior to placing concrete for exterior flatwork.

8.10 Asphaltic Concrete (AC) Pavements

Recommendations are provided below for new asphaltic concrete pavements planned as part of the new construction and are not for intended for pervious pavements.

8.10.1 The subgrade soils for asphaltic concrete pavements should be prepared as recommended in the "Site Preparation" section of the recommendations in this report.

8.10.2 The following pavement sections are based on an R-value of 20, a minimum asphaltic concrete thickness of 3 inches, and traffic index values ranging from 5.0 to 7.0. A traffic index of 6.5 appears to correspond with the CVS criteria of a maximum of 60,000 ESALs. The appropriate paving section should be determined by the project civil engineer or applicable design professional based on the actual vehicle loading (traffic index) values. It should be noted that if pavements are constructed prior to construction of the buildings, the traffic index value should account for construction traffic. After rough subgrade preparation, samples should be obtained and tested to confirm the design R-value and provide final pavement section thickness recommendations.

Table No. 5
Two-Layer Asphaltic Concrete Pavements

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	7.0	12
5.5	3.0	9.0	12
6.0	3.0	10.5	12
6.5	3.5	11.0	12
7.0	4.0	12.0	12

AC - Asphaltic Concrete compacted as recommended in this report
AB - Class II Aggregate Base with minimum R-value of 78 and compacted to at least 95 percent relative compaction (ASTM D1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D1557)

8.10.3 The curbs where pavements meet irrigated landscape areas or uncovered open areas should extend at least to the bottom of the aggregate base section. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.10.4 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement sections should be re-evaluated for the changed subgrade conditions.

8.10.5 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement sections should be re-evaluated for the anticipated traffic.

8.10.6 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

8.10.7 Pavement materials and construction method should conform to the State of California Standard Specifications.

8.10.8 It is recommended that the base course of asphaltic concrete consist of a ¾ inch maximum medium gradation. The top course or wear course should consist of a ½ inch maximum medium gradation.

8.10.9 The asphaltic concrete, including the joint density, should be compacted to an average relative compaction of 93 percent, with no single test value being below a relative compaction of 91 percent and no single test value being above a relative compaction of 97 percent of the referenced laboratory density according to ASTM D2041.

8.10.10 The asphalt concrete should comply with the requirements for a Type "B" asphalt concrete as described in Section 39 of the 2010 (non-revised) State of California Department of Transportation (Caltrans) Standard Specification, or the requirements of the governing agency, whichever is more stringent.

8.11 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections and are not intended for pervious pavements. The design professional should specify the pavement sections based on the anticipated type and frequency of traffic.

8.11.1 The subgrade soils for PCC pavements should be compacted as recommended in the "Site Preparation" section of the recommendations in this report. The concrete pavement should be underlain by a minimum of 6 inches of aggregate base over 6 inches of non-expansive fill over the prepared subgrade.

8.11.2 The following pavement section designs are based on a design modulus of subgrade reaction, K-value, of 160 psi/in. The design thicknesses were prepared based on the procedures outlined in the Portland Cement Association (PCA) document, "Thickness Design for Concrete Highway and Street Pavements," our review of the CVS Geo-Technical Investigation Requirements, and assuming the following: 1) minimum modulus of rupture of 500 psi for the concrete, 2) a design life of 20 years, 3) load transfer by aggregate interlock or dowels, 4) concrete shoulder, 5) a load safety factor of 1.1, and 6) truck loading consisting of 1 single axle load of 18 kips, and 7) a maximum load of 60,000 ESAL.

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Table No. 6
Two-Layer Portland Cement Concrete Pavements

Traffic Index	ADTT (Trucks/day)	PCC thickness (inches)	Aggregate Base[1] (inches)	Imported, Non-Expansive, Granular Fill (inches)	Compacted Subgrade[2] (inches)
6.5	8.0	7.0	6.0*	6.0*	12.0

ADTT - Average Daily Truck Traffic
PCC - Portland Cement Concrete (minimum Modulus of Rupture=500 psi)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)
* - As an alternative to the use of a 6 inch layer of non-expansive fill below the aggregate base layer, a 12 inch layer of aggregate base may be used below the PCC pavement

8.11.3 Concrete used for PCC pavements shall possess a minimum flexural strength (modulus of rupture) of 500 pounds per square inch. A minimum compressive strength of 3,500 pounds per square inch, or greater as required by the pavement designer, is recommended. Specifications for the concrete to reduce the effects of excessive shrinkage, such as maximum water requirements for the concrete mix, allowable shrinkage limits, contraction joint construction requirements, curing methods, etc. should be provided by the designer of the PCC slabs.

8.11.4 The pavement section thickness design provided above assumes the design and construction will include sufficient load transfer at construction joints. Coated dowels or keyed joints are recommended for construction joints to transfer loads.

8.11.5 Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade soils. The type of joint filler should be specified by the pavement designer. The joint sealer and filler material should be maintained throughout the life of the pavement.

8.11.6 Contraction joints should have a depth of at least one-fourth the slab thickness, e.g., 1.5-inch for a 6-inch slab. Specifications for contraction joint spacing, timing and depth of sawcuts should be included in the plans and specifications.

8.11.7 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

8.11.8 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet. Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

8.11.9 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas.

8.11.10 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

8.12 Temporary Excavations

8.12.1 It is the responsibility of the Contractor to provide safe working conditions with respect to excavation slope stability. The Contractor is responsible for site slope safety, and classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

8.12.2 Temporary excavations should be constructed in accordance with CAL OSHA requirements. Temporary cut slopes should not be steeper than 1.5 to 1, horizontal to vertical, and flatter if possible. If excavations cannot meet these criteria, the temporary excavations should be supported by engineered shoring systems.

8.12.3 In no case should non-shored excavations extend below a 1.5H to 1V zone below existing utilities, top of foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 1.5 H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.12.4 Shoring systems should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Moore Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.12.5 Surface sheet flow drainage shall be directed away from the tops of all excavations. Positive drainage shall be established and maintained throughout the construction process.

8.12.6 Excavation and shoring stability should be monitored by the Contractor. Slope gradient estimates provided in this report do not relieve the Contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Moore Twining should be notified immediately and the Contractor should take appropriate actions to minimize further damage or injury.

8.13 Corrosion Protection

8.13.1 Buried metal objects should be protected in accordance with the manufacturer's recommendations based on a "highly corrosive" corrosion potential. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

8.13.2 Corrosion of concrete due to sulfate attack is not anticipated based on the concentration of sulfates determined for the near-surface soils (0.024 and 0.0019 percent by dry weight concentration of sulfate). According to provisions of ACI 318, section 4.3, the sulfate concentration falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, no restrictions are required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content. However, a low water to cement ratio is recommended for slabs on grade as recommended in section 8.8.8 of this report.

8.13.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Moore Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

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9.0 DESIGN CONSULTATION

9.1 Moore Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is not part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Moore Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Moore Twining.

10.0 CONSTRUCTION MONITORING

10.1 It is recommended that Moore Twining be retained to observe the excavation, earthwork, and foundation phases of work to determine that the subsurface conditions are compatible with those used in the analysis and design. In the event Moore Twining does not conduct the observations and testing of the building pad preparation, reports signed by a registered geotechnical engineer documenting the earthwork inspections, in-place density testing and certification of the pad as meeting the project requirements should be provided to our firm for review.

10.2 Moore Twining can conduct the necessary observation and field testing to provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, a written summary of our observations, field testing and conclusions will be provided regarding the conformance of the completed work to the intent of the plans and specifications. This service is not, however, part of this current contractual agreement.

10.3 In the event that the earthwork operations for this project are conducted such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) it is recommended that the exposed subgrade that will receive floor slabs be tested to verify adequate compaction and/or moisture conditioning. If adequate compaction or moisture contents are not verified, the fill soils should be over-excavated, scarified, moisture conditioned and compacted are recommended in the Recommendations of this report.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Moore Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Moore Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Moore Twining will not be responsible for compliance of the earthwork preparation with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. It is recommended that if a firm other than Moore Twining is selected to conduct these services that they provide evidence of professional liability insurance satisfactory to the owner and review this report. After their review, the firm should, in writing, state that they agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations. Moore Twining should be notified, in writing, if another firm is selected to conduct observations and field-testing services prior to construction.

10.6 Upon the completion of work, a final report should be prepared by Moore Twining. This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Moore Twining upon the completion of work to prepare a final report summarizing the observations during site preparation activities relative to the recommendations of this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations. The nature and extent of subsurface variations between borings may not become evident until construction.

11.2 If variations or undesirable conditions are encountered during construction, Moore Twining should be notified promptly so that these conditions can be reviewed and our recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

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11.3 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (over 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.4 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.5 The conclusions and recommendations contained in this report are valid only for the project discussed in the Anticipated Construction section of this report. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in this report is not recommended. The entity or entities that use or cause to use this report or any portion thereof for other structures or site not covered by this report shall hold Moore Twining, its officers and employees harmless from any and all claims and provide Moore Twining's defense in the event of a claim.

11.6 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, designers, contractors, subcontractors, and other parties having interest in the project so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.7 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.8 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

11.9 Reliance on this report by a third party (i.e., that is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Moore Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Boos Development West, LLC. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,

MOORE TWINING ASSOCIATES, INC.
Geotechnical Engineering Division

Allen H. Harker

Allen H. Harker, PG
Professional Geologist

PROFESSIONAL GEOLOGIST
ALLEN H. HARKER
No. 7925
Exp. 7-31-18
STATE OF CALIFORNIA

Read L. Andersen, RGE
Manager

REGISTERED PROFESSIONAL ENGINEER
READ L. ANDERSEN
2810
EXP. 12-31-18
GEOTECHNICAL
STATE OF CALIFORNIA

CITY OF SANTA CRUZ

WATER DEPARTMENT

212 Locust Street, Suite C Santa Cruz CA 95060 Phone (831) 420-5200 Fax (831) 420-5201

November 27, 2017

Amelia Beltran
555 Capitol Mall, Suite 300
Sacramento, CA 95814

Re: PROPOSED RETAIL/PHARMACY/COMMERCIAL DEVELOPMENT AT 1515 COMMERCIAL WAY; APN 025-071-05 & 025-071-20.

Dear Ms. Beltran:

This letter is to advise you that the subject parcels are located within the service area of the Santa Cruz Water Department and potable water is currently available for normal domestic use and fire protection. Service will be provided to each and every lot of the development upon payment of the fees and charges in effect at the time of service application and upon completion of the installation, at developer expense, of any water mains, service connections, fire hydrants and other facilities required for the development under the rules and regulations of the Santa Cruz Water Department. The development will also be subject to the City's Landscape Water Conservation requirements.

At the present time:

- the required water system improvements are not complete; and
- financial arrangements have not been made to the satisfaction of the City to guarantee payment of all unpaid claims.

This letter will remain in effect for a period of two years from the above date. It should be noted, however, that the City Council may elect to declare a moratorium on new service connections due to drought conditions or other water emergency. Such a declaration would supersede this statement of water availability.

If you have any questions regarding service requirements, please call the Engineering Division at (831) 420-5210. If you have questions regarding landscape water conservation requirements, please contact the Water Conservation Office at (831) 420-5230.

Sincerely,

Rosemary Menard

Rosemary Menard
Water Director

RM/bjd
Cc: SCWD Engineering

ATTACHMENT 6

Kimley»Horn

Preliminary Stormwater Control Plan

for

CVS pharmacy™

Soquel Dr. & Commercial Way

Santa Cruz County, CA

Revised: March 2019

November 2018

Prepared for:

Boos Development West, LLC

2020 L Street, Suite 245

Sacramento, Ca 95811

Leanna Swenson

(916) 346-4797

Prepared By:

Kimley-Horn and Associates

5555 Capitol Mall, Suite 300

Sacramento, CA 95814

Contact: Sheetal K. Bhatt

(916) 859 - 3609

ATTACHMENT

I. Project Data

Table 1. Project Data

Project Name/Number	CVS Santa Cruz
Application Submittal Date	3/11/2019
Project Location	1515 Commercial Way, Santa Cruz, CA
Project Phase No.	N/A
Project Type and Description	The project proposes to improve two existing commercial properties at 1515 Commercial Way into a CVS pharmacy . Project scope includes construction of a CVS Pharmacy store and associated improvements such as paving, landscaping and utilities.
Total Project Site Area (acres)	1.29 acres (51,904 S.F. On-Site & 4,278 S.F Off-Site)
Total New Impervious Surface Area	0.30 acres
Total Replaced Impervious Surface Area	0.74 acres
Total Pre-Project Impervious Surface Area	0.74 acres
Total Post-Project Impervious Surface Area	1.04 acres
Watershed Management Zone(s)	1
Design Storm Frequency and Depth	95th Percentile 24-hr rainfall event (2")
Urban Sustainability Area	N/A

II. Setting

II.A. Project Location and Description

The CVS Santa Cruz project (Project) site comprises approximately 1.19 acres (onsite) and 0.1 ac (offsite) and is located at 1515 Commercial Way, in Santa Cruz Count, California. The site is bounded by Mid County Auto Center to the east, Soquel Drive to the north, Commercial Way to the south and a 76 Gas Station to the west. The project site is located approximately 2 miles north of the Pacific Ocean and Woods Lagoon, 0.1 east of Highway 1. The project is located at Lat 36.98784 Lon 121.984. The site is located in Watershed Management Zone 1.

The project includes the construction of a new 13,111 S.F. CVS Pharmacy Building with 1,712 S.F. Mezzanine and associated on-site improvements such as landscaping, utilities and paving. To address stormwater quality, the project proposes constructing three (3) biofilteration treatment systems/basins – two (2) along Soquel Drive and one (1) along Commercial Way (Appendix B – C4.0).

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II.B. Opportunities and Constraints for Stormwater Control

The site is adjacent to two public streets. There is a 10' setback required on street frontages. These required buffers create an area that can be vegetated and can contain the required stormwater quality facility.

According to the "Geotechnical Investigation Proposed CVS Pharmacy 1505 and 1515 Commercial Way by Moore Twining Associates, Inc dated January 15, 2018, the project site is characterized with clayey sands extending 2 to 10 feet below the surface or lean clays, lean clays with sand or sandy lean clays extending to depths of 2.5' to 8.5'. The near surface clayey sands were underlain by lean clays, clayey sands, silty sands extending to the maximum depth explored, 26.5' clay within the upper 15 to 35 feet of the surface. The site is characterized by soil type D with low infiltration rates. During their field investigation, Moore Twining Associates, Inc stated that they encountered groundwater in their test bore holes at depths between 14.5 feet to 23.75 feet below ground surface. Additionally, per Hydrologic Soil Group from the Central Coast Stormwater Management Requirements GIS Soil Data, the project has soil Type D, reference Appendix A. The report states that on-site retention is not recommended for this project site due to low infiltration rates. For water quality treatment design, it is assumed that there is no infiltration potential on-site. Hence, the project proposes installing underdrains at the bottom of the gravel layer in the proposed biofiltration treatment basins.

III. Low Impact Development Design Strategies

III.A. Optimization of Site Layout

The existing project site consists of two parcels. The parcel on the west side of the site contains an existing building and open space. The parcel on the east side of the site contains an existing building and paved parking areas. Since project replaces a partially developed site, there is little increase in impervious area, and drainage patterns remain relatively similar.

III.A.1. Limitation of development envelope

A portion of the site is developed in the existing conditions. See Appendix A for Figures 1 and 2 for the existing and proposed conditions drainage maps.

III.A.2. Preservation of natural drainage features

A portion of the site is currently developed. The proposed drainage features will tie into the existing City of Santa Cruz Municipal Storm Drain System located in both Soquel Drive and Commercial Way.

III.A.3. Setbacks from creeks, wetlands, and riparian habitats

There are no creeks, wetlands and riparian habitats on the site.

III.A.4. Minimization of imperviousness

The project proposes using vegetation and parking spaces in the areas of the site not encompassed by the building and paving. The minimum number of required parking spaces and necessary drive aisles are provided to minimize the impervious area.

III.A.5. Use of drainage as a design element

The proposed project directs runoff from the parking areas and building onto vegetated areas. These areas are located on the north and south edges of the site.

III.B. Use of Permeable Pavements

Permeable pavement is not used on this project. Low site infiltration rates make it infeasible to use permeable pavement on the project site.

III.C. Dispersal of Runoff to Pervious Areas

The proposed project directs runoff from the parking areas onto vegetated areas. These areas are located on the north and south edges of the site. The runoff conveyed in roof drains are collected by storm drain inlets and conveyed to the biofiltration treatment areas through storm drain pipes and bubble-up structure.

III.D. Stormwater Control Measures

Three stormwater control measures are proposed as part of this project. The locations of these Best Management Practices (BMPs) are provided as shown in Appendix A - Figure 2 and Appendix B – C4.0. Two BMPs are located on the north edges of the property and one BMP is located on the south edge of the property. Runoff from the adjacent parking areas is directed via sheet flow through curb cuts and storm drain system to the BMPs. The BMPs have a surface area that is a minimum of 5% of the tributary impervious area.

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IV. Documentation of Drainage Design

Proposed pervious and impervious areas are summarized in Appendix B – C4.0. The total proposed impervious area at project completion is approximately 1.04 acres, compared to 0.74 acres of existing impervious area.

Per the County's Standards, the project is required to conduct site design measures and runoff reduction (Performance Requirement #1), treat the 85th percentile 24-hour storm event (Performance Requirement #2), retain the 95th percentile 24-hour storm event (Performance Requirement #3), and apply peak flow management for the 2-year and 10-year design flow events (Performance Requirement #4).

The water quality flow and runoff retention for the project site were determined following Santa Cruz County Design Criteria and the Central Coast Post-Construction Stormwater Management Requirements for Development Projects in the Central Coast Region.

To meet Performance Requirement # 1, the proposed project will direct runoff from sidewalks, parking lots, and buildings into adjacent biofiltration treatment areas. The project proposes construction of three (3) biofiltration treatment areas. The two biofiltration treatment areas along the north side of the property will act 1 basin by using an equalizer pipe.

To comply with Performance Requirement #2, the project must treat runoff produced by a rain event equal to 0.2 inches per hour intensity. The project is proposing to use biofiltration treatment systems with a maximum surface loading rate equal to 5 inches per hour and using the flow of runoff produced by a rain event equal to 0.2 inches per hour intensity using the "4 percent method" (0.2 in/hour divided by 5in/hour = 0.04). The project's biofiltration treatment areas were sized so that the footprint provided for stormwater treatment is minimum 0.05 times the effective impervious area. The effective impervious area was determined by taking the sum of the total proposed impervious area and 0.1 times the total proposed pervious area.

Table 2. Water Quality Treatment

BMP	Effective Impervious Area (SF)	4% Required Treatment Area (SF)	Provided Treatment Area (SF)	Provided Treatment Area (SF)
1	15,337	613	687	4.5%
2	22,396	896	2,237	10.0%
3	8,550	342	6859	7.7%

The proposed biofiltration treatment areas will provide six (6) inches of surface ponding, 24 inches of planting media, and 12 inches of gravel. The effective depth of storage to the top of the ponded area in the biofiltration treatment area is 1.5 feet, providing a total volume of 5,375 cubic feet. From the geotechnical report, infiltration is not feasible on the site. The proposed biofiltration treatment areas will have underdrains to convey the treated stormwater into the existing County of Santa Cruz storm drain system.

To comply with Performance Requirement #3, the project must retain the 95th percentile event from impervious areas. The project replaces an existing development and qualifies for a 50% reduction for being a redevelopment project.

Table 3a. Runoff Retention

BMP	Total Area (SF) A	Pervious Area (SF) P	New Impervious (SF) In	Replaced Impervious Area (SF) Ir	Retention Tributary Area (SF) RTA = In+0.5Ir
1	17,185	2,053	13,234	1,898	14,183
2	26,259	4,292	0	21,967	10,984
3	12,667	4,574	0	8,093	4,047

Table 3b. Runoff Retention

BMP	Retention Tributary Area (SF) RTA	i (fraction Impervious Area) (In+Ir)/A	C**	95^{th} Percentile Rainfall depth (in) I	Retention Volume Required (CF) C*I*RTA	Retention Volume Provided (CF)
1*	14,183	0.88	0.70	2	1,661	1,031
2*	10,984	0.84	0.64	2	1,179	3,356
3	4,047	0.64	0.44	2	297	989

*BMP 1 and BMP 2 will be connected by an equalizer pipe and will act as one basin.

** C = 0.858*i3-0.78i2+0.774i+0.04

Table 3b. shows that the proposed project meets the required volume for the 95^{th} Percentile Rainfall. Due to the unsuitability of existing soil for infiltration, the majority of volume on-site will be detained in the biofiltration systems.

To comply with Performance Requirement #4, the proposed project is required to manage post-development peak flows such that the discharges from the site do not exceed the pre-project peak flows for the 2- through 10-year storm events. Since the project replaces a partially developed site, there is some increase in impervious area, and drainage patterns remain relatively similar, there is a small increase in peak flows due to the development (Table 4 & 5).

Table 4. Existing and Proposed Peak Flows

DMA	Runoff Coefficient	Area (SF)		Peak Flow (CFS)	
		Pervious	Impervious	2-Year	10-Year
Existing Conditions					
Area A1	0.53	0.50	0.57	0.69	1.07
Area A2	0.84	0.02	0.19	0.21	0.33
Total		**0.52**	**0.76**	**0.90**	**1.41**
Proposed Conditions					
Area B1	0.80	0.05	0.35	0.39	0.60
Area B2	0.79	0.08	0.50	0.56	0.87
Area B3	0.77	0.03	0.19	0.21	0.32
Area B4	0.21	0.02	0.00	0.01	0.01
Area B5	0.10	0.01	0.00	0.00	0.00
Area B6	0.10	0.06	0.00	0.01	0.01
Area B7	0.90	0.00	0.00	0.00	0.00
Total		**0.25**	**1.04**	**1.17**	**1.82**

Table 5. Existing and Proposed Peak Flows

Area Tributary To	Runoff Coefficient	Area (SF)		Peak Flow (CFS)	
		Pervious	Impervious	2-Year	10-Year
Undeveloped Conditions					
A1	0.53	0.50	0.57	0.69	1.07
A2	0.84	0.02	0.19	0.21	0.33
Total		**0.52**	**0.76**	**0.90**	**1.41**
Proposed Conditions					
BMP 1	0.80	0.05	0.35	0.39	0.60
BMP 2	0.77	0.10	0.50	0.56	0.88
BMP 3	0.61	0.11	0.19	0.22	0.34
Total		**0.25**	**1.03**	**1.17**	**1.82**

Detention volumes were determined using the County of Santa Cruz's Runoff Detention by the Modified Rational Method. The SWM-17 spreadsheet was utilized and are attached in Appendix C. The detention volumes were calculated to limit the 25-year post-development flow to 10-year pre-development flow. See Table 8 below. The table below shows the total required detention (SWM-17) and required Retention: 3,919 CF. The table below shows the total provided volume: 5,375 CF. Therefore, there is enough storage for both the retention and detention.

Table 6. 25 Year Post Development Detention Storage Volume

Area Tributary To	**Required per SWM-17 Volume (CF)**	**Required Retention Volume (CF)**	**Provided Volume (CF)**
BMP 1*	320	1,661	1,031
BMP 2*	428	1,179	3,356
BMP 3	35	279	989
Total	**783**	**3,136**	**5,375**

Self Treating areas are included in the peak flow calculation. To mitigate the proposed peak flow to the undeveloped peak flow, the project includes caps on the underdrains of each of the three biofiltration treatment basins with a circular orifice drilled into the plate capping the underdrain to meter flow out to the pre-developed condition. The overflow weir height of the outflow structure is the height between the underdrain and the overflow riser. Reference Appendix A – Figure 3 for Orifice Sizing.

Table 7. Peak Flows Mitigation

Area Tributary To	**Orifice Diameter (in)**	**Orifice Area (cfs)**	**Orifice Flow (cfs)**
BMP 1*	5	0.14	0.8
BMP 2*			
BMP 3	2.5	0.03	0.21

*BMP 1 and 2 will act as one basin, therefore only 1 orifice is designed for the two basins.

Descriptions of each Drainage Management Area

IV.A.1. Table of Drainage Management Areas

See C4.0 Appendix B.
IV.B.

IV.B. Tabulation and Sizing Calculations

IV.B.1. Information Summary for LID Facility Design

Total Project Area (Acres)	1.29 acres
Design Storm Depth	2" (Per 95th Percentile Rainfall Data, see Appendix for Reference)
Applicable Requirements	Performance Requirement No. 4: Peak Management

IV.C. Self-Treating Areas

Table 8. Self-Treating Areas

DMA	Description	Size (SF)
1A	Landscape Island	1,843
1B	Landscape Island	210
2A	Landscape Island	3,518
3A	Landscape Island	1,499
4A	Landscape Island	744
5	Landscape Island	2,600
6	Landscape Island	475

Areas Draining to Biofiltration treatment Facilities

Table 11. Areas Draining to Biofiltration treatment Facilities

DMA	Description	Type	Size (SF)	C Factor	Sizing Factor	Treatment Area Required
1A	Parking NW & Sidewalk	Impervious	10,854	1	0.05	543
1B	Parking N	Impervious	4,278	1	0.05	214
Total Required - BMP 1						757
Total Provided - BMP 1						687
2A	Parking Lot NE & Building & Drive Thru	Impervious	12,739	1	0.05	637
2B	Building & Drive-Thru	Impervious	9,035	1	0.05	452
Total Required - BMP 2						1,089
Total Provided - BMP 2						2,237
3A	Building & Parking SW	Impervious	6,890	1	0.05	345
3B	South Driveway (Off-Site)	Impervious	1,203	1	0.05	60
Total Required - BMP 3						405
Total Provided - BMP 3						659

Note: BMP 1 and BMP 2 will act as one biofiltration treatment basin

Source Control Measures

IV.D. Site activities and potential sources of pollutants

IV.E. Source Control Table

Table 12. Source Control Table

Potential source of runoff pollutants	Permanent source control BMPs	Operational source control BMPs
On-site storm drain inlets (unauthorized non-stormwater discharges and accidental spills or leaks)	• Mark all inlets with the words "No Dumping! Flows to Bay" or similar.	• Maintain and periodically repaint or replace inlet markings. • Provide stormwater pollution prevention information to new site owners, lessees, or operators. • See applicable operational BMPs in

		Fact Sheet SC-44, "Drainage System Maintenance," in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com • Include the following in lease agreements: "Tenant shall not allow anyone to discharge anything to storm drains or to store or deposit materials so as to create a potential discharge to storm drains."
Landscape/ Outdoor Pesticide Use/Building and Grounds Maintenance	• State that final landscape plans will accomplish all of the following. • Preserve existing native trees, shrubs, and ground cover to the maximum extent possible. • Design landscaping to minimize irrigation and runoff, to promote surface infiltration where appropriate, and to minimize the use of fertilizers and pesticides that can contribute to stormwater pollution. • Where landscaped areas are used to retain or detain stormwater, specify plants that are tolerant of saturated soil conditions. • Consider using pest-resistant plants, especially adjacent to hardscape. • To insure successful establishment, select plants appropriate to site soils, slopes, climate, sun, wind, rain, land use, air movement, ecological consistency, and plant interactions.	• Maintain landscaping using minimum or no pesticides. • See applicable operational BMPs in Fact Sheet SC-41, "Building and Grounds Maintenance," in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com • Provide IPM information to new owners, lessees and operators.
Refuse areas	• State how site refuse will be handled and provide supporting detail to what is shown on plans. • State that signs will be posted on or near dumpsters with the	• State how the following will be implemented: Provide adequate number of receptacles. Inspect receptacles regularly; repair or replace leaky receptacles. Keep receptacles covered. Prohibit/prevent dumping of liquid or

ATTACHMENT 7

	words "Do not dump hazardous materials here" or similar.	hazardous wastes. Post "no hazardous materials" signs. Inspect and pick up litter daily and clean up spills immediately. Keep spill control materials available on-site. See Fact Sheet SC-34, "Waste Handling and Disposal" in the CASQA Stormwater Quality Handbooks at www.cabmphandbooks.com
Fire Sprinkler Test Water	• Boiler drain lines shall be directly or indirectly connected to the sanitary sewer system and may not discharge to the storm drain system. • Condensate drain lines may discharge to landscaped areas if the flow is small enough that runoff will not occur. Condensate drain lines may not discharge to the storm drain system. • Rooftop equipment with potential to produce pollutants shall be roofed and/or have secondary containment. • Any drainage sumps on-site shall feature a sediment sump to reduce the quantity of sediment in pumped water. • Avoid roofing, gutters, and trim made of copper or other unprotected metals that may leach into runoff. • Include controls for other sources as specified by local reviewer.	
Sidewalks, and parking lots.		• Sweep plazas, sidewalks, and parking lots regularly to prevent accumulation of litter and debris. Collect debris from pressure washing to prevent entry into the storm drain system. Collect washwater containing any cleaning agent or degreaser and discharge to the sanitary sewer not to a storm drain.

Truck Loading Area		• Install "No idling" signs near the loading area. No maintenance or repair of the trucks at the loading area will be allowed. • Sweep loading area regularly to prevent accumulation of litter and debris. Collect debris from pressure washing to prevent entry into the storm drain system. Collect washwater containing any cleaning agent or degreaser and discharge to the sanitary sewer not to a storm drain.

IV.F. Features, Materials, and Methods of Construction of Source Control BMPs

See the improvement plans for the CVS Santa Cruz Project for the materials and methods of construction of the Source Control BMPs.

Additional Design and Construction Considerations for Biofiltration treatment Basins

- Sediment controls and fencing should be installed to prevent clogging and compaction of engineered and existing site soils during construction.
- Whenever possible, avoid the use of heavy equipment during construction on areas where biofiltration treatment systems are to be installed. If soils are compacted, additional ripping may be necessary to re-establish soil permeability.
- After basin excavation, do not compact the native underlying soils.
- When installing the engineered soil mix, drop it from the bucket and do not compact it.

V. Stormwater Facility Maintenance

V.A. Ownership and Responsibility for Maintenance in Perpetuity

This Stormwater Control will provide the Ownership and Responsibility for Maintenance in Perpetuity at a later date.

V.B. Summary of Maintenance Requirements for Each Stormwater Facility

Biofiltration treatment Basins

Inspection and Maintenance

Primary maintenance activities include vegetation management and sediment removal. Mosquito control is also a concern in extended detention basins that are designed to include pools of standing water. The typical maintenance requirements include:

- Conduct semi-annual inspection as follows:
  - Evaluate the health of the vegetation and remove and replace any dead or dying plants.

ATTACHMENT 4

  - Remove any trash and debris.
  - Inspect the outlet, embankments, dikes, berms, and side slopes for structural integrity and signs of erosion or rodent burrows. Fill in any holes detected in the side slopes.
  - Examine outlets and overflow structures and remove any debris plugging the outlets.
  - Identify and minimize any sources of sediment and debris. Check rocks or other erosion control and replace, if necessary.
  - Check inlets to make sure piping is intact and not plugged. Remove accumulated sediment and debris near the inlet. Ensure that engineered energy dissipation is functioning adequately by checking for evidence of local scour around the inlet.
  - Inspect for standing water and correct any problems that prevent the extended detention basin from draining as designed.
  - Confirm that any fences around the facility are secure.
- Maintenance activities at the bottom of the basin shall NOT be performed with heavy equipment, which would compact the soil and limit infiltration.
- Harvest vegetation annually, during the summer.
- Trim vegetation at beginning and end of the wet season and inspect monthly to prevent establishment of woody vegetation and for aesthetic and mosquito control reasons.
- Invasive vegetation contributing up to 25% of vegetation of all species shall be removed and replaced.
- Dead vegetation shall be removed to maintain less than 10% of area coverage or when vegetative filter strip function is impaired. Vegetation shall be replaced immediately to control erosion where soils are exposed and within 3 months to maintain cover density.
- Avoid the use of pesticides and quick release synthetic fertilizers, and follow the principles of integrated pest management (IPM). Check with the local jurisdiction for any local policies regarding the use of pesticides and fertilizers.
- Remove sediment from the forebay when the sediment level reaches the level shown on the fixed vertical sediment marker.
- Remove accumulated sediment and regrade about every 10 years or when the accumulated sediment volume exceeds 10 % of the basin volume.

Storm drain clean out structures and pipe layout can be found in the plan set. Storm drain facilities should be cleaned out once a year.

VI. Construction Checklist

Table 13. Construction Plan C.3 Checklist

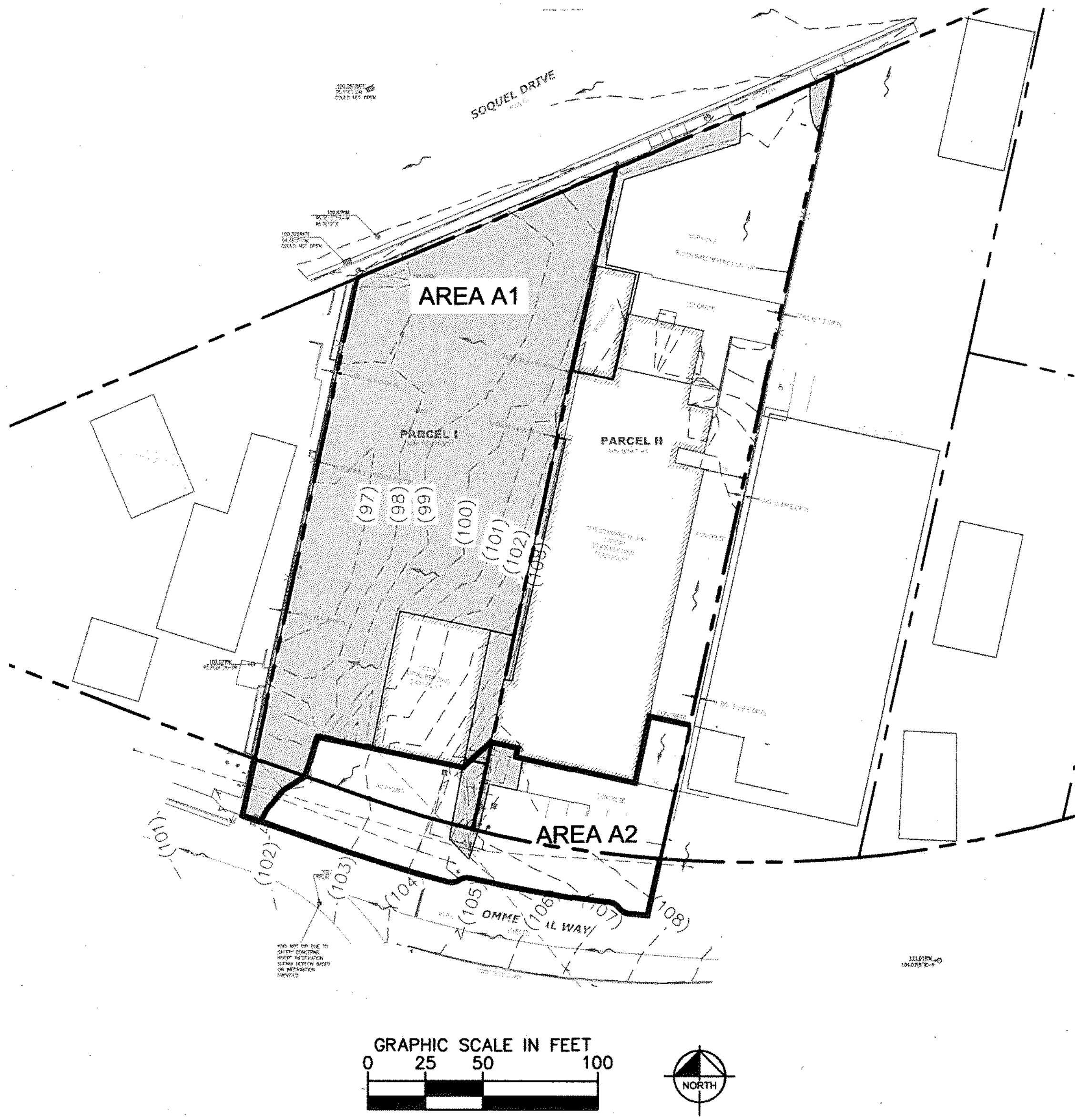
Stormwater Control Plan Page #	BMP Description	See Plan Sheet #s
C4.0	Biofiltration treatment Area 1	C4.0
C4.0	Biofiltration treatment Area 2	C4.0
C4.0	Biofiltration treatment Area 3	C4.0

Plan Sheet C4.0 is shown in Appendix B.

VII. Certifications

The preliminary design of stormwater treatment facilities and other stormwater pollution control measures in this plan are in accordance with the current edition of the Santa Cruz County Design Criteria.

Appendix A. Figures



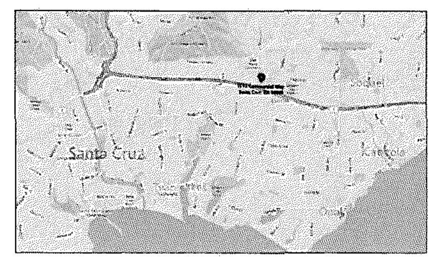
LEGEND

PROJECT BOUNDARY

EXISTING FLOW ARROW

EXISTING CONDITIONS PERVIOUS AREA

(108) EXISTING CONTOUR



VICINITY MAP
NTS

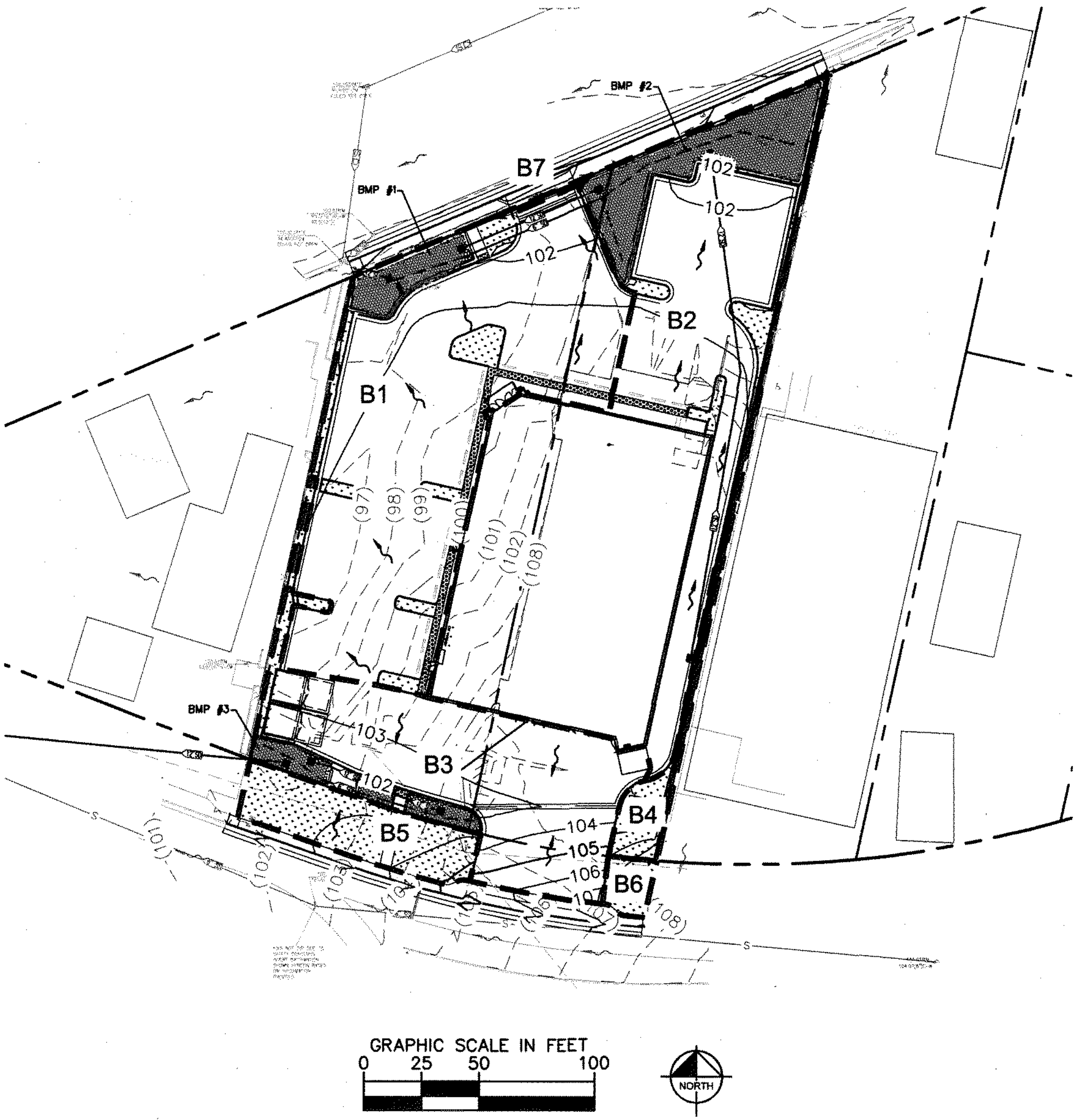
NORTH

FIGURE 1 - EXISTING DRAINAGE
CVS SANTA CRUZ

Kimley»Horn ©2019

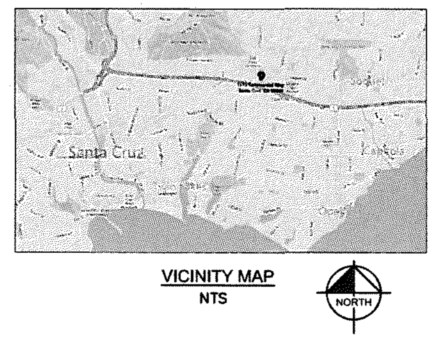
K:\SAC_LDEV\CVS\097175026 CVS Santa Cruz 105634 (Ground Up)\08 CADD\Exhibits\Drainage_1_FIGURE 1_EXISTING CONDITIONS.dwg

ATTACHMENT 7



LEGEND

— — —	PROJECT BOUNDARY
—— 104 ——	PROPOSED CONTOUR
	PROPOSED FLOW ARROW
— — (108) — —	EXISTING CONTOUR
	EXISTING FLOW ARROW



VICINITY MAP
NTS

FIGURE 2 - PROPOSED DRAINAGE
CVS SANTA CRUZ
Kimley»Horn ©2019

K:\SAC_LDEV\CVS\097175028 CVS Santa Cruz 105634 (Ground Up)\08 CADD\Exhibits\Drainage_2_FIGURE 2_PROPOSED CONDITIONS.dwg

ATTACHMENT

Calculated by AMB Date 3/11/2019
Checked by SKB Date 3/11/2019

CVS Santa Cruz Development Project
PR#4 Peak Management
Post- development peak flows, discharged from the site, shall not exceed pre-project
*Assuming Pre-Project Conditions
BMP 1 & BMP 2

Orifice Diameter (in)	Orifice Diameter (ft)	Orifice Area (ft^2)	Effective Depth of Ponding/Height to Overflow (ft)	Existing 2-Year Peak Flow
5.0	0.42	0.14	1.5	0.7

Orifice

$Q=C_DA(2gh)^{0.5}$	0.8	cfs

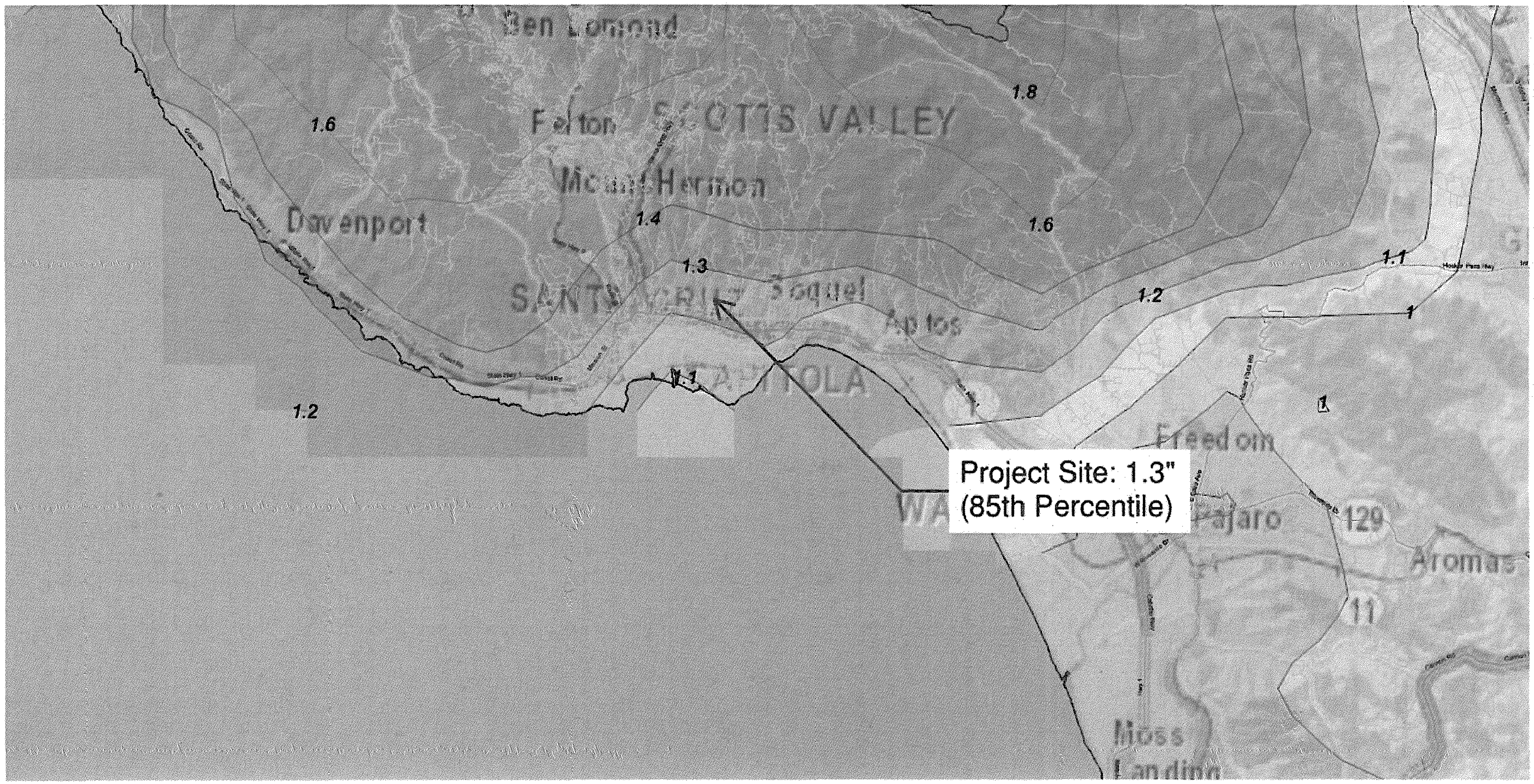
BMP 3

Orifice Diameter (in)	Orifice Diameter (ft)	Orifice Area (ft^2)	Effective Depth of Ponding/Height to Overflow (ft)	Existing 2-Year Peak Flow
2.5	0.21	0.03	1.5	0.21

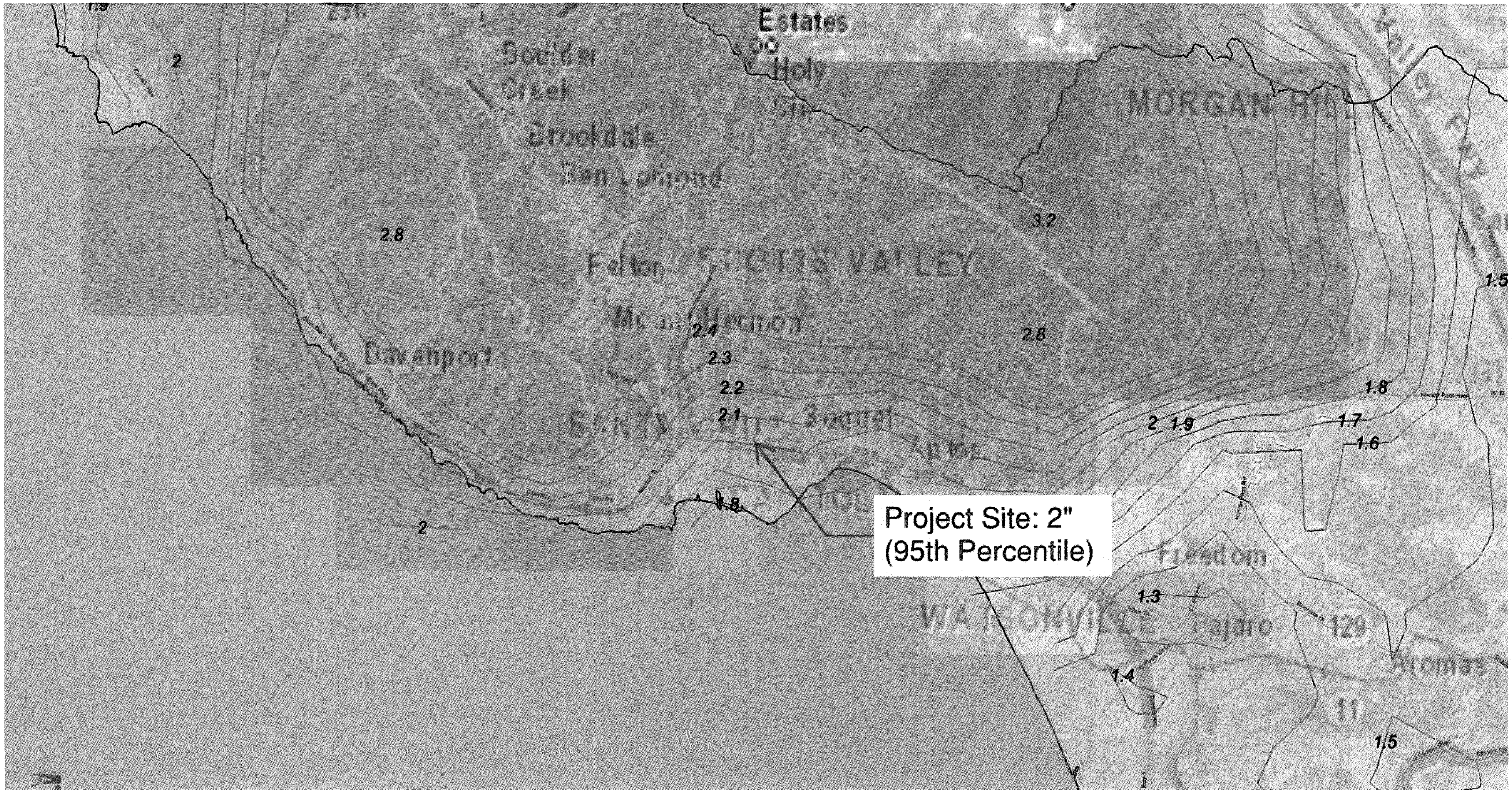
Orifice

$Q=C_DA(2gh)^{0.5}$	0.21	cfs

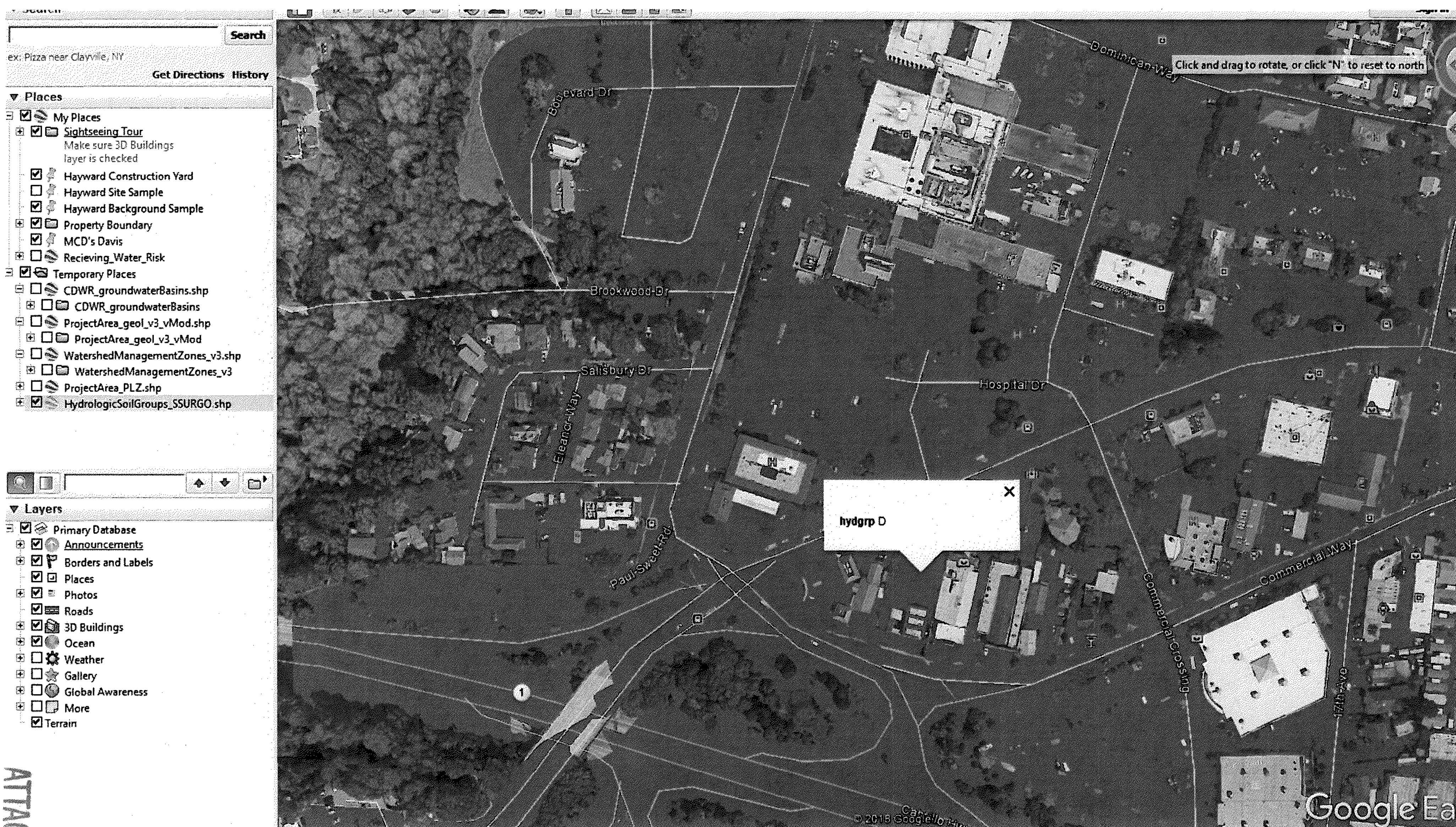
ATTACHMENT 7


Ben Lomond
1.8
1.6
Felton
SCOTTS VALLEY
Mount Hermon
Davenport
1.4
1.6
1.3
1.1
Soquel
1.2
1
Aptos
1.1
1.2
1
Freedom
Project Site: 1.3"
(85th Percentile)
Pajaro
129
Aromas
11
Moss
Landing

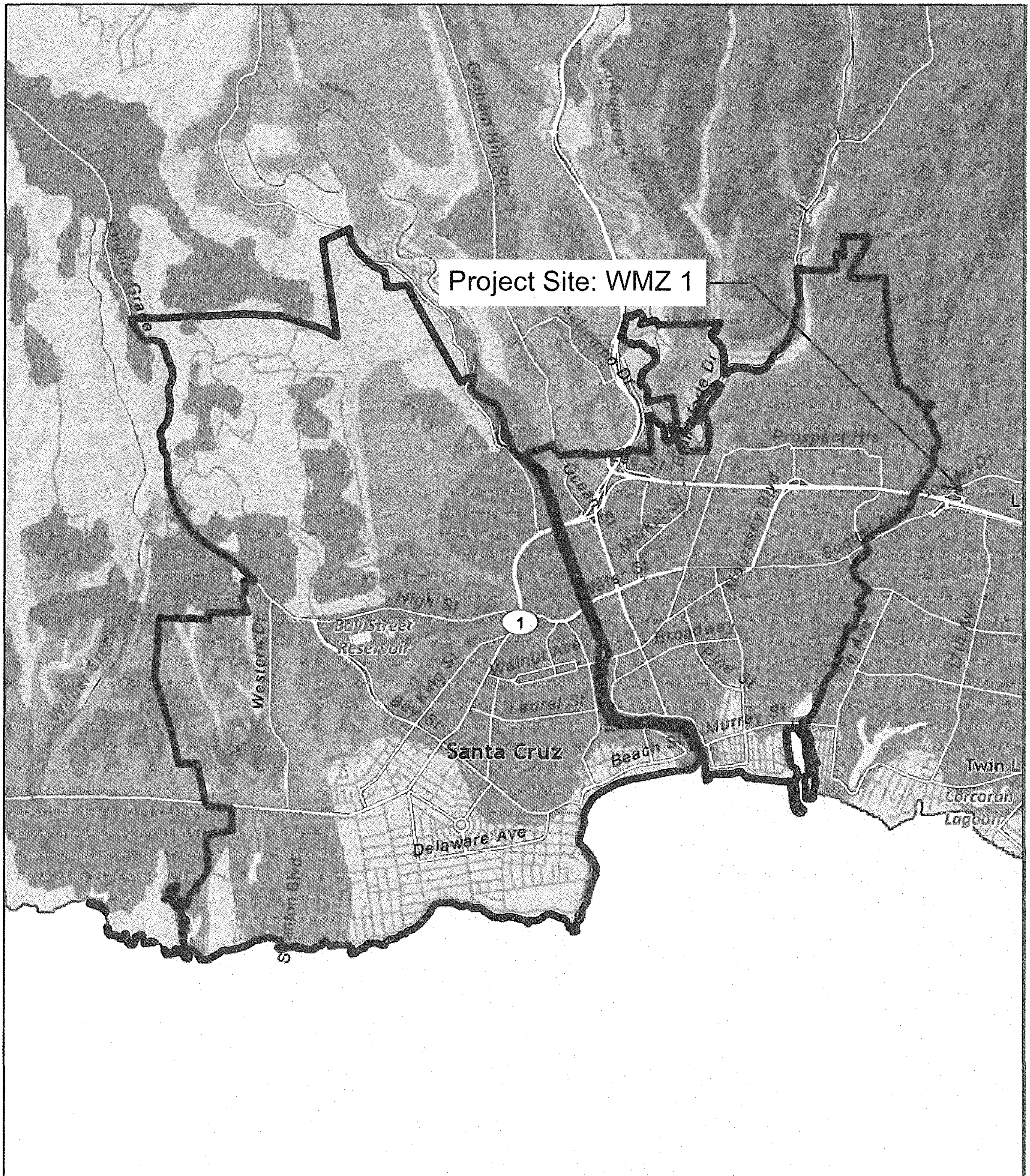
ATTACHMENT



ATTACHMENT 7



ATTACHMENT 7



CENTRAL COAST JOINT EFFORT — **Santa Cruz, California**

Watershed management zones

1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Urban area boundary

Data sources
Watershed management zones: Stillwater Sciences, 2012
Base data: ESRI 2010

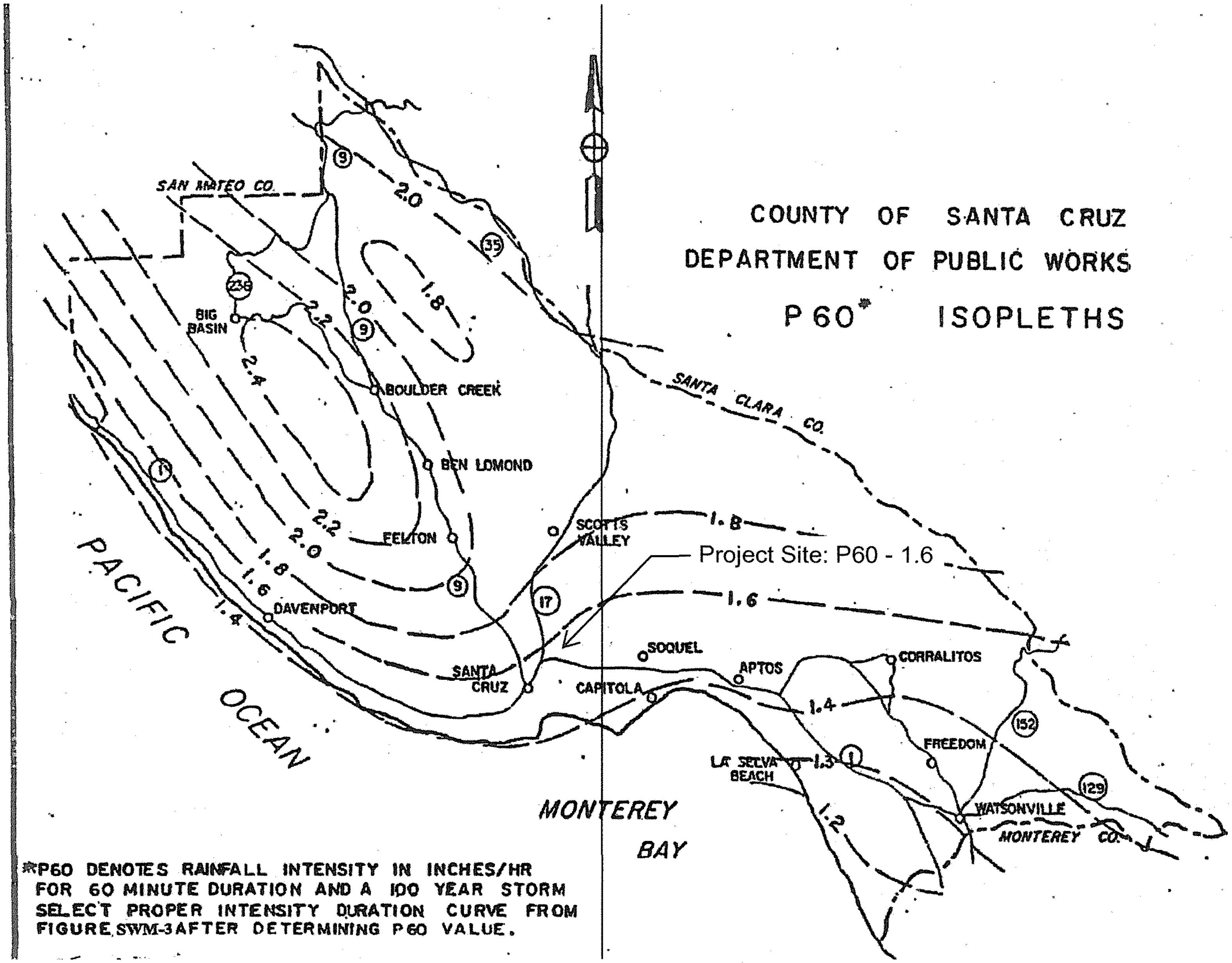
0 0.25 0.5 1 Miles
0 0.5 1 2 km

N

ATTACHMENT 7

Stillwater Sciences
www.stillwatersci.com

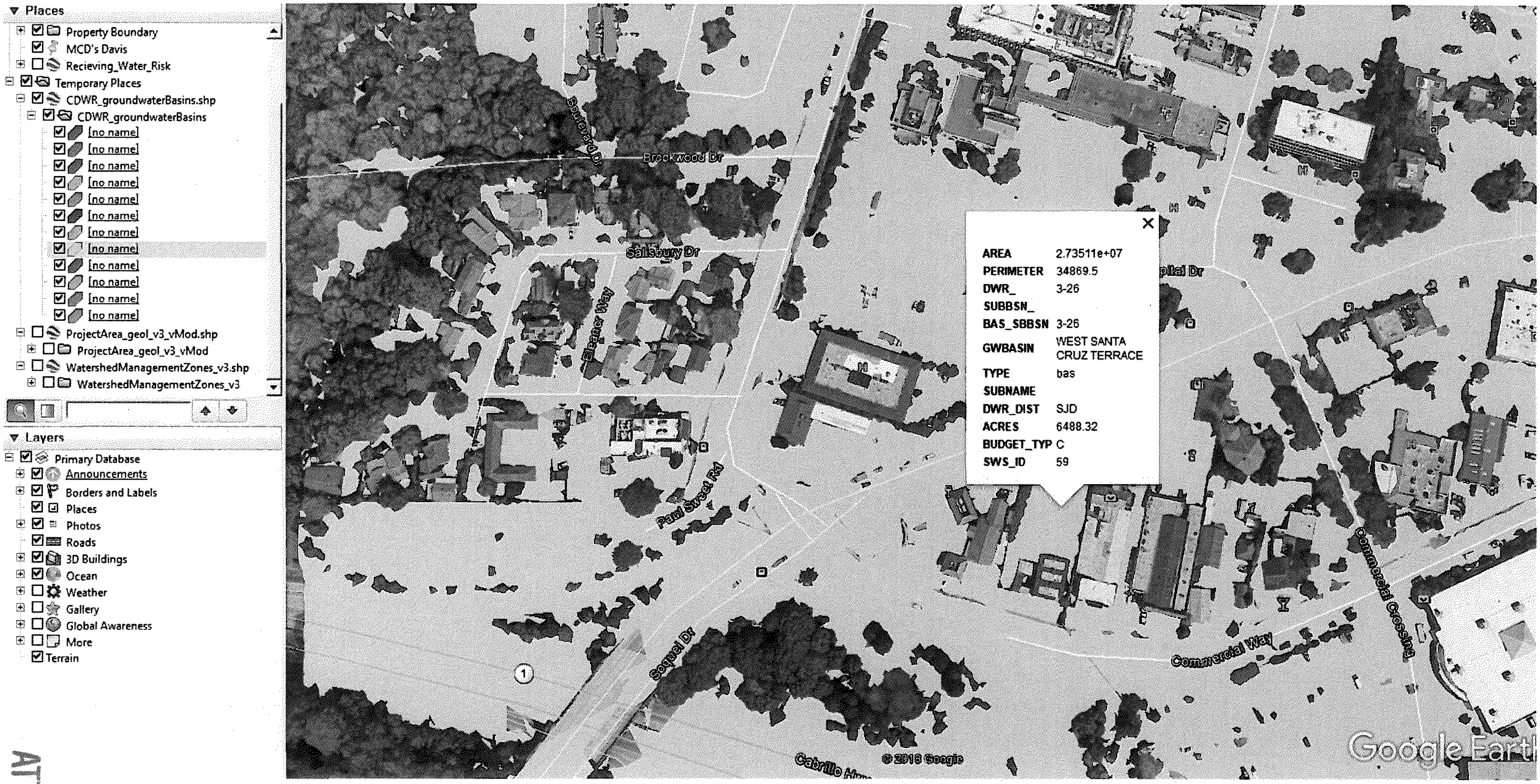
Figure SWM-2: Rainfall Intensity Isopleths



*P60 DENOTES RAINFALL INTENSITY IN INCHES/HR FOR 60 MINUTE DURATION AND A 100 YEAR STORM SELECT PROPER INTENSITY DURATION CURVE FROM FIGURE SWM-3 AFTER DETERMINING P60 VALUE.

FIG. SWM-2

ATTACHMENT 7



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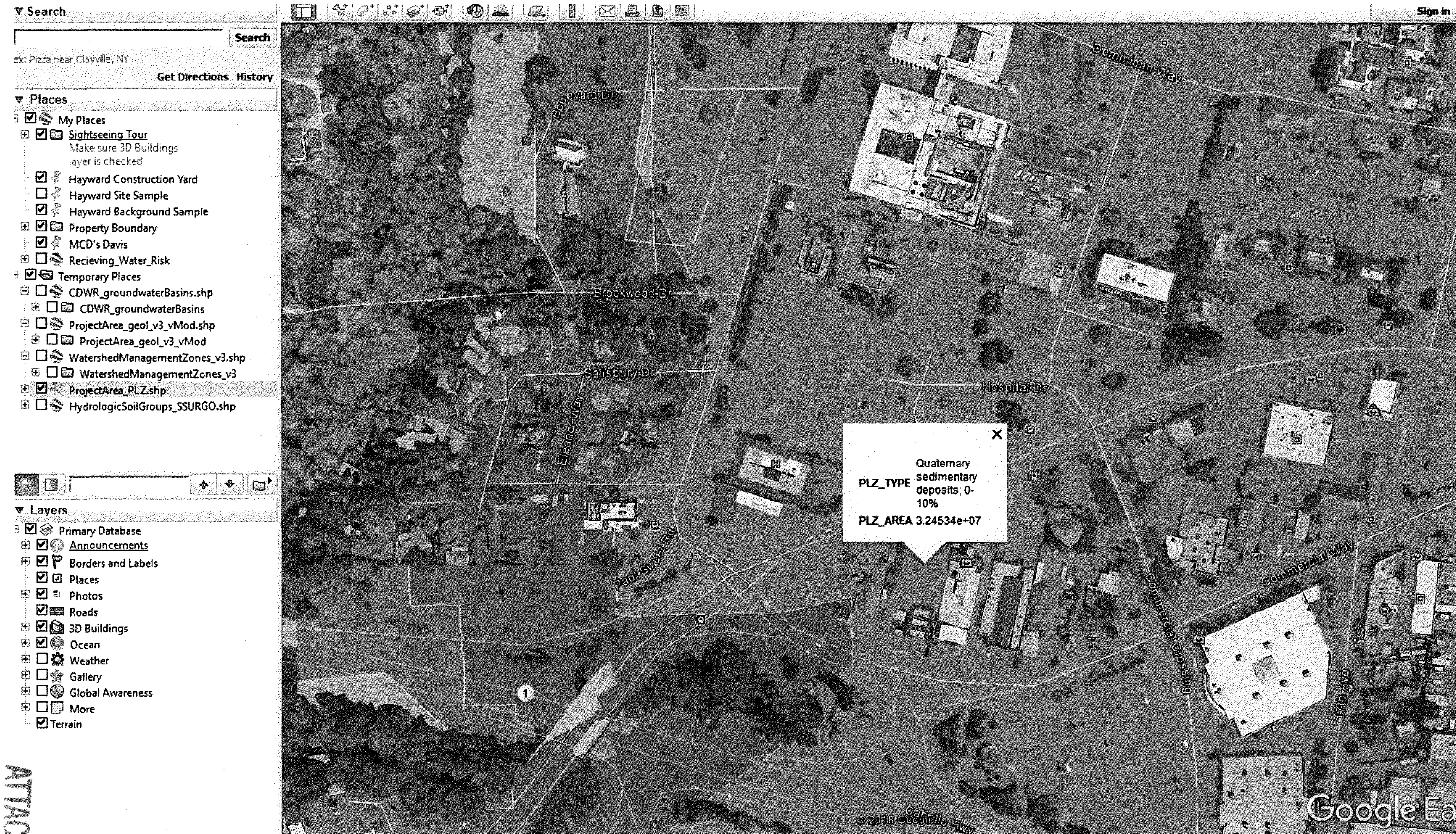

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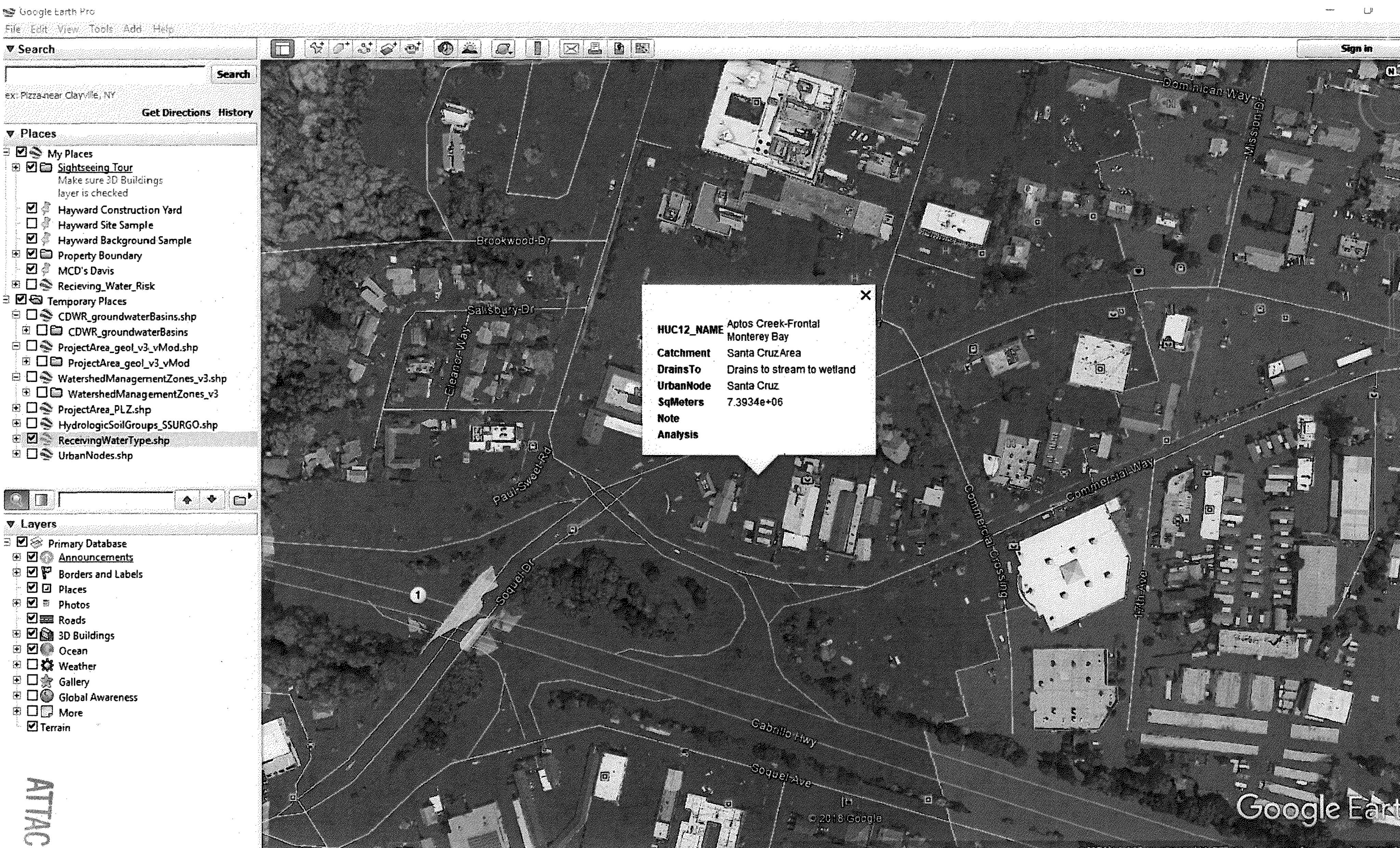
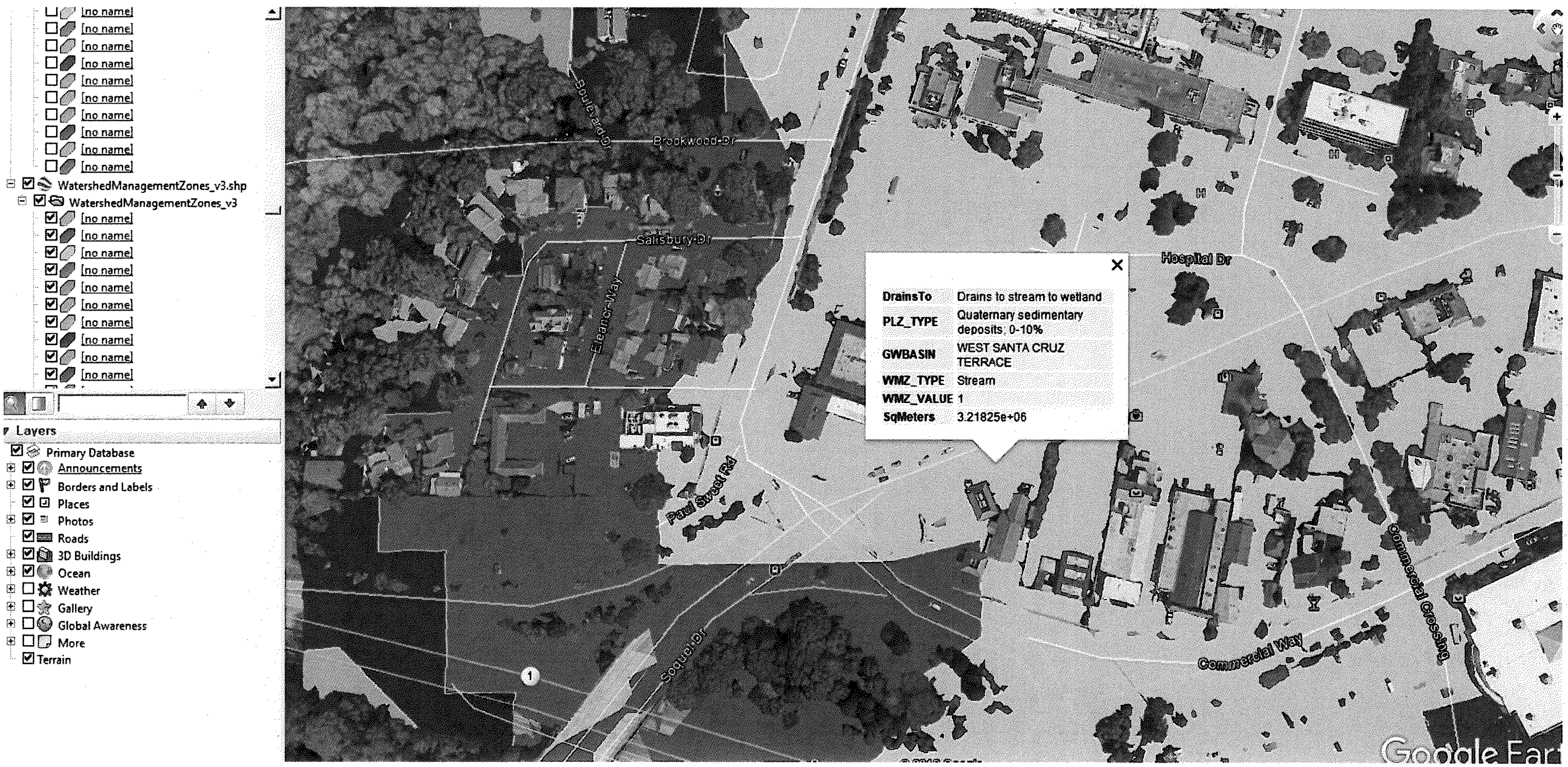


ATTACHMENT 7


Search
ex: Pizza near Clayville, NY
Get Directions History
Places
My Places
Sightseeing Tour
Make sure 3D Buildings layer is checked
Hayward Construction Yard
Hayward Site Sample
Hayward Background Sample
Property Boundary
MCD's Davis
Recieving_Water_Risk
Temporary Places
CDWR_groundwaterBasins.shp
CDWR_groundwaterBasins
ProjectArea_geol_v3_vMod.shp
ProjectArea_geol_v3_vMod
WatershedManagementZones_v3.shp
WatershedManagementZones_v3
ProjectArea_PLZ.shp
HydrologicSoilGroups_SSURGO.shp
ReceivingWaterType.shp
UrbanNodes.shp
Layers
Primary Database
Announcements
Borders and Labels
Places
Photos
Roads
3D Buildings
Ocean
Weather
Gallery
Global Awareness
More
Terrain
Click and drag to rotate, or click "N" to reset to north
Brookwood Dr
Salisbury Dr
Eleanor Way
Paul Sweet Rd
Soquel Dr
Commercial Crossing
Commercial Way
17th Ave
Mission
Cabrillo Hwy
Soquel Ave
NAME Live Oak
NAMELSAD Live Oak CDP
geol_lyr 1:62,500
GeolSrc USGS Santa Cruz County, 1:62,500
Scale 62500
© 2018 Google
Google Earth

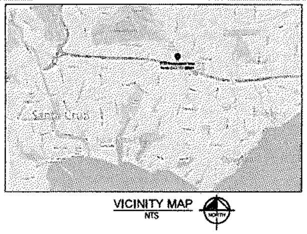
ATTACHMENT



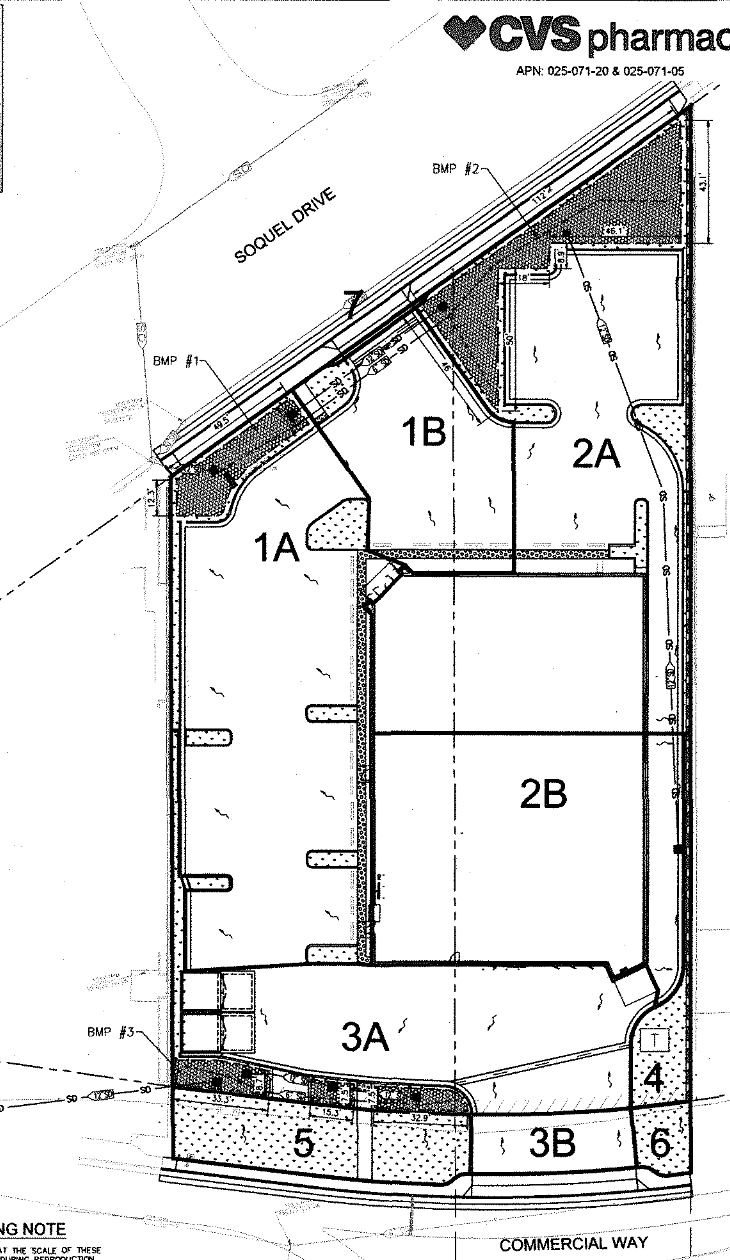
ATTACHMENT 7

Appendix B. Stormwater Control Plan

ATTACHMENT 7

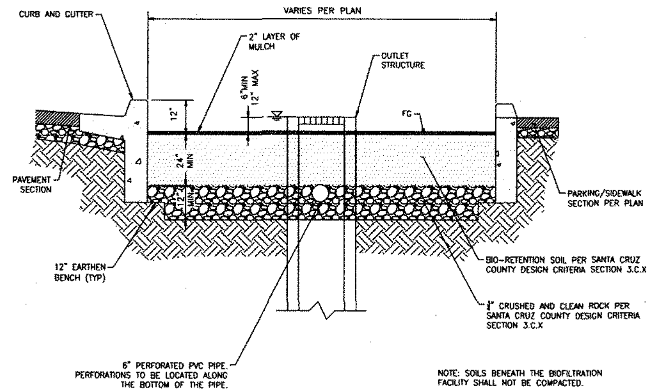


DIRECTIONS: EXIT HIGHWAY 1. GO EAST ON SOQUEL DRIVE. THE PROJECT SITE IS LOCATED ON THE SOUTH SIDE OF SOQUEL DR BETWEEN COMMERCIAL WAY AND COMMERCIAL CROSSING



LEGEND

- PROPERTY BOUNDARY
- SETBACK
- RETAINING WALL
- BIORETENTION AREA WITH IMPERMEABLE LINEAR
- DMA AREA
- LANDSCAPE
- FLOW DIRECTION



NOTE: SOILS BENEATH THE BIOFILTRATION FACILITY SHALL NOT BE COMPACTED.

BIOFILTRATION SYSTEM (1)
N.T.S.

DRAINAGE MANAGEMENT AREAS

DMA	Description	Pervious Area (SF)	Impervious Area (SF)	Sizing Factor	Treatment Area Required	Directed to:
1A	Parking NW & E	1,843	10,854	0.05	543	BMP 1
1B	Parking N	210	4,278	0.05	214	BMP 1
2A	Parking NE & Building & Drive-Thru	3,518	12,739	0.05	637	BMP 2
2B	Building & Drive-Thru	0	9,035	0.05	452	BMP 2
3A	Building & Parking SW	1,499	6,890	0.05	345	BMP 3
3B	South Driveway (Off-Site)	0	1,203	0.05	60	BMP 3
4	East Landscape	774	193	0.05	10	Self-Treating
5	South Landscape (Off-Site)	2,600	0	0.05	0	Self-Treating
6	East Landscape (Off-Site)	475	0	0.05	0	Self-Treating
7	North Driveway	0	71	0.05	4	Self-Treating

PR#2 - WATER QUALITY TREATMENT

BMP	Effective Impervious Area (SF)	4% Required Treatment Area (SF)	Provided Treatment Area (SF)	Provided Treatment Area (%)
BMP 1*	15,337	613	687	4.5%
BMP 2*	22,396	896	2,237	10.0%
BMP 3	8,550	342	659	7.7%

PR#3 - RUNOFF RETENTION

BMP	Total Area (SF) A	Pervious Area (SF) P	New Impervious	Replaced Impervious Area (SF) Ir	Retention Tributary Area (SF) RTA=In+0.5Ir
BMP 1*	17,185	2,053	13,234	1,898	14,183
BMP 2*	26,259	4,292	0	21,967	10,984
BMP 3	12,667	4,574	0	8,093	4,047

BMP	Retention Tributary Area (SF) RTA	I (Fraction of Impervious Area) (In+Ir)/A	C **	95th Percentile Rainfall Depth (in) I	Retention Required (CF) C*I*RTA	Retention Provided (CF)
BMP 1*	14,183	0.88	0.70	2	1,661	1,031
BMP 2*	10,984	0.84	0.64	2	1,179	3,356
BMP 3	4,047	0.64	0.44	2	297	989
				Total	3,136	5,375

PR#4 - PEAK MANAGEMENT

POST-DEVELOPMENT PEAK FLOWS/DISCHARGE FROM THE SITE SHALL NOT EXCEED PRE-PROJECT PEAK FLOWS FOR THE 2-THROUGH 10-YEAR STORM EVENTS.

DMA	Runoff Coefficient	Area (SF)		Peak Flow (CFS)	
		Pervious	Impervious	2-Year	10-Year
Existing Conditions					
Area A1	0.53	0.50	0.57	0.69	1.07
Area A2	0.84	0.02	0.19	0.21	0.33
Total		0.52	0.76	0.90	1.41
Proposed Conditions					
Area B1	0.80	0.05	0.35	0.39	0.60
Area B2	0.79	0.08	0.50	0.56	0.87
Area B3	0.77	0.03	0.19	0.21	0.32
Area B4	0.21	0.02	0.00	0.01	0.01
Area B5	0.10	0.01	0.00	0.00	0.00
Area B6	0.10	0.06	0.00	0.01	0.01
Area B7	0.90	0.00	0.00	0.00	0.00
Total		0.25	1.04	1.17	1.82

Area Tributary To	Runoff Coefficient	Area (SF)		Peak Flow (CFS)	
		Pervious	Impervious	2-Year	10-Year
Undeveloped Conditions					
A1	0.53	0.50	0.57	0.69	1.07
A2	0.84	0.02	0.19	0.21	0.33
Total		0.52	0.76	0.90	1.41
Proposed Conditions					
BMP 1	0.80	0.05	0.35	0.39	0.60
BMP 2	0.77	0.10	0.50	0.56	0.88
BMP 3	0.61	0.11	0.19	0.22	0.34
Total		0.25	1.03	1.17	1.82

NORTH

GRAPHIC SCALE IN FEET
0 10 20 40

U.S.A. NOTIFICATION SERVICE

ELECTRONIC DRAWING NOTE

ATTENTION IS DRAWN TO THE FACT THAT THE SCALE OF THESE DRAWINGS MAY HAVE BEEN DISTORTED DURING REPRODUCTION PROCESSES. USERS OF THESE DRAWINGS ARE ADVISED TO USE PAPER COPIES OF THE PLANS.

User = Beltran, Amelia (Whittaker)
Date = Tuesday, March 05, 2019 13:41:18
Filename = K:\SAC_LDEV\CVS\097175026 CVS Santa Cruz 105634 (Ground Up)\08 CADD\Plan Sheets\Planning Documents\C4.0 STORMWATER CONTROL PLAN.dwg

ATTACHMENT

CVS pharmacy™
13,111 - RIGHT CORNER BUMPOUT
STORE NUMBER: NEW STORE
APN: 025-071-20 & 025-071-05
(SEC) COMMERCIAL WAY & SOQUEL DRIVE
SANTA CRUZ, CA
PROJECT TYPE: NEW CONSTRUCTION
DEAL TYPE: FEE FOR SERVICE
CS PROJECT NUMBER: 105634

ARCHITECT:
wd
19100 VON KARMAN AVE.
SUITE 600
IRVINE, CA 92612
949.753.7676 T
WDPARTNERS.COM

ENGINEER:
Kimley»Horn

555 CAPITOL MALL, SUITE 300
SACRAMENTO, CA 95814
PHONE: 916-858-5800
FAX: 916-608-5805
WWW.KIMLEY-HORN.COM

DEVELOPER:
BOOS DEVELOPMENT WEST
(916) 346-4797
2020 L STREET, SUITE 245
SACRAMENTO, CA 95811

SEAL:
REGISTERED PROFESSIONAL ENGINEER
SHEETAL K. BHATT
No. C76927
CIVIL
STATE OF CALIFORNIA

REVISIONS:
3/11/2019 Planning Resubmittal 1

DRAWING BY: AMB
DATE: 11/15/2018
JOB NUMBER: 097175026
TITLE: STORMWATER CONTROL PLAN
SHEET NUMBER: C4.0
COMMENTS: NOT RELEASED FOR CONSTRUCTION

Kimley»Horn

Appendix C. Runoff Detention By Modified Rational Method

ATTACHMENT 7

PROJECT: CVS Santa Cruz - APN: 025-071-05 BMP 1 App: 18-1000 **Calc by:** AMB **Date:** 3/8/2019

RUNOFF DETENTION BY THE MODIFIED RATIONAL METHOD

Data Entry: PRESS TAB & ENTER DESIGN VALUES SS Ver: 1.0

Site Location P60 Isopleth:	1.60		Fig. SWM-2 in County Design Criteria
Rational Coefficients Cpre:	0.53		See note # 2
Cpost:	0.80		See note # 2
Impervious Area:	15337	ft^2	See note # 2 and # 4

STRUCTURE DIMENSIONS FOR DETENTION

320 ft^3 storage volume calculated

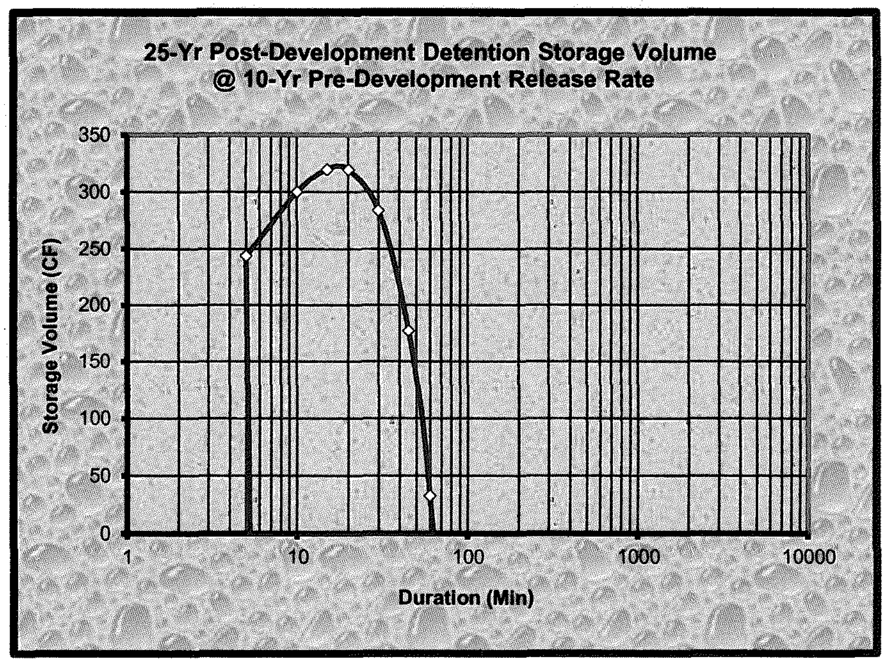
100 % void space assumed

320 ft^3 excavated volume needed

Structure	Length	Width*	Depth*
Ratios	100.00	10.00	5.00
Dimen. (ft)	**39.99**	**4.00**	**2.00**

*For pipe, use the square root of the sectional area

25 - YEAR DESIGN STORM				DETENTION @ 15 MIN.	
Storm Duration (min)	25 - Year Intensity (in/hr)	10 - Yr. Release Qpre (cfs)	25 - Year Qpost (cfs)	Detention Rate To Storage (cfs)	Specified Storage Volume (cf)
1440	0.34	0.053	0.096	-0.254	-27461
1200	0.36	0.057	0.103	-0.247	-22207
960	0.40	0.063	0.113	-0.237	-17045
720	0.45	0.071	0.128	-0.222	-12009
480	0.53	0.083	0.151	-0.199	-7165
360	0.60	0.094	0.170	-0.180	-4858
240	0.71	0.111	0.201	-0.149	-2679
180	0.80	0.125	0.227	-0.123	-1665
120	0.94	0.148	0.268	-0.082	-737
90	1.06	0.167	0.302	-0.048	-324
60	1.26	0.197	0.357	0.007	33
45	1.42	0.222	0.403	0.052	177
30	1.68	0.263	0.476	0.126	283
20	1.98	0.311	0.563	0.213	319
15	2.23	**0.350**	0.634	0.284	320
10	2.64	0.414	0.750	0.400	300
5	3.52	0.552	0.999	0.649	243



Notes & Limitations on Use:

1) The modified rational method, and therefore the standard calculations are applicable in watersheds up to 20 acres in size.
2) Required detention volume determinations shall be based on all net new impervious area both on and off-site, resulting from the proposed project. Pervious areas shall not be included in detention volume sizing; an exception may be made for incidental pervious areas less than 10% of the total area.
3) Gravel packed detention chambers shall specify on the plans, aggregate that is washed, angular, and uniformly graded (of single size), assuring void space not less than 35%.
4) A map showing boundaries of both regulated impervious areas and actual drainage areas routed to the hydraulic control structure of the detention facility is to be provided, clearly distinguishing between the two areas, and noting the square footage.
5) The EPA defines a class V injection well as any bored, drilled, or driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system. Such storm water drainage wells are "authorized by rule". For more information on these rules, contact the EPA. A web site link is provided from the County DPW Stormwater Management web page.
6) Refer to the County of Santa Cruz Design Criteria, for complete method criteria.

ATTACHMENT 7

PROJECT: CVS Santa Cruz - APN: 025-071-05 BMP 2 App: 18-1000 **Calc by:** AMB **Date:** 3/8/2019

RUNOFF DETENTION BY THE MODIFIED RATIONAL METHOD

Data Entry: PRESS TAB & ENTER DESIGN VALUES SS Ver: 1.0

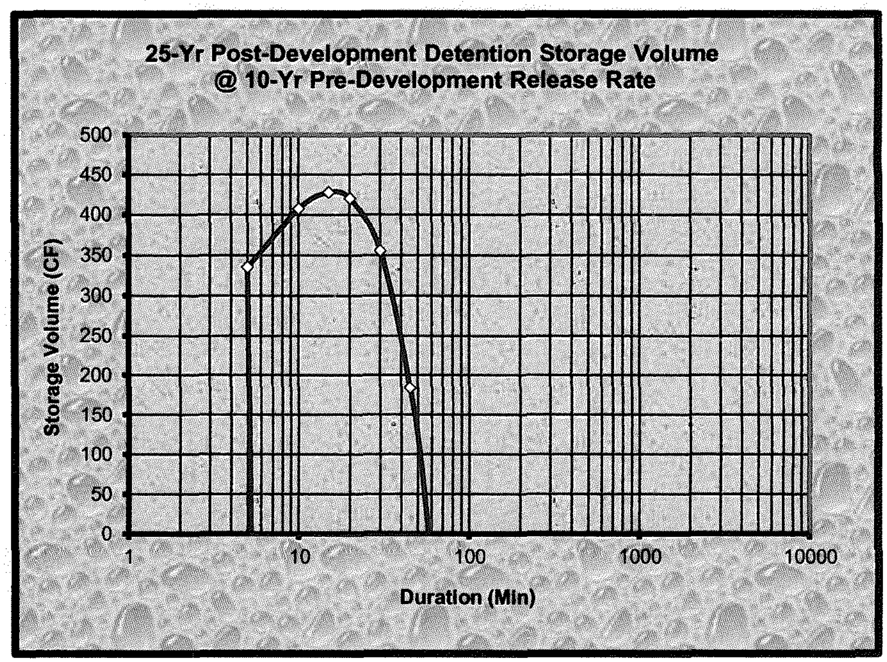
Site Location P60 Isopleth:	1.60		Fig. SWM-2 in County Design Criteria
Rational Coefficients Cpre:	0.53		See note # 2
Cpost:	0.77		See note # 2
Impervious Area:	22396	ft^2	See note # 2 and # 4

STRUCTURE DIMENSIONS FOR DETENTION

428	ft^3 storage volume calculated
100	% void space assumed
428	ft^3 excavated volume needed

Structure	Length	Width*	Depth*	
Ratios	25.00	2.00	2.00	*For pipe, use the square root of the sectional area
Dimen. (ft)	**40.58**	**3.25**	**3.25**	

25 - YEAR DESIGN STORM				DETENTION @ 15 MIN.	
Storm Duration (min)	25 - Year Intensity (in/hr)	10 - Yr. Release Qpre (cfs)	25 - Year Qpost (cfs)	Detention Rate To Storage (cfs)	Specified Storage Volume (cf)
1440	0.34	0.077	0.135	-0.377	-40668
1200	0.36	0.083	0.145	-0.366	-32938
960	0.40	0.091	0.160	-0.352	-25338
720	0.45	0.103	0.180	-0.332	-17914
480	0.53	0.122	0.213	-0.299	-10761
360	0.60	0.137	0.239	-0.272	-7346
240	0.71	0.162	0.283	-0.228	-4110
180	0.80	0.183	0.319	-0.193	-2599
120	0.94	0.216	0.377	-0.134	-1209
90	1.06	0.244	0.425	-0.087	-585
60	1.26	0.288	0.502	-0.009	-41
45	1.42	0.325	0.566	0.054	184
30	1.68	0.384	0.669	0.158	355
20	1.98	0.454	0.792	0.280	420
15	2.23	**0.511**	0.892	0.380	428
10	2.64	0.605	1.055	0.543	407
5	3.52	0.806	1.405	0.893	335



Notes & Limitations on Use:

1) The modified rational method, and therefore the standard calculations are applicable in watersheds up to 20 acres in size.
2) Required detention volume determinations shall be based on all net new impervious area both on and off-site, resulting from the proposed project. Pervious areas shall not be included in detention volume sizing; an exception may be made for incidental pervious areas less than 10% of the total area.
3) Gravel packed detention chambers shall specify on the plans, aggregate that is washed, angular, and uniformly graded (of single size), assuring void space not less than 35%.
4) A map showing boundaries of both regulated impervious areas and actual drainage areas routed to the hydraulic control structure of the detention facility is to be provided, clearly distinguishing between the two areas, and noting the square footage.
5) The EPA defines a class V injection well as any bored, drilled, or driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system. Such storm water drainage wells are "authorized by rule". For more information on these rules, contact the EPA. A web site link is provided from the County DPW Stormwater Management web page.
6) Refer to the County of Santa Cruz Design Criteria, for complete method criteria.

ATTACHMENT

PROJECT: CVS Santa Cruz - APN: 025-071-05 BMP 3 App: 18-1000 **Calc by:** AMB **Date:** 3/8/2019

RUNOFF DETENTION BY THE MODIFIED RATIONAL METHOD

Data Entry: PRESS TAB & ENTER DESIGN VALUES SS Ver: 1.0

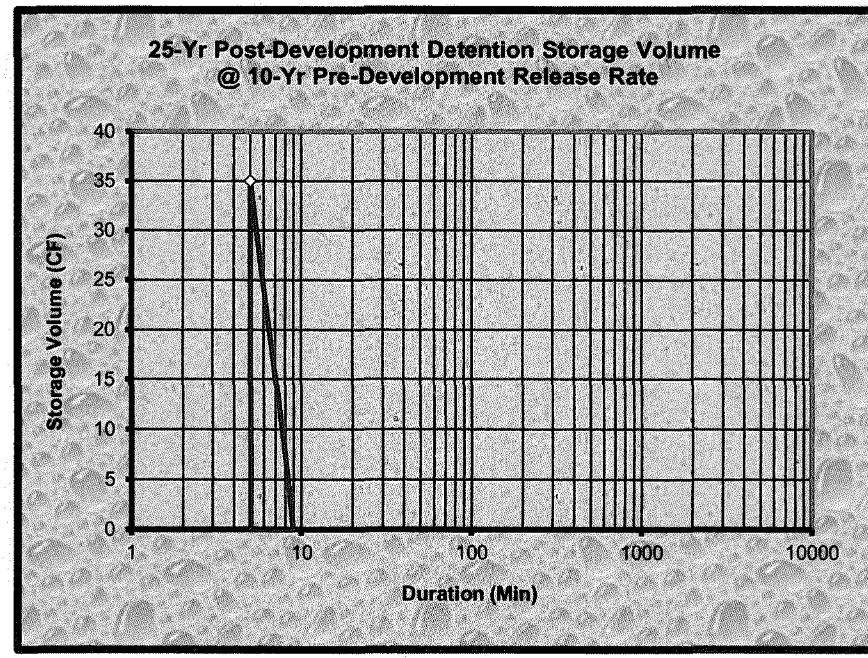
Site Location P60 Isopleth:	1.60		Fig. SWM-2 in County Design Criteria
Rational Coefficients Cpre:	0.90		See note # 2
Cpost:	0.61		See note # 2
Impervious Area:	8550	ft^2	See note # 2 and # 4

STRUCTURE DIMENSIONS FOR DETENTION

35	ft^3 storage volume calculated		
100	% void space assumed		
35	ft^3 excavated volume needed		
Structure	Length	Width*	Depth*
Ratios	100.00	2.00	2.00
Dimen. (ft)	**44.39**	**0.89**	**0.89**

*For pipe, use the square root of the sectional area

25 - YEAR DESIGN STORM				DETENTION @ 15 MIN.	
Storm Duration (min)	25 - Year Intensity (in/hr)	10 - Yr. Release Qpre (cfs)	25 - Year Qpost (cfs)	Detention Rate To Storage (cfs)	Specified Storage Volume (cf)
1440	0.34	0.050	0.041	-0.291	-31402
1200	0.36	0.054	0.044	-0.288	-25881
960	0.40	0.059	0.048	-0.283	-20398
720	0.45	0.067	0.054	-0.277	-14969
480	0.53	0.079	0.064	-0.267	-9622
360	0.60	0.089	0.072	-0.259	-6997
240	0.71	0.105	0.086	-0.246	-4427
180	0.80	0.119	0.096	-0.235	-3174
120	0.94	0.140	0.114	-0.217	-1957
90	1.06	0.158	0.128	-0.203	-1371
60	1.26	0.187	0.152	-0.180	-808
45	1.42	0.210	0.171	-0.160	-541
30	1.68	0.249	0.202	-0.129	-291
20	1.98	0.294	0.239	-0.092	-138
15	2.23	**0.332**	0.270	-0.062	-70
10	2.64	0.392	0.319	-0.013	-9
5	3.52	0.522	0.425	0.093	35



Notes & Limitations on Use:

1) The modified rational method, and therefore the standard calculations are applicable in watersheds up to 20 acres in size.
2) Required detention volume determinations shall be based on all net new impervious area both on and off-site, resulting from the proposed project. Pervious areas shall not be included in detention volume sizing; an exception may be made for incidental pervious areas less than 10% of the total area.
3) Gravel packed detention chambers shall specify on the plans, aggregate that is washed, angular, and uniformly graded (of single size), assuring void space not less than 35%.
4) A map showing boundaries of both regulated impervious areas and actual drainage areas routed to the hydraulic control structure of the detention facility is to be provided, clearly distinguishing between the two areas, and noting the square footage.
5) The EPA defines a class V injection well as any bored, drilled, or driven shaft, or dug hole that is deeper than its widest surface dimension, or an improved sinkhole, or a subsurface fluid distribution system. Such storm water drainage wells are "authorized by rule". For more information on these rules, contact the EPA. A web site link is provided from the County DPW Stormwater Management web page.
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ATTACHMENT 7