APPENDIX C
GEOTECHNICAL INVESTIGATION

GEOTECHNICAL EXPLORATION

CARDEN PROPERTY
HAYWARD, CALIFORNIA

Submitted to:
Ms. Carrie Aitken Seranella
192 Carrick Circle
Hayward, CA 94542

Prepared by:
ENGEO Incorporated

July 15, 2016
Revised: March 24, 2017

Project No:
4920.003.000

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Ms. Carrie Aitken Seranella
192 Carrick Circle
Hayward, CA 94542

Subject: Carden Property
Hayward, California

GEOTECHNICAL EXPLORATION

Dear Ms. Seranella:

With your authorization, we have conducted a geotechnical exploration for the Carden Property project located in Hayward, California. The accompanying report presents the results of our site exploration and our conclusions and recommendations regarding the geotechnical aspects of the project. Based on our study, it is our opinion that the currently proposed development is feasible from a geotechnical standpoint provided the recommendations included in this report are followed.

We are pleased to provide our services to you on this project and look forward to consulting further with you and your design team.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
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No. 86636
CIVIL
STATE OF CALIFORNIA

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this exploration has been to characterize subsurface conditions and engineering properties of the soil and bedrock materials at the site and to provide design-level conclusions and recommendations regarding geotechnical aspects of the project.

The scope of our exploration included the following:

- Review of pertinent geologic maps and literature.
- Examination of aerial photographs covering the site.
- Review of previous geotechnical exploration reports regarding the project.
- Excavation and logging of 6 test pits using a track-mounted excavator. Bulk samples of representative soil materials were collected for laboratory testing.
- Laboratory testing of collected samples of soil materials including moisture content, Atterberg limits, and sieve testing.
- Analysis of the geological and geotechnical data.
- Preparation of this report providing our conclusions and recommendations.

This report was prepared for the exclusive use of Ms. Carrie Aitken Seranella and her design team consultants. In the event that any changes are made in the character, design or layout of the development, the conclusions and recommendations contained in this report should be reviewed by ENGEO to determine whether modifications to the report are necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO Incorporated.

1.2 SITE LOCATION AND DESCRIPTION

The study area is an irregular star-shaped parcel located on the western flank of the northwest-southeast trending Walpert Ridge, as shown on the attached Vicinity Map, Figure 1. The existing topography consists of gentle-to-moderate rolling terrain. The peak and approximate center of the development area is flanked to the north and south by natural swale formations. The site is bordered to the west, north, and east by aspects of the greater Stonebrae development project which included mass grading and drainage improvements directly adjacent the subject site.

The site is currently occupied by a small single-story residence with nearby horse stable and material storage.

1.3 PROPOSED DEVELOPMENT

A preliminary grading plan for the project has been prepared by Ruggeri, Jenson, and Azar (RJA) that is dated March 13, 2017. The plans indicate that the property will be developed with 28 single-family residential lots. The majority of the lots will be situated along the northern and eastern edges of the site.

Grading for the project site involves cuts of the hill peaks and fills in the natural swales, both up to about 25 feet thick. Proposed graded slope gradients are as steep as approximately 3:1. Up to 3-foot tall retaining walls are shown on the plans. Bioretention areas and underground hydromodification boxes are also shown for stormwater treatment.

The project design also includes an emergency vehicle access (EVA) road, perpendicularly traversing the northern swale.

2.0 GEOLOGY AND SEISMICITY

2.1 GEOLOGIC SETTING

The site is located within the Coast Ranges geologic province of California, which is dominated by a series of northwest-trending ridges and valleys. Bedrock in the province has been folded and faulted during regional uplift beginning in the Pliocene, roughly 4 million years before present. Regional geologic mapping by Graymer (2000) indicate that the site is underlain by an unnamed sandstone, conglomerate, and shale formation of the late Cretaceous period (Figure 3).

2.2 FAULTING

The site is not located within a State of California Special Studies Zone for active faults (CGS, 2012). Graymer (2000) indicates an unnamed thrust fault crossing the western perimeter of the site. Graymer (1996) shows an unnamed thrust fault in roughly the same location as the Niles fault mapped by Crane (1988). The Niles fault mapped by Crane (1988) and Graymer (2000) is not known to be active.

3.0 FIELD EXPLORATION

3.1 PREVIOUS EXPLORATION

ENGEO has conducted extensive geotechnical explorations in the vicinity of the site since approximately 2000 for the ongoing development of the adjacent Stonebrae community.

3.2 GEOTECHNICAL EXPLORATION

The field exploration for this study was conducted in July 2016 and consisted of the excavation of six test pits. The approximate test pit locations are shown on Figure 2. These areas of subsurface exploration were located by pacing from existing features and should be considered accurate only to the degree implied by the method used.

The test pits were excavated to depths of about 1 to 18 feet below ground surface using a track-mounted excavator. The test pits were logged in the field by us and the logs are included in Appendix A. Selected bulk samples from test pits were collected and transported to our laboratory for testing. All exploratory test pits were backfilled with the excavated soil with nominal compactive effort and is considered nonengineered fill.

4.0 LABORATORY TESTING

Following field explorations, we re-examined the samples in our laboratory to confirm field classifications. Representative samples recovered from our borings and test pits were tested for the following physical characteristics:

TABLE 4.0-1
Laboratory Testing

Test	Designation	Location of Results
Natural Moisture Content	ASTM D-2216	Appendix B
Atterberg Limits	ASTM D-4318	Appendix B
Gradation	ASTM D-422	Appendix B

Results of individual laboratory tests are presented in Appendix B.

5.0 GEOLOGIC UNITS

5.1 EXISTING FILL

Existing fill was not encountered during our exploration. However, the existing buildings may be underlain by minor quantities of pad leveling fill. Based on the preliminary plans by RJA, any existing fill material will likely be removed during grading operations.

5.2 COLLUVIUM (Qc)

Areas of thicker soil cover in swales are shown as colluvium (Qc) on Figure 2. Colluvial soils encountered in test pits consist of dark brown, olive, and yellowish brown silty clay that have a low to medium plasticity and low to medium expansion potential. The colluvial deposits encountered were stiff to very stiff and ranged from about 1 to 18 feet thick.

5.3 UNNAMED SANDSTONE (Kcv)

The Unnamed Sandstone (Kcv) includes pebble to cobble conglomerate, sandstone, siltstone, and shale. Bedding within the units are described as distinctly bedded in the lower and middle units and indistinctly to distinctly bedded in the upper unit. Bedrock structure was poorly developed in some of the test pit exposures. Bedrock in the northern portion of the site (TP-1 through TP-4) was extremely weak and weathered completely. Bedrock in the southern portion of the site (TP-5 through TP-6) tended to be medium strong with less weathering.

The excavator equipment used for this study was able to penetrate the Unnamed Sandstone to the depths explored without difficulty.

6.0 DISCUSSION AND CONCLUSIONS

6.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake may include primary ground rupture, ground shaking, lurching, liquefaction, lateral spreading and earthquake-induced landsliding. These potential hazards are discussed below. Risks from seiches, tsunamis and inundation due to embankment failure are considered low at the site.

6.1.1 Ground Rupture

The site is not within a State of California Earthquake Fault Zone and no known seismogenic faults have been mapped on the site. As previously discussed, the thrust fault mapped by Graymer (2000) is not considered active. Based on these findings, the potential for fault rupture at the site is considered low.

6.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the 2013 California Building Code (CBC) requirements, as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

6.1.3 Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater in poorly consolidated colluvial and alluvial deposits. Overexcavation of compressible materials and construction of engineered fills underlying all developed portions of the project is expected to mitigate this hazard.

6.1.4 California Building Code (CBC) Seismic Design Parameters

Based on the subsurface soil conditions encountered and local seismic sources, the site may be characterized for design based on California Building Code using the following information.

TABLE 6.1.4-1
2013 California Building Code

Coefficient	Value
Site Class	C
Mapped MCE Spectral Response Acceleration at Short Periods, S_S	2.09
Mapped MCE Spectral Response Acceleration at a Period of 1 second, S_1	0.86
MCE, 5% Damped, Spectral Response Acceleration at Short Periods Adjusted for Site Class Effects, S_{MS}	2.09
MCE, 5% Damped, Spectral Response Acceleration at a Period of 1 second Adjusted for Site Class Effects, S_{M1}	1.12
Design, 5% Damped, Spectral Response Acceleration at Short Periods, S_{DS}	1.40
Design, 5% Damped, Spectral Response Acceleration at a Period of 1 second, S_{D1}	0.74
MCE Geometric Mean Peak Ground Acceleration (PGA_M)	0.80

6.1.5 Liquefaction Potential

Liquefaction is a phenomenon in which saturated, loose or medium dense, cohesionless soils are subject to a temporary, but essentially total, loss of shear strength because of pore pressure build-up under the reversing cyclic shear stresses associated with earthquakes. Historically, standard geotechnical engineering practice for liquefaction assessment have included evaluating layers of loose to medium dense and saturated sandy deposits for liquefaction potential. However, empirical evidence from recent major earthquakes and published research by major universities indicate that some fine-grained soils may develop excess pore water pressures and exhibit liquefaction (sand-like) or cyclic softening (clay-like) and associated strength loss during a seismic event.

As previously mentioned, we encountered low to medium plastic fine grain soil during our exploration. We evaluated the potential for fine-grained liquefaction triggering using methods outlined by Bray and Sancio (2006) and Seed (2003).

The project site is not located in mapped Seismic Hazard Zone (2012) by the California Geologic Survey (Figure 5). Based on the above analysis and no observed groundwater during our exploration, the potential for liquefaction triggering is low.

6.1.6 Lateral Spreading

Lateral spreading is a failure within weaker soil material which causes the soil mass to move towards a free face or down a gentle slope. Surficial soils on slopes will be removed as a part of the recommended grading operations. Keyways for fills slopes are expected to extend to bedrock. These recommendations are intended to mitigate the potential for adverse impacts from lateral spreading.

6.1.7 Seismically Induced Landsliding

As with most of the surrounding hillside developments, landslides and slope stability are important issues for the project. Slopes onsite are not shown on the State of California Seismic Hazard Zones Map (2012, Figure 5) as areas that may be susceptible to seismically induced landsliding. Although seismically induced landsliding can be a significant hazard, it can generally be mitigated through proper grading procedures. Mitigation measures for this project include removing loose colluvial soil and rebuilding slopes with engineered fill keyed into bedrock.

6.2 SLOPE STABILITY

Existing slope gradients are as steep as 2:1 (H:V) and appear to be stable with no indication of movement other than minor surficial sloughing. The proposed maximum on site slope gradient of 3:1 per the plans prepared by RJA includes the regrading of the existing 2:1 slopes. The recommendations in Section 7 are intended to mitigate potential slope stability issues.

6.2.1 Slope Stability Analysis

Based on conservative strength parameters derived from our experience with sites in the vicinity and compared with this site’s soil types, the effective strength parameters selected for use in slope stability analysis are presented in the following table.

TABLE 6.2.1-1
Shear Strength Parameters For Use in Stability Analyses

Material	γ'	Static Condition		Seismic Condition	
		C’	ϕ’	C’	ϕ’
Engineered Fill (General)	125	150	27	350	18
Colluvium	125	0	25	1000	0
Bedrock	130	0	38	0	38

Note: γ’ = Moist Unit Weight (pcf)
C’ = Effective Cohesion (psf)
ϕ’ = Effective Angle of Internal Friction (Degrees)

Slope stability analysis was conducted on cross sections through the highest proposed fill slopes. The cross section locations are shown on Figure 2.

In accordance with current guidelines for analysis of slopes under seismic conditions (State of California Special Publication SP117A), a seismic coefficient of 0.22g was applied for the slope stability analyses based on a peak ground of 0.80 from the 2013 CBC and a moment magnitude of 7.3 from a theoretical rupture of the Hayward fault. In general, a Factor of Safety (FS) of 1.5 under the static condition and 1.0 under seismic condition are considered acceptable for slope stability analyses. A summary of our slope stability results is presented in the following table and the slope stability analyses results are provided in Appendix C.

TABLE 6.2.1-2
Summary of Slope Stability Analyses

Cross Section Designation	Factor of Safety Static Case	Factor of Safety Seismic (Pseudo-Static) Case
A-A’	2.0	1.0
B-B’	2.1	1.2

6.3 EXPANSIVE SOILS

The expansive nature of the near-surface native soils is of significant geotechnical concern in this region. The clayey soil at the subject area is considered moderately expansive. Conversely, the sandstone and siltstone bedrock at the site is considered low to non-expansive.

Expansive soils are susceptible to shrink and swell resulting from variations in moisture content. Expansive soils and bedrock may cause heaving and cracking of slabs-on-grade, pavements and foundations. Building damage due to volume changes associated with expansive soils may be reduced by the following measures: (1) selectively placing the more expansive materials in the lower portions of the deeper fill areas (generally at depths below 10 feet from finished grades), or placing these expansive materials outside the limits of the proposed house structures and site improvements (such as placing these in landscape areas); (2) performing proper moisture conditioning and compaction of fill materials within specified ranges to reduce their swell potential; and (3) supporting houses upon structurally reinforced mats and/or post-tensioned mats designed to resist the deflections associated with expansion/compression-related movements. Foundation criteria are further discussed in the "Foundations” section of this report.

Successful construction on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist by occasional sprinkling for several days prior to placement of concrete for foundation construction. It is extremely difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction. Mitigation measures should include the prevention of moisture variation.

6.4 SWELL/SETTLEMENT RELATED TO DEEP FILL

According to the preliminary grading plans, fills up to about 25 feet thick will be placed in the swales and low-lying areas. Studies have shown that engineered fills in residential development sites typically experience increases in moisture content after building construction due to increases in irrigation or natural conditions and to alternation of drainage pattern. This process may take about 5 to 10 years after irrigation commences, or even more, before the fill becomes fully wetted. The wetting process can cause settlement or swell (hydrocompression due to wetting) depending on soil type, compaction, moisture content, and the overburden pressures (fill thickness).

Based on our experience of soil and bedrock in the site vicinity, the swelling potential for low confining pressures (shallow fill) is significantly reduced when the soils are compacted to a relatively lower compaction and at a higher moisture content. In addition, the settlement potential for fill with high confining pressures (deep fill) is minimized when the soils are compacted to a relatively higher compaction. Our compaction and moisture content requirements are provided in “Fill Placement” section.

Our experience with similar fills in the vicinity indicates the deepest fills onsite should experience less than one inch of post-construction settlement.

6.5 COMPRESSIBLE SOILS

Excessive total and differential settlement at the site may also result from: (1) consolidation of the compressible colluvial deposits in swale areas where fills will be placed; and (2) settlement of foundation elements supported directly over these compressible colluvial deposits. To reduce settlement resulting from these deposits, it is recommended that these deposits be over-excavated to expose stiff in-place materials with grades restored with properly compacted engineered fill material as discussed in the “Grading” section of this report. It is anticipated that these deposits may be reused as fill material.

6.6 DIFFERENTIAL FILL THICKNESS

Some of the single-family residential lots planned on fills above existing steep slopes could have a differential fill thickness if not graded properly. Differential building movements, although not seriously damaging, may become apparent for large differentials in fill thickness. Overexcavation requirements are provided in the recommendations sections.

6.7 CUT-FILL TRANSITION LOTS AND CUT LOTS

Some residential lots in this project site will likely be entirely in cut or traversed by a cut-fill transition. We anticipate that variations in material properties may occur in areas of cut or cut-and-fill daylighting if not mitigated during site grading. Atterberg Limits test data indicate that there is a potential for a differential in swell characteristics across cut areas and cut/fill transitions. Such situations can be detrimental to building performance. Recommendations are provided subsequently in this report to mitigate the effect on structures caused by differential subgrade performance over cuts and cut-fill transition zones.

6.8 CORROSIVITY CONSIDERATIONS

Sulfate testing was not conducted at the site. It is our opinion that near-surface soil samples should be collected from the building pads for sulfate testing after the site grading is complete. If testing is deemed undesirable, modified Type II cement can be used in foundation concrete for structures at the project site. Additionally, concrete should incorporate a maximum water cement ratio of 0.45 and a minimum compressive strength of 4,500 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications.

6.9 EXCAVATION CHARACTERISTICS

Some well-cemented, thickly bedded sandstone layers were encountered during this and previous nearby site explorations. In general, we anticipate that conventional heavy-duty grading equipment should be able to rip the bedrock to a depth of approximately 15 to 20 feet, although some well-cemented beds or lenses may be encountered that will be very difficult to rip and may require blasting. Within the sandstone units, the excavation of bedrock could produce considerable oversized material, 3 to 6 feet in diameter or larger, and will likely require blasting.

Within the sandstone cut areas, trenching using conventional equipment may prove very difficult to impractical in most of these areas. During mass grading, zones of hard rock exposed near finished grade should be identified; overexcavation in roadway areas may be appropriate to facilitate installation of utilities. Also, in these hard rock areas it may be appropriate to overexcavate cut lots and transition lots to facilitate foundation, pool and or utility installation.

Rocks greater than 6 inches in diameter that are generated or encountered during grading should be placed in accordance with recommendations provided in the "Selection of Materials" section.

6.10 CONCLUSIONS

Based on the findings of our explorations, we conclude that the proposed development is feasible from a geotechnical standpoint. The primary geotechnical concerns are slope stability, expansive soil, and the swell/settlement of the proposed fills.

7.0 RECOMMENDATIONS

7.1 GRADING RECOMMENDATIONS

Based on the findings subsurface exploration, geologic mapping, and laboratory test results, as well as our local field experience, we provide the following geotechnical recommendations for use during grading and construction.

ENGEO should be notified at least one week prior to grading to coordinate our schedule with the grading contractor. Grading operations should meet the requirements of the Supplemental Recommendations included in Appendix D and must be observed and tested by ENGEO's field representatives. After the grading operations commence, geologic observations of cut areas should be made at frequent intervals by the Engineering Geologist. This is advised so that modified geologic recommendations can be incorporated into updated grading plans as grading proceeds. The Engineering Geologist should be notified at least 48 hours prior to the start of cutting of significant slopes.

Ponding of stormwater, other than within engineered detention basins, should not be permitted at the site, particularly during work stoppage for rainy weather. Before the grading is halted by rain, positive slopes should be provided to carry surface runoff to storm drainage structures in a controlled manner to prevent erosion damage.

7.1.1 Demolition and Stripping

Site development will commence with the removal of improvements and their foundations, and buried structures, including abandoned utilities and septic tanks and their leach fields, if any exist. All existing non-documented artificial fills, vegetation, and soft or compressible soils should be removed as necessary for project requirements. Tree roots should be removed to a depth of at least 3 feet below finished grade in cut lots and 3 feet below original grade in fill lots. Actual depths of stripping and unsuitable material and tree root removal will be determined by the Geotechnical Engineer's field representative during grading.

Within the development areas, excavations resulting from demolition and stripping which extend below final grades should be cleaned to firm undisturbed soil as determined by the Geotechnical Engineer's representative. Following clearing and grubbing, all depressions in areas to be filled should be scarified, moisture conditioned and backfilled with compacted engineered fill. The requirements for backfill materials and placement procedures are the same as those for engineered fill described below. No loose or uncontrolled backfilling of depressions resulting from demolition and stripping should be permitted.

7.1.2 Toe Keyways

After stripping, mass grading should begin with construction of toe keyways and subdrains. All fills should be adequately keyed into firm natural materials unaffected by shrinkage cracks. Typical keyway details and typical subdrain details are shown on Figures 6 and 7. Horizontal benches should be excavated into firm soil or bedrock as the filling proceeds. The vertical spacing of benches should be no more than 5 feet unless otherwise recommended by the Geotechnical Engineer. The actual size of the keyways and benches should be determined by the Geotechnical Engineer in the field during grading.

7.1.3 Placement of Fill

Areas to receive fill should be scarified to a depth of 12 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts. The lift thickness should not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less. Track rolling to compact faces of slopes is usually not sufficient; typically, slopes should be overfilled a minimum of 2 feet and cut back to design grades. The following compaction recommendations should be used for the placement and compaction of fills:

TABLE 7.1.3-1
Compaction and Moisture Content Requirements

Description	Materials	Minimum Relative Compaction (%)	Minimum Moisture Content (Percentage Points Above Optimum)
Within the upper 5 feet of finished grade	Expansive (PI>12)	87 to 92	5
	Non-expansive	90	2
From 5 to 50 feet below finished grade	Expansive (PI>12)	90	4
	Non-expansive	95	2

Relative compaction refers to in-place dry density of the fill material expressed as a percentage of the maximum dry density as determined by ASTM D-1557. Optimum moisture is the moisture content corresponding to the maximum dry density. We recommend that the fills be compacted at higher than optimum moisture contents as shown above to minimize the effects of swell and/or hydrocompression.

7.1.4 Selection of Materials

With the exception of the organically contaminated near-surface materials, the site soils and rocks containing less than 3 percent organics are suitable for use as engineered fill. Rocks greater than 6 inches in size (if encountered) should be placed at depths greater than 15 feet from finished grade. Rocks greater than 18 inches in size (if any) should be broken down such that their maximum dimension is less than 12 inches, or otherwise removed from the site.

7.1.5 Import Materials

The Geotechnical Engineer should be informed if any importation of soil is contemplated. Import materials, if any are needed, must meet the requirements contained in the Supplemental Recommendations in Appendix D. A sample of the proposed import material should be submitted to the Geotechnical Engineer for evaluation by laboratory testing prior to site delivery.

7.1.6 Cut-Fill Transition Lots and Cut Lots

Some of the lots in this project will be entirely in cut or traversed by a cut-fill transition. Significant variations in material properties may occur in areas of cut or cut-and-fill transition, due to the highly variable nature of site soils and bedrock, and such situations can be detrimental to building performance.

Based on the laboratory properties and performance of nearby sites with similar soils, we recommend that upper portions of transition lots be re-worked so as to achieve a 3-foot layer of properly compacted engineered fill. Cut lots should be reworked so as to achieve at least a 2-foot layer of properly compacted engineered fill. Cut/fill transition lots and cut lots should be designated on corrective grading plans prepared by ENGEO for the project.

7.1.7 Differential Fill Lots

Some of the single-family residential lots planned for fills over steep natural slopes could have a differential fill thickness greater than 10 feet. Differential building movements could occur with these large differential fill thicknesses if not properly mitigated. Local subexcavation of soil material and replacement by engineered fill will be necessary to achieve a maximum differential fill thickness of 10 feet across the building envelope. Overexcavation requirements will be provided by the Geotechnical Engineer based on a review of the final grading plans. In addition, additional swell testing may be performed during the mass grading.

7.1.8 Settlement Monitoring

Due to the increased load in areas of deep fill, settlement is anticipated within the engineered fill. We recommend that surface markers be installed after fill placement to monitor the settlement magnitude and duration. Surface markers may consist of standard survey control stakes (metal rods), driven to 3 feet below finished graded. We recommend the surface monument locations and elevation be surveyed once a month following finish grading, and to include measurements during following rainy season. The interval of field measurements may be reduced based on gathered data, as appropriate.

Construction of homes in areas of deeper fill over existing soils may need to be delayed until the majority of settlement has taken place. The results of the settlement monitoring will be used to determine when house construction may begin in the deeper fills have been placed over existing soils.

7.1.9 Construction of Subsurface Drainage Facilities

Subsurface drainage systems should be installed in all keyways and in swales or natural drainage ways which are to be filled. Swales should be cleaned to a firm soil or rock base. A subdrain should then be installed through the subexcavation. Desiccated, cracked surface clays and slumping soils located along the swale areas should be removed, and the slopes should be benched prior to the placement of fill. Actual limits of subexcavation should be determined in the field at the time of grading by the Geotechnical Engineer.

Additional subdrains should be added where seepage or wet conditions are encountered during excavation. Subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in an 18-inch minimum thickness of Caltrans Class 2 permeable material or coarse rock wrapped in geotextile filter fabric, as shown on the typical detail on Figure 7. The subdrain pipe and drainage blanket should meet the requirements contained in the Supplemental Recommendations presented in Appendix D.

Discharge from the subdrains will generally be low but in some instances may be continuous. Subdrains should be conveyed to approved outlets, and their locations should be documented for future maintenance. New sources of seepage may be created by a combination of changed topography, irrigation patterns, and potential utility leakage. Since uncontrolled water flows are one of the major causes of detrimental soil movements, it is of utmost importance that a Geotechnical Engineer be advised of any seepage conditions so that remedial action may be initiated if necessary. All subdrain connections and tie-ins to storm drain inlets should be observed and approved by the Geotechnical Engineer.

7.1.10 Monitoring and Testing

It is important that all site preparations for site grading be done under the observation of a Geotechnical Engineer’s field representative. The Geotechnical Engineer’s field representative should observe all graded area preparation, including demolition and stripping following the recommendations contained in the Supplemental Recommendations in Appendix D. The final grading plans should be submitted to the Geotechnical Engineer for review.

7.1.11 Surface Drainage Requirements

Improper drainage may result in fill saturation with consequent loss of fill strength. It is very important that all lots be positively graded at all times to provide for rapid removal of surface water. Ponding of water under floors or seepage toward foundation systems at any time during or after construction must be prevented.

As a minimum requirement, finished grades should provide a slope gradient of at least 3 percent within 5 feet from exterior walls (perpendicular to the wall alignment) to allow surface water to drain positively away from the structures. For paved areas, the slope gradient can be reduced to 2 percent. Care should be exercised to ensure that landscape mounds will not interfere with these requirements. Sufficient area drains should be provided around the buildings to remove excess surface water.

All lots should be drained individually. Stormwater from roof downspouts should be conveyed in closed drain systems to a drainage facility. If planting adjacent to a building is desired, the use of drought-tolerant plants that require very little moisture is recommended, and irrigation of landscape areas should be limited strictly to that necessary for plant growth.

7.1.12 Erosion Control

The tops of fill or cut slopes should be graded in such a way as to prevent water from flowing freely down the slopes. Due to the nature of the site soil and bedrock, graded slopes may experience severe erosion when grading is halted by heavy rain. Therefore, before work is stopped, a positive gradient away from the tops of slopes should be provided to carry the surface runoff away from the slopes to areas where erosion can be controlled. It is vital that no completed slope be left standing through a winter season without erosion control measures having been provided.

Because the existing bedrock is relatively nutrient-poor, it may be difficult for vegetation to become properly established, resulting in a potential for slope erosion. Revegetation of graded slopes can be aided by retaining the organic-rich strippings and spreading these materials in a thin layer (approximately 6 inches thick) on the graded slopes prior to the winter rains and following rough grading. When utilizing this method, it is sometimes possible to avoid hydroseeding. All landscaped slopes should be maintained in a vegetated state after project completion. The use of drought-tolerant vegetation requiring infrequent drip irrigation during summer is recommended. No pressurized irrigation lines should be placed on or near the tops of graded slopes.

7.1.13 Uphill Slope Condition

Where a building pad is adjacent to uphill slopes, all permanent structures should be set back from the toe the equivalent distance of one-half the vertical graded slope height. The maximum required setback is 15 feet from the toe of slope.

7.1.14 Downhill Slope Condition

All permanent structures should be set back from the top of a downhill slope the equivalent distance of one-third of the vertical graded slope height. The maximum setback distance is 40 feet from the top of slope. If a shorter setback distance is desired, a buried retaining wall may be recommended on a case-by-case basis.

7.2 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from property lines, keyways and structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements that will experience lateral loads (such as from impact or traffic patterns), additional design considerations may be required. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should minimize the exposure time such that the improvements are not detrimentally impacted.

7.3 FOUNDATION RECOMMENDATIONS

7.3.1 Foundation Recommendations

The following preliminary foundation recommendations are provided for planning purpose and should be evaluated once the final foundation layouts are provided.

It is anticipated that the residential dwellings and related buildings will consist of wood-frame structures. As recommended in the previous sections, special subgrade preparation will be implemented for cut lots, cut/fill lots and differential fill lots. Based on the soil data and provided that the building pads will be prepared as recommended, it is our opinion that the proposed residential structures can be supported on structural mat foundations provided that the structures are located on level ground and a minimum 10 feet from top of slopes. The following preliminary mat foundation recommendations are based on soil materials collected during field explorations. Soil sampling should be conducted on the building pads once the site grading is complete to confirm the following mat foundation recommendations are valid for the site.

7.3.2 Post-Tensioned Slab

The soil design parameters presented below assume that post-tensioned mats are designed according to the method recommended in “Design of Post-Tensioned Slabs-On-Ground” (Post-Tensioning Institute, 2004, 3rd Edition).

Center Lift Condition:

Edge Moisture Variation Distance, e_m = 9.0 feet
Differential Soil Movement, y_m = 0.2 inch

Edge Lift Condition:

Edge Moisture Variation Distance, e_m = 4.7 feet
Differential Soil Movement, y_m = 0.6 inches

Post-tensioned mats should be designed for an average allowable soil pressure of 1,000 pounds per square foot (psf) or 1,500 psf for concentrated loads. These values may be increased by one-third when considering total loads, including wind or seismic loads.

A minimum mat thickness of 10 inches is recommended. The actual thickness of the mat should be determined by the project Structural Engineer.

7.3.3 Subgrade Treatment for Structural Mat Foundations

The subgrade material under structural mats and post-tensioned slabs should be smooth and uniform to final design pad grade. In addition, the foundation subgrade should be moisture conditioned by presoaking or sprinkling these areas immediately (typically within 48 hours) prior to placement of the concrete foundation elements. In general, presoaking should be performed in such a manner as to obtain moisture contents of at least optimum and at least 5percent above optimum for non- to low expansive materials and moderate to highly expansive materials, respectively. The subgrade should not be allowed to dry prior to concrete placement. The moisture content of the foundation subgrade soils should be checked by the Geotechnical Engineer prior to their acceptance to receive concrete.

The Structural Engineer should be consulted on the advisability of using a 2-inch-thick sand cushion under mats for concrete curing purposes. Where floor coverings are anticipated, we recommend that the concrete be underlain by a tough, vapor retarder at least 10 mils thick to reduce moisture transmission through the concrete. The vapor retarder under the mats should meet ASTM E 1745 – 11 Class A requirements for vapor permeance, tensile strength, and puncture resistance.

7.3.4 Secondary Slabs-on-Grade

This section provides guidelines for secondary slabs, such as walkways around the buildings. Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with reduced potential for foundation distress. Where secondary slab-on-grade construction is anticipated, care must be exercised in attaining a near-saturation condition of the subgrade soil before concrete placement.

Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected due to concrete shrinkage. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. As a minimum, secondary slabs-on-grade should be reinforced with No. 3 bars spaced 18 inches on center each way.

Secondary slabs-on-grade should have a minimum thickness of 4 inches. A 4-inch-thick layer of clean crushed rock or gravel (Part I of the Supplemental Recommendations) should be placed under slabs. Exterior slabs should be constructed with thickened edges extending at least beneath the granular material into compacted soil to reduce water infiltration. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building.

7.4 RETAINING WALLS

Unrestrained walls constructed on level foreground should be designed for active lateral fluid pressure and a dynamic increment as provided below. The resultant force of the dynamic increment portion of wall loading should be applied at a height one third of the way up the wall.

TABLE 7.4-1

Backfill Slope Condition	Active Pressure (pcf)	Dynamic Increment (pcf)
Level	50	10
4:1	55	25
3:1	60	40
2:1	70	55

Passive pressures acting on foundations and keyways may be assumed as 250 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper one foot of soil should be excluded from passive pressure computations unless it is confined by pavement or a concrete slab.

The friction factor for sliding resistance may be assumed as 0.35. It is recommended that retaining wall footings be designed using an allowable bearing pressure of 2,500 pounds per square foot (psf) in firm native materials or fill. The footings should be at least 24 inches below lowest adjacent grades. Walls on sloping terrain should be supported on drilled piers. Passive resistance may be applied at a depth where there is 10 feet horizontal distance to the face of the slope from the pier. An equivalent fluid weight of 250 pounds per cubic foot (pcf) acting on 1.5 times the pier diameter may be used to evaluate passive resistance. An allowable skin friction value of 500 psf can be used for supporting vertical loads. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

The Geotechnical Engineer should be consulted on design values where surcharge loads, such as from automobiles or buildings, are expected. In addition, if the walls are located close together, surcharge from the structure above the wall should be incorporated in the wall design.

All retaining walls should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in Class 2 permeable material (Part I of Supplemental Recommendations), or free-draining gravel surrounded by synthetic filter fabric. The width of the drain blanket should be at least 12 inches. The drain blanket should extend to about one foot below the finished grades. As an alternative, prefabricated synthetic wall drain panels can be used. The upper one foot of wall backfill should consist of onsite clayey soils. Collector perforated pipes should be directed to an outlet approved by the Civil Engineer. Subdrain pipe, drain blanket and synthetic filter fabric should meet the minimum requirements as listed in Part I of the Supplemental Recommendations.

All backfill should be placed in accordance with recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls.

7.5 PRELIMINARY PAVEMENT DESIGN

The following sections are based on an assumed R-value of 5 and a range of Traffic Indices.

TABLE 7.5-1

Road Type	Traffic Index	AC (inches)	AB (inches)
Local - Residential Streets	5.0	3.0	10.0
Collector - Residential Streets	7.0	4.0	15.5

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement section is provided for estimating only. We recommend that the R-Value of the actual subgrade material be confirmed through testing after pavement subgrades are established.

Pavement construction and all materials should conform to the specifications and requirements of the Standard Specifications by the Division of Highways, Department of Public Works, State of California, latest edition, City of Hayward requirements, and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned to at least 3 percentage points above optimum, and compacted to at least 95 percent relative compaction and in accordance with city requirements.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate base materials are placed and compacted.

- Adequate provisions must be made such that the subgrade soils and aggregate base materials are not allowed to become saturated.

- Aggregate base materials should meet current Caltrans specifications for Class 2 aggregate base and should be compacted to at least 95 percent of maximum dry density within the road and should be compacted to at least 90 percent of maximum dry density beneath the sidewalk with moisture at or above optimum.

- Asphalt paving materials should meet current Caltrans specifications for asphalt concrete.

7.6 DRAINAGE

The building pads must be positively graded at all times to provide for rapid removal of surface water runoff from the foundation systems and to prevent ponding of water under floors or seepage toward the foundation systems at any time during or after construction.

Ponding of stormwater must not be permitted on the building pads during prolonged periods of inclement weather. As a minimum requirement, finished grades should have slopes of at least 3 to 5 percent within 7 feet from the exterior walls at right angles to them to allow surface water to drain positively away from the structures. For paved areas, the slope gradient can be reduced to 2 percent. All surface water should be collected and discharged into the storm drain system. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should be directed to a solid pipe that discharges to the street, to an approved outlet, or onto an impervious surface, such as the concrete apron or pavement area that will drain at a 2 percent slope gradient.

7.7 REQUIREMENTS FOR LANDSCAPING IRRIGATION

Vegetation should not be planted immediately adjacent to structures. If planting adjacent to the building is desired, we recommend using plants that require very little moisture with drip irrigation systems.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 5 feet of the walls or under structures. Ponding or saturation of foundation soils may cause loss of soil strength, and movements of the foundation and slabs.

Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. Excessive irrigation could result in saturation and weakening of foundation soils. The Landscape Architect and prospective owners should be informed of the surface drainage requirements included in this report.

7.8 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Pipe zone backfill (i.e., material beneath and immediately surrounding the pipe) may consist of a well-graded import or native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e., material placed between the pipe zone backfill and the ground surface) may consist of native soil compacted in accordance with recommendations for engineered fill.

Material used for pipe zone backfill should consist of fine- to medium-grained sand or a well-graded mixture of sand and gravel, but this material should not be used within 2 feet of finish grades. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of: (1) soil into the relatively large void spaces present in this type of material, and (2) water along trenches backfilled with this type of material. All utility trenches entering buildings and paved areas must be provided with an impervious seal consisting of native materials or concrete where the trenches pass under the building perimeter or curb lines. The impervious plug should extend at least 3 feet to both sides of the crossing.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in paved areas should be constructed in accordance with City of Dublin requirements. Compaction of trench backfill by jetting should be avoided. The owner should be notified if a conflict between city or other agency requirements and the recommendations contained in this report is observed to provide a resolution prior to submitting bids.

7.9 ADDITIONAL SERVICES

We should be given the opportunity to review 40-scale grading plans for the project and to prepare corrective grading plans. The corrective grading plans will show the recommended locations of keyways, subdrains and other overexcavation areas.

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damage inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our work.

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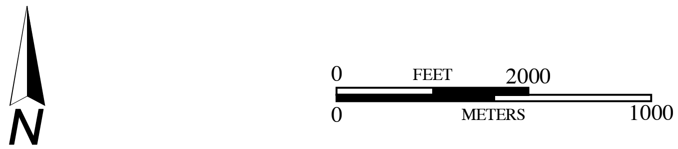
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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VICINITY MAP CARDEN PROPERTY HAYWARD, CALIFORNIA	PROJECT NO.: 4920.003.000		FIGURE NO.
	SCALE: AS SHOWN		1
	DRAWN BY: SRP	CHECKED BY: EH	

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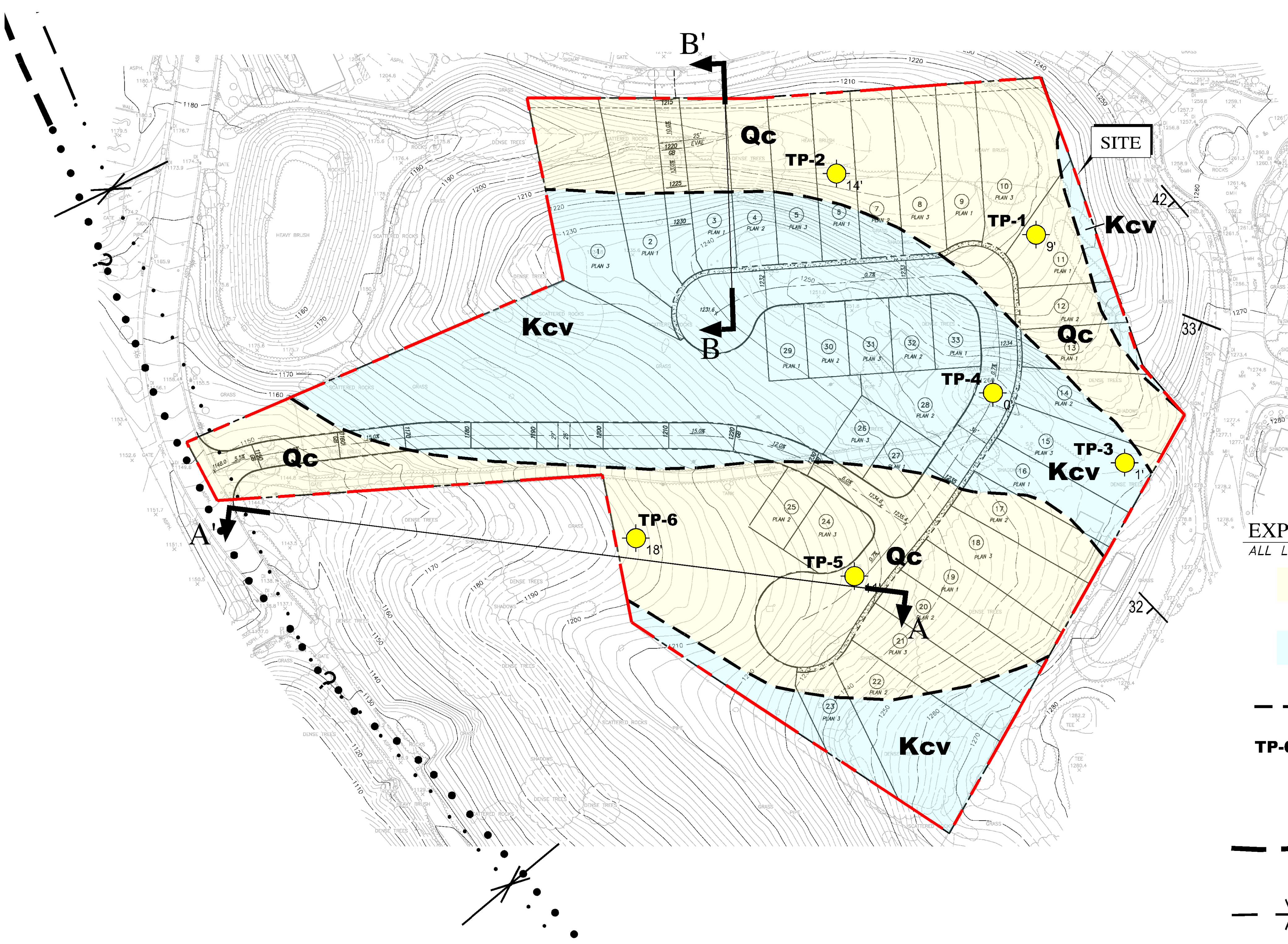


N

0 FEET 120
0 METERS 60

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qc COLLUVIUM (APPROXIMATE EXTENT WHERE THICKNESS GREATER THEN APPROXIMATELY 3')

Kcv LATE CRETACEOUS, SUBUNIT OF THINLY BEDDED CLAYSTONE, SILTSTONE AND SANDSTONE

GEOLOGIC CONTACT

TP-6 18' TEST PIT SHOWING DEPTH TO SANDSTONE BEDROCK (ENGEO, 2016)

33 STRIKE AND DIP OF BEDDING

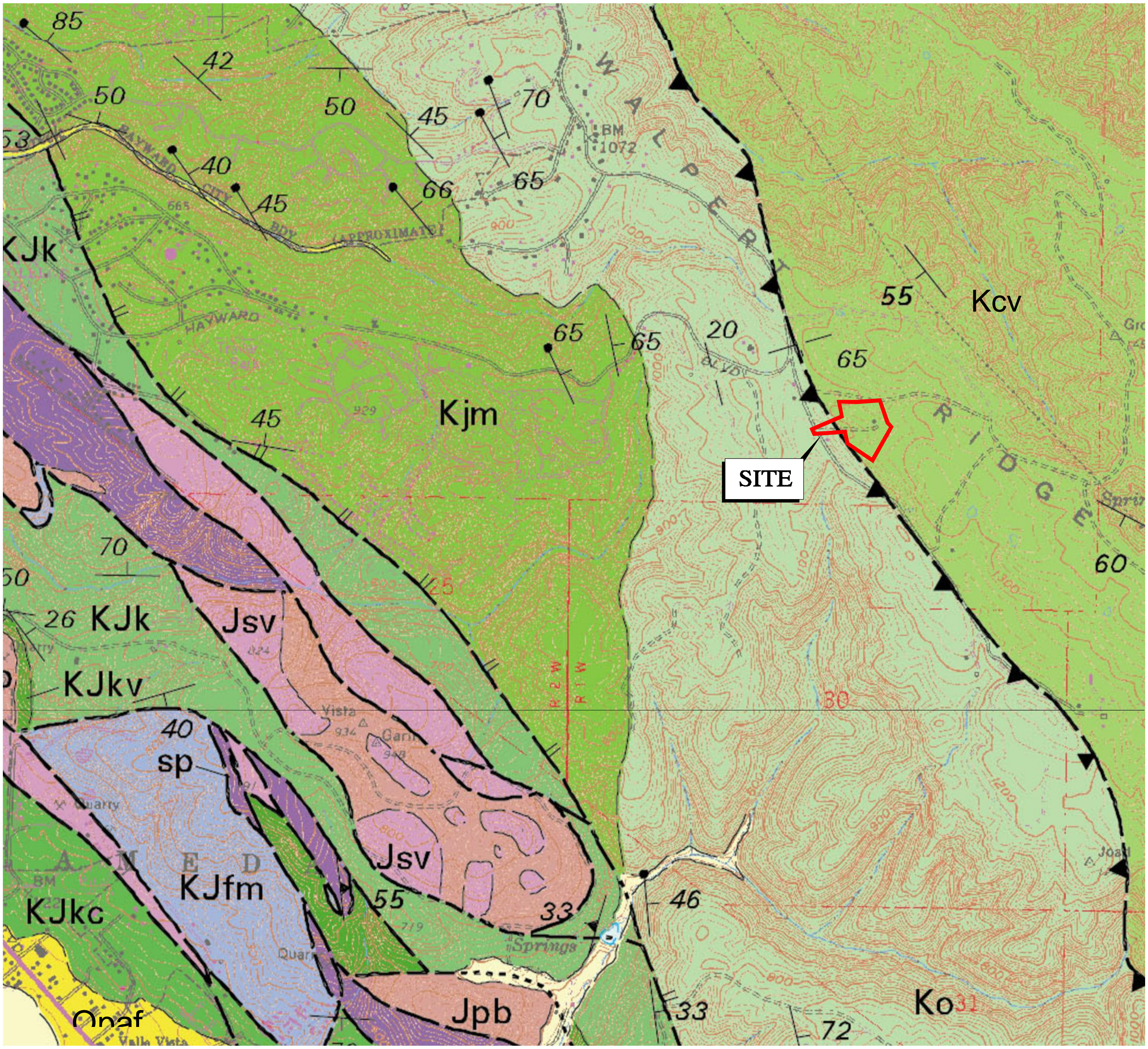
? INACTIVE FAULT, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED OR INFERRED, QUERIED WHERE UNCERTAIN

SYNCLINE, DASHED WHERE APPROXIMATE, DOTTED WHERE CONCEALED OR INFERRED, QUERIED WHERE UNCERTAIN

A A' CROSS SECTION LOCATION

BASE MAP SOURCE: RJA

ENGEO Expect Excellence	SITE PLAN CARDEN PROPERTY HAYWARD, CALIFORNIA	PROJECT NO.: 4920.003.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: EH	FIGURE NO. 2



EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL.

REVERSE OR THRUST FAULT

STRIKE AND DIP OF STRATA

INCLINED

Qpaf	MODERN STREAM DEPOSITS
Kjm	JOAQUIN MILLER FORMATION
KJk	KNOXVILLE FORMATION
KJkv	VOLCANOCLASTIC BRECCIA
KJfm	FRANCISCAN COMPLEX-MELANGE
Kcv	UNNAMED SANDSTONE
Ko	OAKLAND CONGLOMERATE
Jsv	KERATOPHYRE
Jpb	PILLOW BASALT

N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: GRAYMER, 2000

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REGIONAL GEOLOGIC MAP
CARDEN PROPERTY
HAYWARD, CALIFORNIA

PROJECT NO.:	4920.003.000
SCALE:	AS SHOWN
DRAWN BY: SRP	CHECKED BY: EH
FIGURE NO.	3

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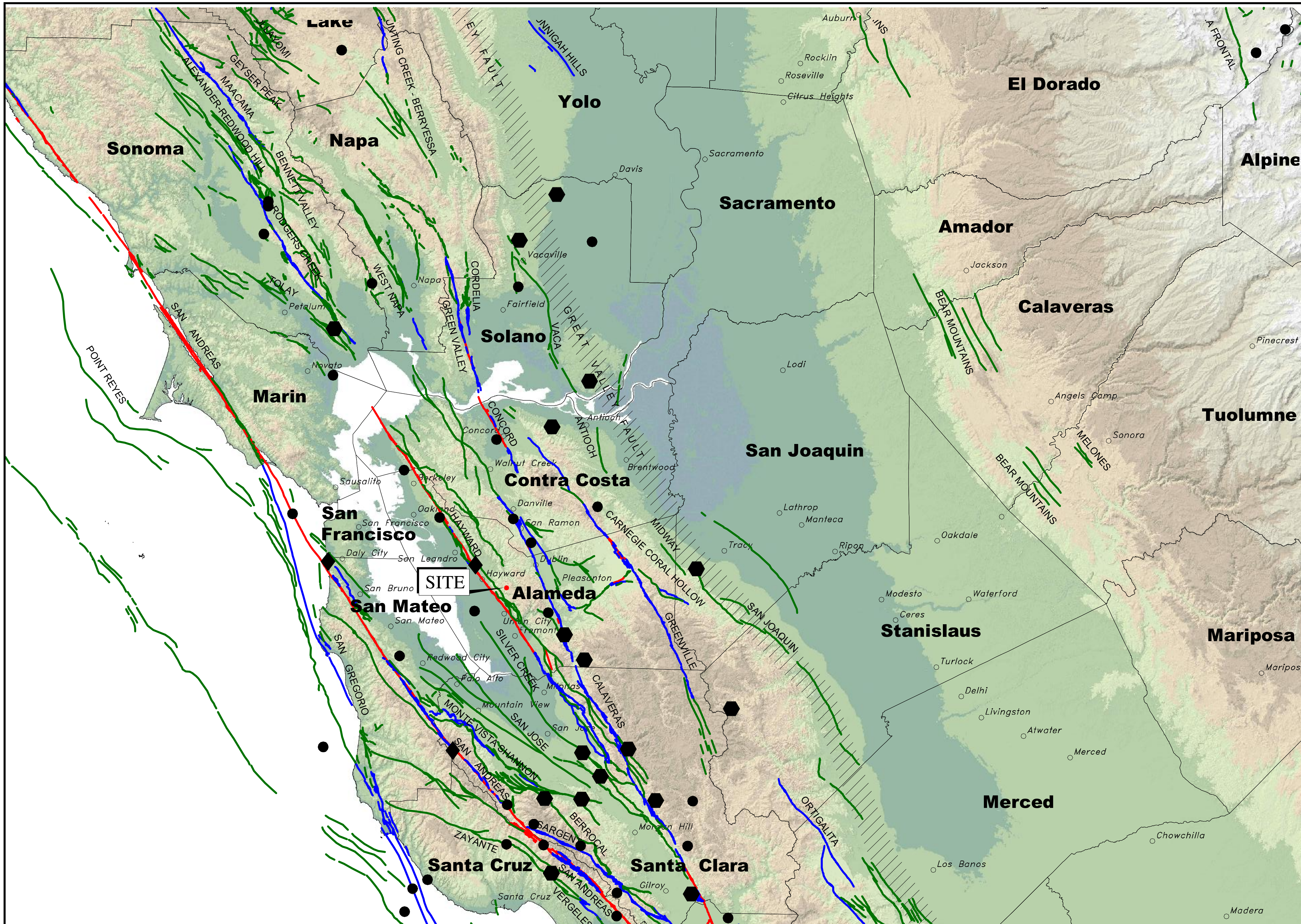

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0 MILES 15
0 KILOMETERS 30

EXPLANATION

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

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REGIONAL FAULTING AND SEISMICITY
CARDEN PROPERTY
HAYWARD, CALIFORNIA

PROJECT NO.: 4920.003.000		FIGURE NO
SCALE: AS SHOWN		4
DRAWN BY: SRP	CHECKED BY: EH	

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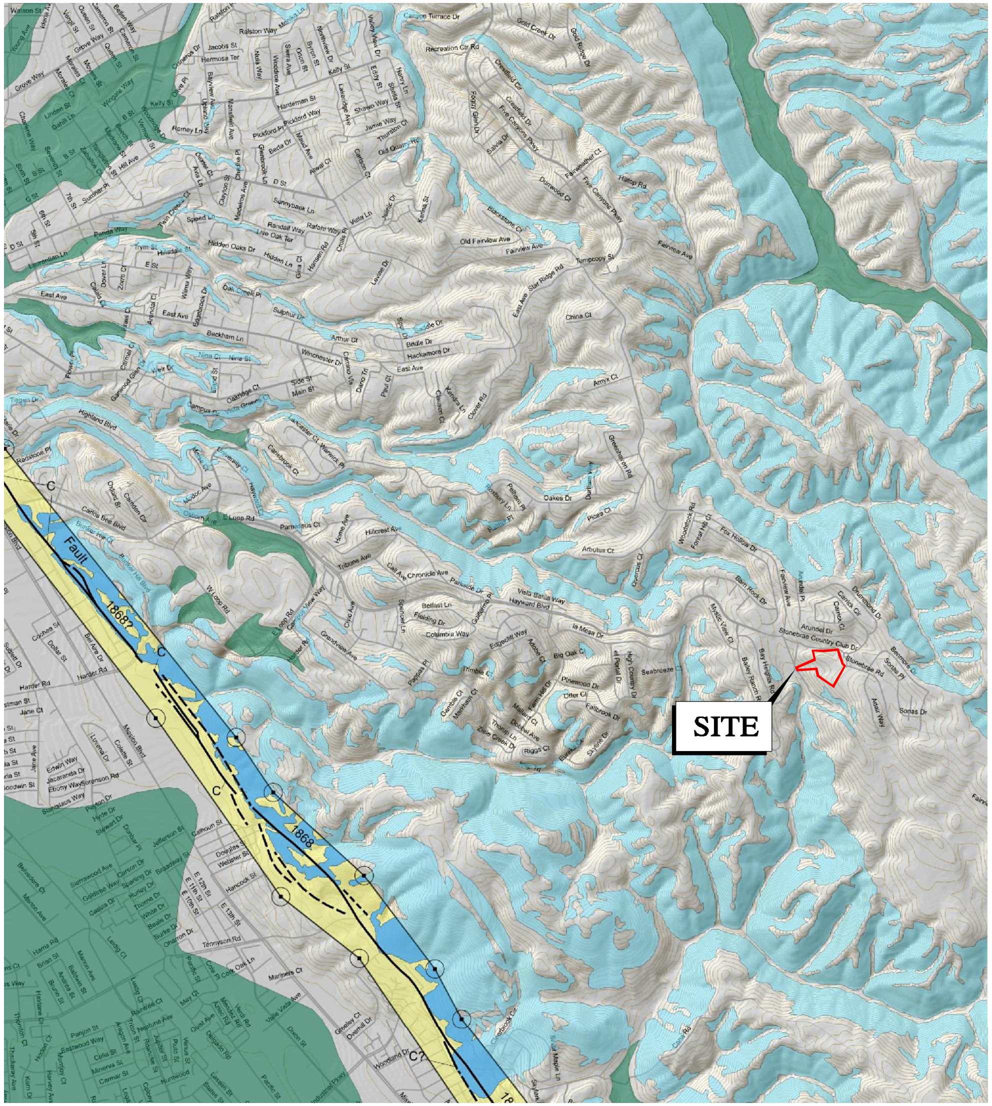

EXPLANATION

EARTHQUAKE FAULT ZONES

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

ZONES OF REQUIRED INVESTIGATION

Earthquake Fault Zones
Zones are areas delineated as straight-line segments that connect encircled turning points encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as defined in Public Resources Code Section 2621.5(a) would be required.

SEISMIC HAZARD ZONES

Liquefaction
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

N

0 FEET 4000
0 METERS 2000

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2012

ENGEO — Expect Excellence

SEISMIC HAZARD ZONES AND EARTHQUAKE FAULT ZONES MAP
CARDEN PROPERTY
HAYWARD, CALIFORNIA

PROJECT NO.: 4920.003.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: EH

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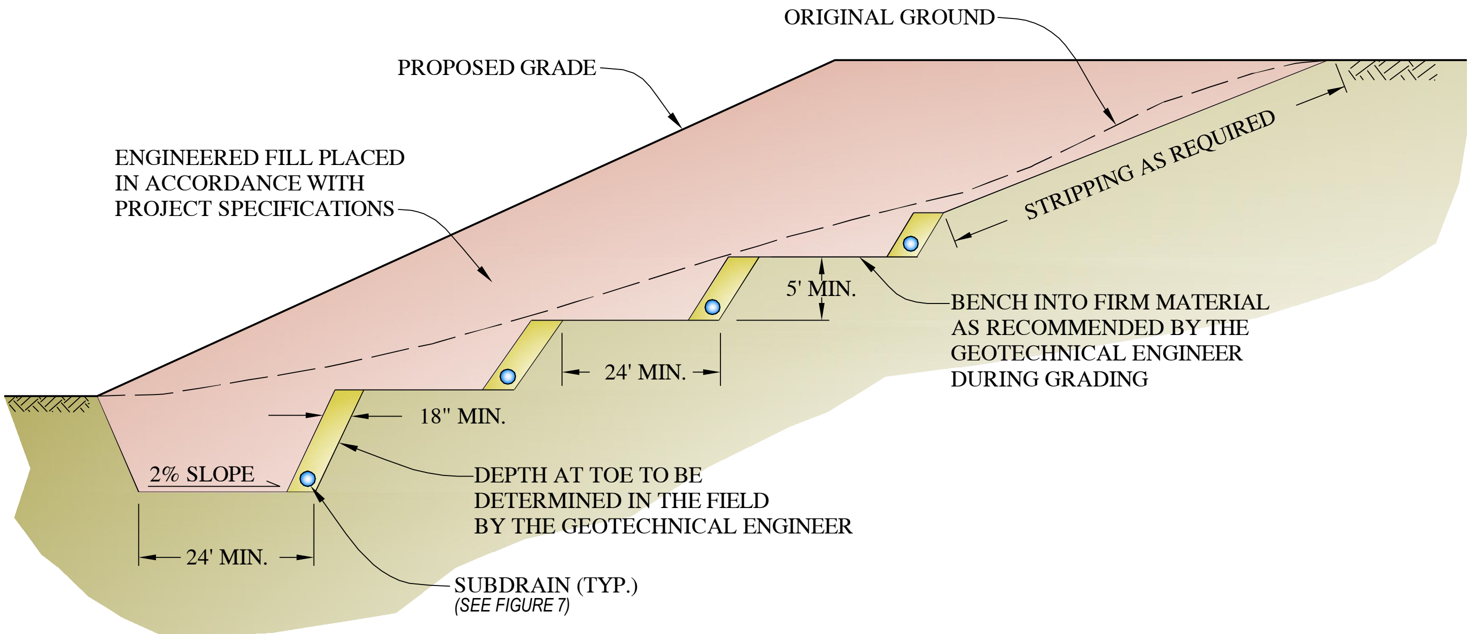
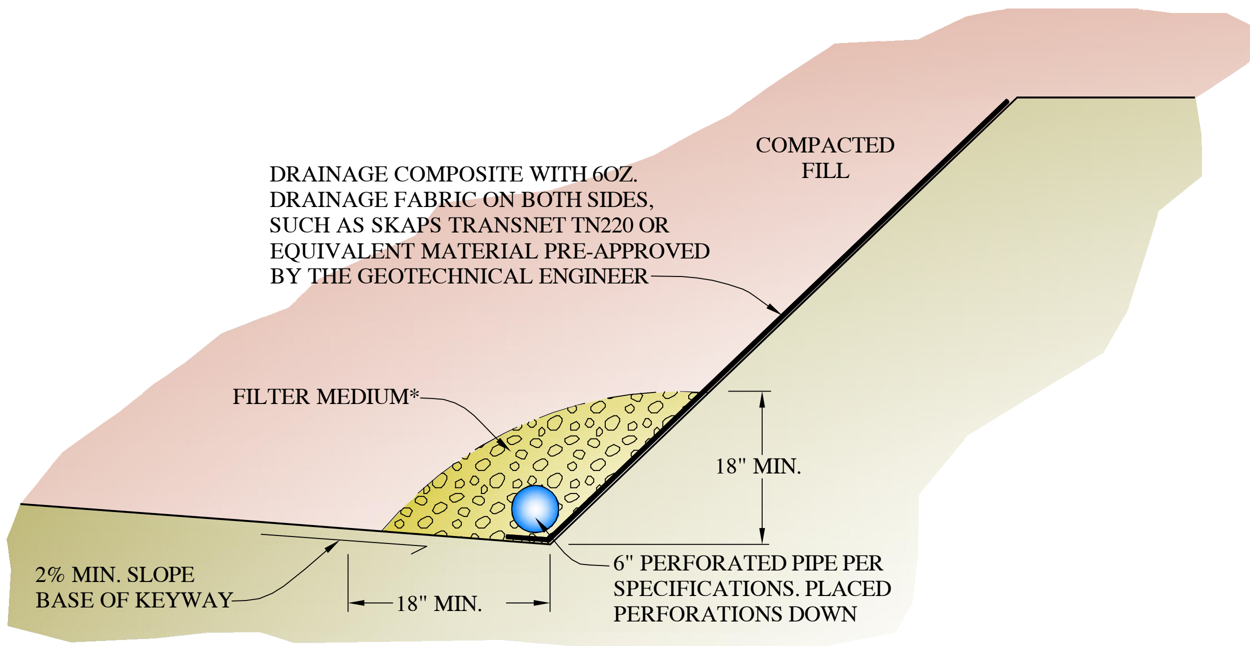
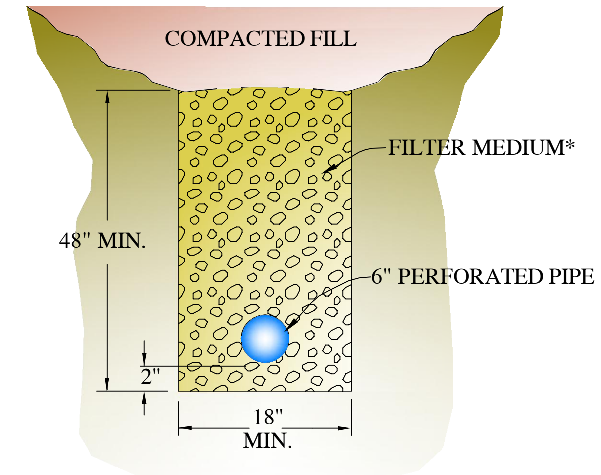


ENGEO Expect Excellence	TYPICAL SUBDRAIN DETAILS CARDEN PROPERTY HAYWARD, CALIFORNIA	PROJECT NO.: 4920.003.000 SCALE: NO SCALE DRAWN BY: SRP CHECKED BY: EH	FIGURE NO. 6

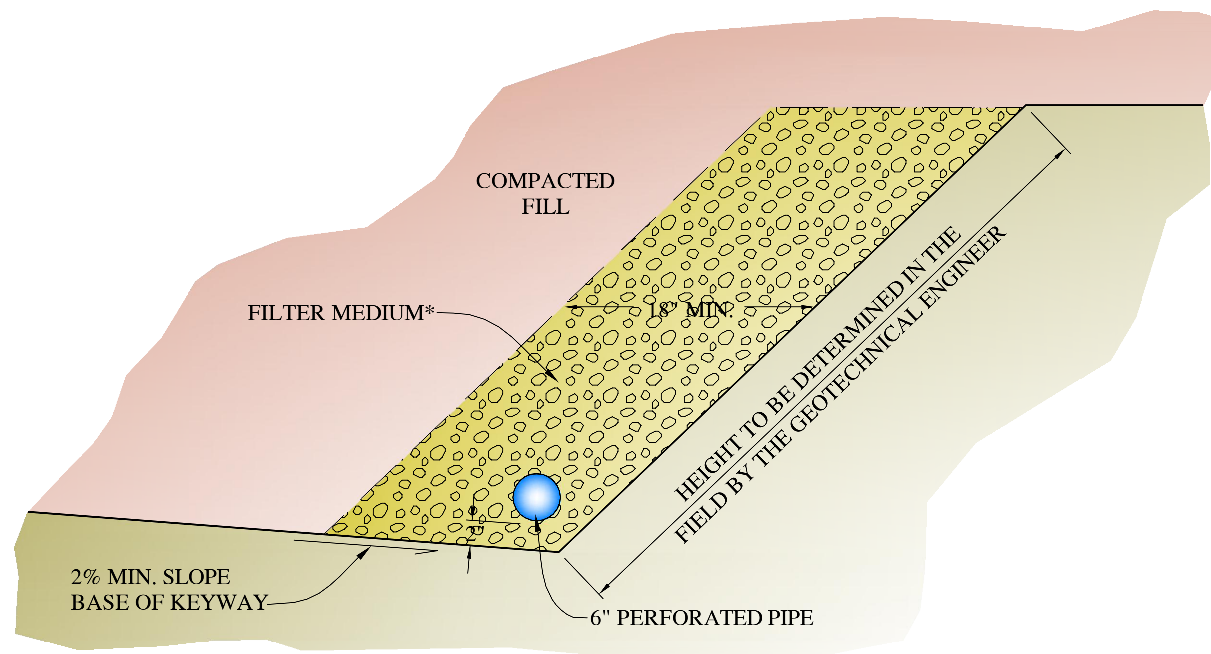
ORIGINAL FIGURE PRINTED IN COLOR



KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN



KEYWAY SUBDRAIN - OPTION 2

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90–100
3/8"	40–100
#4	25–40
#8	18–33
#30	5–15
#50	0–7
#200	0–3

ALTERNATIVE B

CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632)	180 lbs
MASS PER UNIT AREA (ASTM D-4751)	6 oz/yd^2
APPARENT OPENING SIZE (ASTM D-4751)	70–100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D-4833)	80 lbs

NOTES:

1. *ALL PIPE JOINTS SHALL BE GLUED*
2. *ALL PERFORATED PIPE PLACED PERFORATIONS DOWN*
3. *1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES*

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TYPICAL SUBDRAIN DETAILS	PROJECT NO.: 4920.003.000	FIGURE NO.
CARDEN PROPERTY	SCALE: NO SCALE	7
HAYWARD, CALIFORNIA	DRAWN BY: SRP / CHECKED BY: EH	