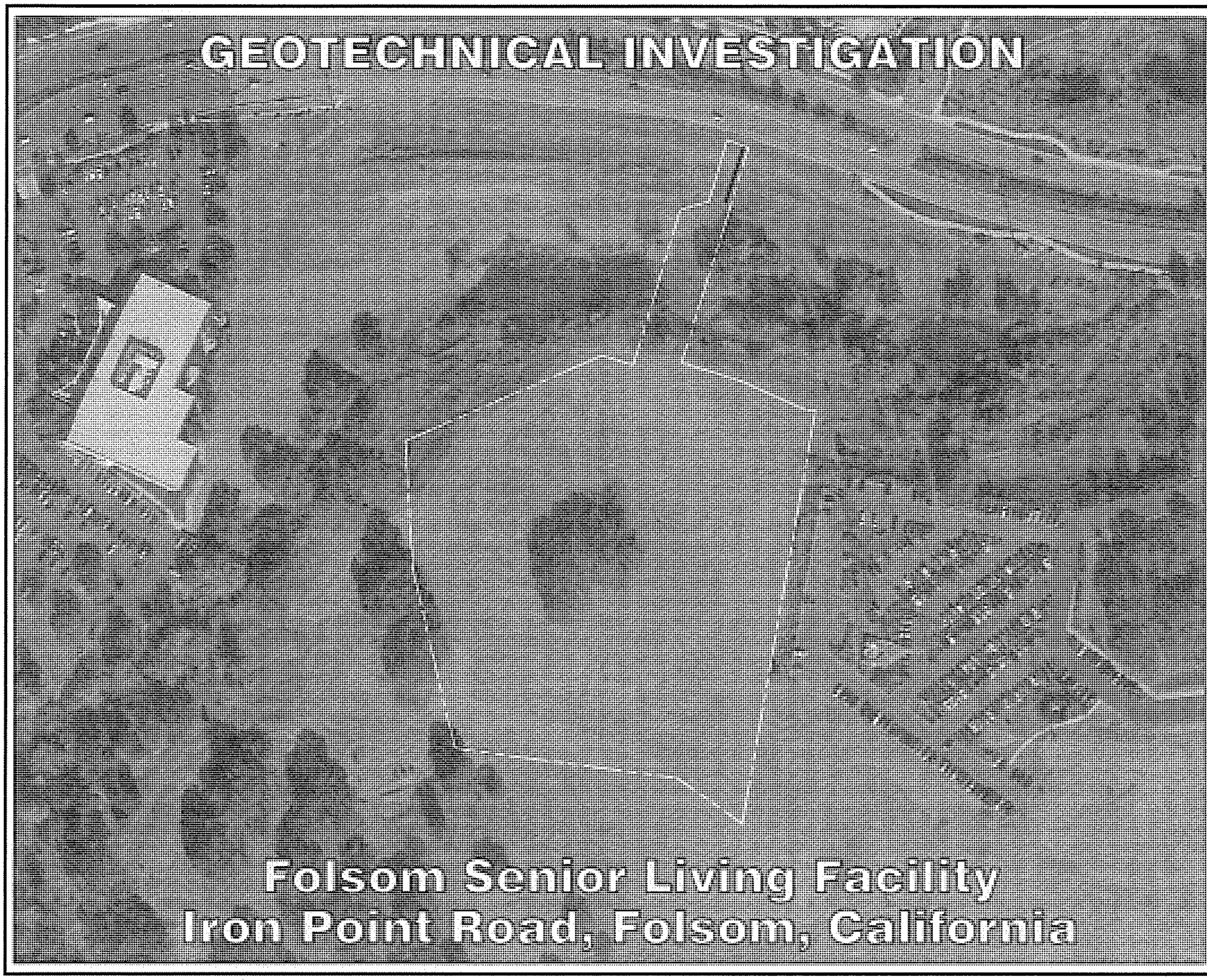
Appendix E

Geotechnical Investigation Report



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Geocon Project No.: S1367-05-01

July 2017

GEOCON
CONSULTANTS, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. S1367-05-01
July 14, 2017

Allison Emmons
The Wolff Company
6710 E. Camelback Road, Suite 100
Scottsdale, Arizona 85251

Subject: GEOTECHNICAL INVESTIGATION
FOLSOM SENIOR LIVING FACILITY
IRON POINT ROAD
FOLSOM, CALIFORNIA

Dear Ms. Emmons:

In accordance with your authorization, we have prepared this geotechnical investigation report for the subject project located on the south side of Iron Point Road near the Oak Avenue Parkway intersection and north of U.S. Highway 50 in Folsom, California.

The accompanying report presents our findings, conclusions, and recommendations regarding geotechnical aspects of designing and constructing the project as presently proposed. In our opinion, no adverse geotechnical conditions were encountered that would preclude development at the site provided the recommendations contained in this report are incorporated into the design and construction of the project.

Please contact us if you have any questions concerning the contents of this report or if we may be of further service.

Sincerely,

GEOCON CONSULTANTS, INC.

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GEOTECHNICAL INVESTIGATION

1.0 PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed senior living facility located on the south side of Iron Point Road near the Oak Avenue Parkway intersection and north of U.S. Highway 50 in Folsom, California. The approximate location of the project is depicted on the Vicinity Map, Figure 1.

The purpose of our geotechnical investigation was to observe and sample the subsurface conditions encountered at the site and provide conclusions and recommendations relative to the geotechnical aspects of constructing the project as presently proposed.

To prepare this report, we performed the following scope of services:

- Performed a limited geologic and geotechnical literature review to aid in evaluating the geologic and geotechnical conditions present at the site. A list of referenced material is included in Section 10.0 of this report.
- Performed a site reconnaissance to review project limits, determine exploration equipment access, and mark exploratory excavation locations for subsequent utility clearance.
- Notified subscribing utility companies via Underground Service Alert at least 48 hours (as required by law) prior to performing exploratory test pits at the site.
- Performed seven exploratory test pits (TP-1 through TP-7) to depths ranging from approximately 3½ to 10 feet using a rubber-tire John Deere 310L backhoe equipped with an 18-inch-wide bucket with rock teeth.
- Obtained soil and rock samples at periodic intervals from the test pits for classification and subsequent laboratory testing.
- Logged the test pits in accordance with the Unified Soil Classification System (USCS).
- Upon completion, backfilled test pits with the excavated soil and tamped with the backhoe bucket. Compaction testing was not performed.
- Performed laboratory tests on selected soil samples to evaluate pertinent geotechnical parameters.
- Prepared this report summarizing our findings, conclusions, and recommendations regarding the geotechnical aspects of designing and constructing the project as presently proposed.

Details of our field exploration program including test pit logs are presented in Appendix A. Approximate locations of exploratory borings and test pits are shown on the Site Plan/Geologic Map, Figure 2, and Proposed Development Plan, Figure 3. Details of our laboratory testing program and test results are summarized in Appendix B.

2.0 SITE AND PROJECT DESCRIPTION

The overall project site is located on the south side of Iron Point Road near the Oak Avenue Parkway intersection, approximately 0.1 miles north of U.S. Highway 50 in Folsom, California (see Vicinity Map, Figure 1). The site is bounded by a pond and marshy area to the north (Photo 5), an asphalt concrete (AC) paved parking lot Kaiser facility to the east, undeveloped land and U.S. Highway 50 to the south, and undeveloped land to the west. The approximate 6-acre site is currently undeveloped. At the time of our investigation, the site was vegetated with a moderate growth of annual grasses and trees along the perimeter and center.

The *Site Plan* provided by the client, dated June 9, 2017, presents topographic information for the site. Based on the topographic information, site elevations range from approximately 340 feet above mean sea level (MSL) near the Kaiser parking lot and gently slopes downward to the north, west, and south to an elevation of approximately 300 feet MSL at the pond and marshy area within the northern portion of the site. The site is generally a hilltop area with surrounding gentle downward slopes inclined at approximately 11H:1V in the portion south of the pond and marshy area and a slope inclined at approximately 7H:1V across the pond within the northern portion of the site with some locally steeper areas. Current site topography (one-foot contours) is shown on the Site Plan/Geologic Map, Figure 2 and Proposed Development Plan, Figure 3. We did not observe any overt evidence or conditions indicative of slope instability at the time of our field exploration.

We understand that the proposed project consists of constructing two five-story senior living residential buildings (approximately 70,000 and 84,000 square feet) and a 19,000-square foot single-story kitchen/common area building. The buildings will be arranged in a circular fashion encompassing an interior courtyard area. The buildings will likely be of wood- or steel-framed construction supported on conventional shallow foundations with interior concrete slabs-on-grade. Other improvements will likely include underground utility infrastructure, concrete flatwork, and paved parking/driveway areas. Pavement will consist of both asphalt concrete and rigid Portland cement concrete (PCC). The entrance driveway will require an arched culvert or box culvert to span the pond and marshy area to the north of the site. Given the rolling topography of the site, we anticipate site grading will consist of cuts and fills on the order of 10 feet or less. The site configuration and locations of existing and proposed improvements are shown on the Proposed Development Plan, Figure 3.

3.0 SOIL AND GEOLOGIC CONDITIONS

We identified soil conditions by observing and sampling exploratory test pits and reviewing the referenced geologic literature (Section 10.0). Site geology consists of existing fill within the northern portion of the site north of the pond and Jurassic-age Gopher Ridge Volcanics (Jgo) and Salt Springs Slate bedrock (Jss) as shown in the Regional Geology Map, Figure 4. Estimated lateral extent of the fill is shown on the Site Plan/Geologic Map, Figure 2, and a generalized geologic cross-section is presented as Figure 5. Soil descriptions below include the USCS symbol where applicable.

3.1 Fill (Qf)

In Test Pit TP-7, located in the northern portion of the site between the pond and Iron Point Road, we encountered existing fill up to approximately 7½ feet thick. Based on the conditions encountered in our test pits, the fill material generally consists of a mixture of slate fragments, gravel, cobbles, and boulders varying in dimension with a clayey silt (ML) soil matrix. As shown in Photo 3, boulders ranging in size from 1 foot to approximately 2½ feet were encountered within the fill. We did not observe existing fill in proposed building areas.

3.2 Residual Soil (Unmapped)

Long-term, in-place weathering of bedrock in the project area has produced a mantle of residual soil overlying the bedrock. The residual soil generally consists of clayey silt (ML) with variable amounts of gravel and cobble (Photos 1 and 2). Residual soil also contains varying amounts of plant roots and other decomposed plant organic material. The thickness of the residual soil varies from approximately 1½ to 2½ feet within our test pits.

3.3 Gopher Ridge Volcanics and Salt Springs Slate Bedrock – (Jgo and Jss)

Below the residual soil and fill (where present), bedrock at the site consists of Jurassic-age weathered metavolcanic rock mapped as Gopher Ridge Volcanics and Salt Springs Slate. These formations generally consist of tan to light grayish brown rock that is moderately to highly weathered and fractured (Photo 2), and grayish brown slate that is moderately to highly weathered and fractured, respectively (Photo 6). Clay and silt infilling in the fractures is common. In general, these formations excavate as clayey gravel (GC) with variable amounts of cobble and boulder-sized rock fragments. Weathering generally decreases with depth and moderate to difficult excavation conditions prevail below about 3 to 10 feet into the rock, depending on location. Based on our experience on nearby projects, these formations generally break down to cobble- and small boulder-sized fragments (12 to 30 inches) when excavated; however, zones of less weathered rock are common and are more resistant to breaking down.

Subsurface conditions described here are generalized. The test pit logs (Figures A2 through A8) detail soil/rock type, color, moisture, consistency, and USCS classification of the materials encountered at specific locations and elevations.

4.0 GROUNDWATER / SEEPAGE

We encountered seepage in Test Pit TP-4 at approximately 10 feet (Photo 4) on June 28, 2017. It is likely that the seepage is associated with the adjacent pond just north TP-4.

Review of the California Department of Water Resources Groundwater Information Center (GIC) Interactive Map (2017) indicates the average groundwater depth from the ground surface approximately two miles west of the site is approximately 150 feet MSL. Given the average elevation of the site at 320 feet, groundwater is approximately at a depth of 170 feet at the site.

Based on our experience in the area, we expect perched groundwater/seepage may develop at variable depths generally at the contacts between surficial soils (residual soil and fill, where present) and formational materials (bedrock), especially during winter and spring. Seepage can also occur within formational material based on the degree of weathering, fracturing, jointing, and bedding as was observed in TP-4 at approximately 10 feet during our investigation. Conclusions, recommendations, and construction considerations with respect to seepage are included in subsequent sections of this report. It should be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors. Depth to groundwater can also vary significantly due to localized pumping, irrigation practices, and seasonal fluctuations.

5.0 SEISMICITY AND GEOLOGIC HAZARDS

5.1 Surface Fault Rupture

The numerous faults in Northern California include active, potentially active, and inactive faults. The criteria for these major groups were developed by the California Geological Survey for the Alquist-Priolo Earthquake Fault Zone (APEFZ) Program (Bryant and Hart, 2007). By definition, an active fault is one that has had surface displacement within the last 11,000 years. A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known movement within the past 11,000 years. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not located within a currently established APEFZ. Based on our reconnaissance, evidence obtained in test pits, and our review of geologic maps and reports, no active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed project is considered low. The site, however, could be subjected to ground shaking in the event of an earthquake on one of the many active Northern California faults.

In order to determine the distance of known active faults within 50 miles of the site, we used the computer program *EQFAULT* (Blake, 2000) and reviewed the 2010 Fault Activity Map of California (Jennings and Bryant, 2010). Results are summarized in Table 5.1.

TABLE 5.1
REGIONAL FAULT SUMMARY

Fault Name	Approximate Distance From Site (miles)	Maximum Moment Magnitude (Mw)
Foothills Fault System	2.6	6.5
Great Valley 4	46.3	6.6
Great Valley 3	46.7	6.8
Great Valley 5	47.3	6.5

5.2 Seismic Hazard Analysis

Seismic design of the structures should be performed in accordance with the provisions of the 2016 California Building Code (CBC) (International Code Council, 2016) which is based on the American Society of Civil Engineers (ASCE) publication: *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-10). We used the United States Geological Survey (USGS) web application *US Seismic Design Maps* (http://geohazards.usgs.gov/designmaps/us/application.php) to evaluate site-specific seismic design parameters in accordance with the 2016 CBC/ASCE 7-10. Results are summarized in Table 5.2.1. The values presented are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 5.2.1
2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC / ASCE 7-10 Reference
Site Class	C	1613.3.2 / Table 20.3-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.474g	Figure 1613.3.1(1) / Figure 22-1
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.241g	Figure 1613.3.1(2) / Figure 22-2
Site Coefficient, F_A	1.200	Table 1613.3.3(1) / Table 11.4-1
Site Coefficient, F_V	1.559	Table 1613.3.3(2) / Table 11.4-2
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.569g	Eq. 16-37 / Eq. 11.4-1
Site Class Modified MCE_R Spectral Response Acceleration (1 sec), S_{M1}	0.376g	Eq. 16-38 / Eq. 11.4-2
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.379g	Eq. 16-39 / Eq. 11.4-3
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.251g	Eq. 16-40 / Eq. 11.4-4

Table 5.2.2 presents additional seismic design parameters for projects with Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_G).

TABLE 5.2.2
2016 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.151g	Figure 22-7
Site Coefficient, F_{PGA}	1.200	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.181g	Section 11.8.3 (Eq. 11.8-1)

Conformance to the criteria presented in Tables 5.2.1 and 5.2.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid structural damage, since such design may be economically prohibitive.

5.3 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with earthquakes. Primary factors that trigger liquefaction are: strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly graded sands and silty sands), and saturated soil conditions (shallow groundwater). Due to the increasing overburden pressure with depth, liquefaction is generally limited to the upper 50 feet of a soil profile. Based on the subsurface conditions encountered at the site, including shallow bedrock and the lack of shallow groundwater, liquefaction potential is not a hazard for the site.

5.4 Landslides and Slope Stability

We did not observe localized slumping, deep-seated slope failures, debris slides/flows, or conditions indicative of active landslides, such as headscarps or toe bulges during our investigation. In addition, we did not observe these features on adjacent properties that may affect the site. The natural slopes (inclinations of approximately 11H:1V and 7H:1V) appear to be performing well without evidence of global instability. Provided that site grading is performed in accordance with the recommendations in this report, we consider the potential for future slope instability to be low.

The stability of cut slopes in metavolcanic bedrock material is generally governed by the degree of weathering and fracturing. Cut slopes may expose localized weak zones or fracture orientation that is prone to sloughing or erosion. We recommend that all cut slopes (if any) be observed by our engineering geologist during grading to determine if adversely oriented bedding planes exist. Recommendations for mitigation, if necessary, can be provided at that time.

5.5 Expansive Soil

Laboratory Expansion Index test results for clayey soils at the site indicate low expansion potential. Mitigation with respect to expansive soils are not necessary for this project.

5.6 Soil Corrosion Screening

Selected soil bulk samples were analyzed for soil corrosion parameters (minimum resistivity, pH, chloride, and sulfate content). Results are presented in Appendix B.

5.7 Naturally Occurring Asbestos (NOA)

Based on the *Relative Likelihood for the Presence of Naturally Occurring Asbestos in Eastern Sacramento County, California* (CGS Special Report 192, 2006), the site is located in an area mapped as "Moderately Likely to Contain NOA." The predominant rock type present at the site (metamorphosed mafic volcanic rock), which is one of the rock types in which NOA may occur.

A geologic evaluation for the presence of NOA in accordance with Title 17 California Code of Regulations (CCR), Section 93105, subsection (c) is beyond the scope of our current study. However, because of the reported occurrences of NOA in the area, the Sacramento Metropolitan Air Quality Management District (SMAQMD) requires that properties located on the Gopher Ridge Volcanics formation, or soils derived from there, comply with the CARB July 29, 2002, Air Toxic Control Measure (ATCM) for construction, grading, quarrying and surface mining operations that may disturb natural occurrences of asbestos as outlined in 17 CCR 93105 unless a geologic evaluation is performed that demonstrates that NOA is not present. The ATCM generally requires that an Asbestos Dust Mitigation Plan (ADMP) be prepared for projects where NOA may be encountered, which outlines mitigation measures to be employed during and after construction to prevent airborne asbestos dust emissions. In our experience, the cost to perform the geologic evaluation for this project to demonstrate that NOA is not present exceeds the cost to prepare an ADMP and perform the required mitigation measures during construction. We can assist the client with this matter further, upon request.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered during our investigation that would preclude construction of improvements at the site as planned, provided the recommendations contained in this report are incorporated into the design and construction of the project.

6.1.2 Conclusions and recommendations provided in this report are based on our review of referenced literature, analysis of data obtained from our field exploration, laboratory testing program, and our understanding of the proposed development at this time.

6.1.3 We should review the project plans as they develop further, provide engineering consultation as needed during final design, and perform geotechnical observation and testing services during construction.

6.2 Excavation Characteristics/Rippability

6.2.1 Excavation characteristics will vary at the site depending on location and excavation depths. Table 6.2 summarizes anticipated excavation characteristics in each geologic material identified at the site.

TABLE 6.2
ANTICIPATED EXCAVATION CHARACTERISTICS

Geologic Unit[1]	Excavation Characteristics
Fill (Qf)	Existing fill generally consists of a mixture of gravel, cobbles, and boulders with a clayey silty soil matrix (Photo 3). We anticipate moderate excavation effort with conventional, heavy-duty grading equipment. The presence of oversize rock (greater than 6 inches in maximum dimension) will increase excavation difficulty.
Residual Soil (unmapped)	Residual soil generally consists of clayey silt with variable amounts of gravel and cobble (Photos 1 and 2). The residual soil also contains varying amounts of plant roots and other decomposed plant organic material. We anticipate moderate excavation effort with conventional, heavy-duty grading equipment. The presence of oversize rock (greater than 6 inches in maximum dimension) will increase excavation difficulty.
Gopher Ridge Volcanics (Jgo) / Salt Springs Slate (Jss)	Gopher Ridge Volcanics and Salt Springs Slate generally consist of very dense tan to light grayish brown rock that is slightly weathered and fractured (Photo 2), and grayish brown slate that is slightly weathered and fractured, respectively (Photo 6). Clay and silt infilling in the fractures is common. Weathering generally decreases with depth, and moderate to heavy ripping will likely be required at depths of 3 to 10 feet into the rock, depending on location. Use of a dozer-mounted impact ripper may be required for deeper excavations. This formation generally breaks down to cobble- and small boulder-sized fragments (12 to 24 inches) when excavated; however, zones of less weathered rock are common and are more resistant to breaking down. Therefore, large boulder-sized fragments (24 inches and larger) may be generated.
Notes: 1. See Site Plan/Geology Map, Figure 2, for approximate lateral extents of geologic units.	

6.2.2 Temporary excavation slopes must meet Cal-OSHA requirements as appropriate. Trench wall sloping, benching, the use of trench shields, and the placement of trench spoils should conform to the latest applicable Cal-OSHA standards. The contractor should have a Cal-OSHA-approved "competent person" onsite during excavation and pipe placement to evaluate trench conditions and to make appropriate recommendations where necessary. It is the contractor's responsibility to provide sufficient and safe excavation support as well as protecting nearby utilities, structures, and other improvements which may be damaged by earth movements.

6.3 Permanent Cut and Fill Slopes

6.3.1 Permanent cut and fill slopes should be constructed no steeper than 2H:1V. Cut slopes in formational material may expose localized weak zones or adverse fracture orientation that would be prone to sloughing or erosion. We recommend that all cut slopes (if any) be observed by our engineering geologist during grading to determine if adversely oriented fractures exist. Recommendations for mitigation, if necessary, can be provided at that time.

6.3.2 To mitigate potential erosion, slopes should be vegetated as soon as possible and surface drainage should be directed away from the tops of slopes. Placing V-ditches across tops of slopes will aid in reducing the potential for surficial erosion.

6.4 Materials for Fill

6.4.1 Excavated soil and rock generated from cut operations at the site are suitable for use as engineered fill in structural areas provided they are examined and selectively placed during grading in accordance with the following recommendations:

- Deleterious material, material with greater than 3% organics, and debris should be exported from the site and not incorporated into structural fill.
- Fill material in areas with underground utilities, foundations, and areas within 5 feet of slope faces should consist of 6-inch-minus material with a sufficient amount of soil to provide adequate binder to reduce the potential for excavation caving.
- In other areas (general fill areas without utilities, foundations, and not within 5 feet of slope faces) rock or cementations larger than 6 inches but less than 2 feet in maximum dimension may be used. Rock or cementations greater than 2 feet in maximum dimension should not be used. This material should contain a sufficient amount of soil to fill void spaces between rocks and reduce rock nesting (concentrations of rock with void space).
- If sufficient soil fill materials are not present at the site to mix with onsite rock material, import of soil fill material will be necessary.

6.4.2 Import soil should be primarily granular with a "very low" expansion potential (Expansion Index (EI) less than 20), a Plasticity Index (PI) less than 15, contain sufficient binder to prevent caving when excavated, be free of organic material and construction debris, and not contain rock/cementations larger than 6 inches in greatest dimension.

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

6.5 Seepage, Groundwater, and Wet Weather Grading Considerations

6.5.1 Seasonal perched groundwater (seepage) could be present during grading especially if it occurs during winter or spring. Perched groundwater typically develops at the contact between fill/residual soil and formational material and can sometimes be present within fractures of the weathered formational material. Fill/residual soil derived from shallow excavations during perched groundwater conditions will likely need to be aerated/dried to achieve suitable moisture content for compaction. We should evaluate conditions in the field at the time of construction and evaluate the type, level, and extent of mitigation alternatives.

6.5.2 If grading commences in winter or spring, or in periods of precipitation, excavated and in-place soils will likely be wet. Earthwork contractors should be aware of the moisture-sensitivity of fine-grained soils that may result in subgrade instability and/or potential compaction difficulties. Earthwork operations in these conditions will likely be difficult with low productivity. Often, a period of at least one month of warm and dry weather is necessary to allow the site to dry sufficiently so that heavy grading equipment can operate effectively. If the construction schedule allows, we highly recommend performing earthwork construction during the seasonal dry months.

6.6 Grading

6.6.1 Earthwork operations should be observed and fills tested for recommended compaction and moisture content by a representative of Geocon. All cut slopes should be observed by our engineering geologist to check that conditions do not differ significantly from those anticipated. For example, if adverse bedrock bedding or characteristics such as large boulders are exposed, recommendations to mitigate this condition can be provided at that time.

6.6.2 References to relative compaction and optimum moisture content in this report are based on the latest American Society for Testing and Materials (ASTM) D1557 Test Procedure. Structural building pad areas should be considered as areas extending a minimum of 5 feet horizontally beyond the outside dimensions of buildings, including footings and overhangs carrying structural loads.

6.6.3 Site preparation should begin with the complete removal of existing surface vegetation, trees, debris, and existing fill (where present) (see Paragraph 6.7.4 for fill removal requirements). Existing trees and similar large vegetation and associated roots larger than 1 inch in diameter should be completely removed. Smaller roots may be left in-place as conditions warrant as evaluated by our representative. Surface vegetation consisting of grasses and other similar vegetation should be removed by stripping to a sufficient depth. We estimate required stripping depths will range from approximately 2 to 4 inches. The actual stripping depth should be determined based on site conditions prior to grading. Material generated during stripping is not suitable for use within 5 feet of structural building pads or engineered fill areas.

6.6.4 Excavations or depressions resulting from site clearing operations, or other existing excavations or depressions, should be restored with engineered fill in accordance with the recommendations of this report.

6.6.5 In general, where fill will be placed on slopes steeper than 5H:1V, we recommend that horizontal benches angled slightly into the slope be cut into competent formational material or existing fill on the slopes prior to placing fill. Benches should roughly parallel slope contours. These benches should extend at least 2 feet into competent material. In addition, a keyway should be cut into competent material at the base of the fill. In general, keyways should be at least 15 feet wide and extend at least 2 feet into competent material. Bench and keyway criteria may need revision during construction based on the actual materials encountered and grading performed in the field.

6.6.6 To reduce potential for differential settlement of planned structures, the cut portion of cut-fill transition building pads, if encountered, should be undercut to at least the depth of the fill, not to exceed 3 feet, and replaced with properly compacted fill soils. The undercut should extend at least 5 feet beyond the structure perimeter.

6.6.7 Where rock or other formational material is exposed at finish grade in cut areas, if any, consideration should be given to undercutting at least 3 feet and replacing the material with compacted soil fill to facilitate construction of foundations, landscaping, and shallow improvements.

6.6.8 Areas to receive fill, or areas left at-grade should be scarified at least 8 inches, uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 90% relative compaction. Scarification in exposed, hard bedrock areas is not required.

6.6.9 Engineered fill should be placed and compacted in horizontal lifts not exceeding 8 inches (loose thickness) and brought to final design elevations. Each lift should be moisture-conditioned at or above optimum moisture content, and compacted to at least 90% relative compaction. Fills containing rocks larger than 6 inches should be placed and proof-rolled under our observation.

6.6.10 Fill slopes should be built such that soils are uniformly compacted to at least 90% relative compaction to the face of the completed slope. This will likely require overbuilding the slopes and cutting them back.

6.6.11 The top 6 inches of final vehicular pavement subgrade, whether completed at-grade, by excavation, or by filling, should be uniformly moisture-conditioned at or above optimum moisture content and compacted to at least 95% relative compaction. Final pavement subgrade should be finished to a smooth, unyielding surface. We further recommend proof-rolling the subgrade with a loaded water truck (or similar equipment with high contact pressure) under our observation to verify the stability of the subgrade prior to placing aggregate base (AB).

6.6.12 Underground utility trenches should be backfilled with properly compacted material. Pipe bedding, shading, and trench backfill should conform to the requirements of the appropriate utility authority. Soil excavated from trenches should be adequate for use as general backfill above shading provided it does not contain deleterious matter, vegetation or rock/cementations larger than 6 inches in maximum dimension. Trench backfill should be placed in loose lifts not exceeding 12 inches. Lifts should be compacted to a minimum of 90% relative compaction at or above optimum moisture content. Compaction should be performed by mechanical means only; jetting of trench backfill is not recommended.

6.7 Foundations

6.7.1 Provided the site is graded in accordance with the recommendations of this report, the proposed buildings may be supported on conventional shallow foundations bearing on undisturbed weathered rock or engineered fill.

6.7.2 To reduce potential for seasonal moisture variations beneath buildings, foundations should consist of continuous perimeter strip footings with isolated interior spread footings. Perimeter strip footings should be continuous around the entire perimeter of the structure without breaks or discontinuities. Strip and spread footings should be embedded at least 18 inches below lowest adjacent pad grade.

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

6.7.4 Shallow foundations may be designed for an allowable bearing capacity of 3,000 pounds per square foot (psf) for dead plus live loads with a one-third increase for transient loads, including wind and seismic.

6.7.5 The allowable passive pressure used to resist lateral movement of the footings may be assumed to be equal to a fluid weighing 350 pounds per cubic foot (pcf). The allowable coefficient of friction to resist sliding is 0.35 for concrete against soil. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

6.7.6 Foundations designed in accordance with the recommendations above should experience total settlements of approximately 1 inch or less and differential settlements of approximately ½ inch or less over a horizontal distance of approximately 50 feet due to building loads. The majority of the settlement will be immediate and will occur as loads are applied during construction.

6.7.7 Continuous footings should be reinforced with at least four No. 4 reinforcement bars, two each placed near the top and bottom of the footing to allow footings to span isolated soil irregularities. The reinforcement recommended above is for soil characteristics only and is not intended to replace reinforcement required for structural considerations. The project structural engineer should evaluate the need for additional reinforcement.

6.7.8 Foundations for pole structures such as light poles may be designed using formulae from the 2016 CBC, Section 1807.3. Assuming ½-inch deflection at the ground surface is acceptable, an allowable lateral soil-bearing pressure (CBC parameters S_1 in equation 18-1 and S_3 in equations 18-2 and 18-3) of 300 psf per foot of depth may be used.

6.7.9 A Geocon representative should observe all foundation excavations prior to placing reinforcing steel or concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.8 Interior Slabs-on-Grade

6.8.1 Conventional interior concrete slabs-on-grade are suitable for the building pads prepared as recommended in this report. Slab thickness and reinforcement should be determined by the structural engineer based on anticipated loading. However, at a minimum, slabs should be at least 4 inches thick and reinforced with No. 4 reinforcing bars placed 18 inches on center, each way. Structural requirements may require additional reinforcement or thicker concrete slabs.

6.8.2 Migration of moisture through concrete slabs or moisture otherwise released from slabs is not a geotechnical issue. However, for the convenience of the owner, we are providing the following general suggestions for consideration by the owner, architect, structural engineer, and contractor. The suggested procedures may reduce the potential for moisture-related floor covering failures on concrete slabs-on-grade, but moisture problems may still occur even if the procedures are followed. If more detailed recommendations are desired, we recommend consulting a specialist in this field.

6.8.3 Where floor coverings are planned, a minimum 10-mil-thick vapor barrier meeting ASTM E1745-97 Class C requirements may be placed directly below the slab, without a sand cushion. To reduce the potential for punctures, a higher quality vapor barrier (15 mil, Class A or B) may be used. The vapor barrier, if used, should extend to the edges of the slab and should be sealed at all seams and penetrations.

6.8.4 At least 4 inches of ½- or ¾-inch crushed rock, with no more than 5 percent passing the No. 200 sieve, may be placed below the vapor barrier to serve as a capillary break.

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

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

6.9 Retaining Walls and Lateral Loads

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

TABLE 6.9
RECOMMENDED LATERAL EARTH PRESSURES

Condition	Equivalent Fluid Density
Active	35 pcf
At-rest	55 pcf

6.9.2 Unrestrained walls should be designed using the active case. Unrestrained walls are those that are allowed to rotate more than 0.001H (where H is the height of the wall). Walls restrained from movement (such as basement walls) should be designed using the at-rest case. The above soil pressures assume level backfill under drained conditions within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall and into the backfill.

6.9.3 Retaining wall foundations with bottoms at least 18 inches below lowest adjacent grade may be designed using the allowable bearing capacity provided in Paragraph 6.7.4 of this report. To resist lateral movement of retaining wall foundations, an allowable passive earth pressure equivalent to a fluid density of 350 pcf for footings or shear keys poured neat against properly compacted engineered fill soils or undisturbed natural soils. This allowable passive pressure is based on the assumption that a horizontal surface extends at least 5 feet or three times the depth of the footing or shear key, whichever is greater, beyond the face of the retaining wall foundation. If this surface is not protected by floor slabs or pavement, the upper 12 inches of material should not be included in the design for lateral resistance. An allowable friction coefficient of 0.35 may be used for resistance to sliding between soil and concrete. Combined passive resistance and friction may be utilized for design provided that the frictional resistance is reduced by 50%.

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

6.10 Hot Mix Asphalt

6.10.1 We performed Resistance-Value (R-Value) testing on a composite near-surface bulk soil sample (TP-1,2 Bulk) in accordance with California Test Method 301. Our testing resulted in an R-Value of 20.

6.10.2 We recommend the following alternative hot mix asphalt (HMA) pavement sections for design to establish subgrade elevations for pavement areas. The project civil engineer should determine the appropriate Traffic Index (TI) for pavement design. We have provided pavement sections comprised of HMA over Class 2 aggregate base (AB) based on a 20-year service life for various TIs ranging from 5.0 to 7.0. We can provide additional sections based on other TIs if necessary.

TABLE 6.10
FLEXIBLE PAVEMENT SECTIONS

	Traffic Index				
	5.0	5.5	6.0	6.5	7.0
HMA, inches	2.5	3.0	3.0	4.0	4.0
AB, inches	8.5	9.0	10.0	11.0	11.5
Total Section Thickness, inches	11.0	12.0	13.0	15.0	15.5

6.10.3 The recommended alternative pavement sections are based on the following assumptions:

1. Pavement subgrade soil has an R-Value of 20.
2. Class 2 AB has a minimum R-Value of 78 and meets the requirements of Section 26 of Caltrans' *Standard Specifications*.
3. Class 2 AB and the top 12 inches of subgrade are compacted to 95% or higher relative compaction at or near optimum moisture content.

6.10.4 To reduce the potential for water from landscaped areas migrating under pavement into the AB, consideration should be given to using full-depth curbs in areas where pavement abuts irrigated landscaping. The full-depth curbs should extend at least 6 inches or more into the soil subgrade beneath the AB. Alternatively, modified drop-inlets that contain weep-holes may be used to encourage accumulated water to drain from beneath the pavement.

6.10.5 Asphalt pavement section recommendations for driveways and parking areas are based on the design procedures of Caltrans' Highway *Design Manual* (Design Manual), Chapter 600, updated May 7, 2012. It should be noted that most rational pavement design procedures are based on projected street or highway traffic conditions and, hence, may not be representative of vehicular loading that occurs in parking lots and driveways. Pavement proximity to landscape irrigation, reduced traffic speed, and short turning radii increase the potential for pavement distress to occur in parking lots even though the volume of traffic is significantly less than that of an adjacent street. The Design Manual indicates that the resulting pavement sections for parking lots are "minimized to keep initial costs down but are reasonable because additional AC surfacing can be added later, if needed, and generally without incurring traffic hazards or traffic handling problems." It is generally not economically feasible to design and construct the entire parking lot and driveways for the unique loading conditions previously described. Periodic maintenance of the pavement in these areas, therefore, should be anticipated.

6.11 Rigid Concrete Pavement

6.11.1 If rigid PCC pavement is used in automobile and truck traffic areas or in front of trash enclosures, we recommend that the concrete be at least 6 inches thick and be underlain by at least 6 inches of Class 2 AB meeting the requirements of Section 26 of Caltrans' *Standard Specifications* and compacted to at least 95% relative compaction. Subgrade soils should be prepared and compacted in accordance with the recommendations of this report.

6.11.2 PCC should have a minimum 28-day compressive strength of 3,500 pounds per square inch (psi). Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. It would be advantageous to provide minimal reinforcement, such as No. 3 steel bars placed 18 inches on center in both horizontal directions to help control cracking.

6.12 Concrete Flatwork

6.12.1 Sidewalk, curb, gutter, and driveway encroachments within City of Folsom right-of-way should be designed and constructed in accordance with the latest City of Folsom improvement standards, as applicable.

6.12.2 Onsite concrete flatwork, such as pedestrian walks, patios, and courtyards, should be underlain by at least 4 inches of Class 2 AB compacted to at least 95% relative compaction at or near optimum moisture content. Prior to placing the AB, the top 6 inches of soil subgrade soil should be uniformly moisture-conditioned at or above optimum moisture content and compacted to 95% relative compaction.

6.12.3 Adequate construction and crack control joints should be used to control cracking inherent in concrete construction. Assuming flatwork is 4 inches thick, we recommend using a maximum control joint spacing of 8 feet in each direction. It would be advantageous to provide reinforcement, such as No. 3 reinforcing bars placed 24 inches on center in both horizontal directions to help control cracking.

6.13 Drainage

6.13.1 Adequate drainage is imperative to reduce the potential for differential soil movement, detrimental soil expansion, erosion, and subsurface seepage. Care should be taken to properly grade the finished surface around the building after the structure and other improvements are in place, so that drainage water is directed away from building and toward the street or other appropriate drainage facilities. Final soil grade should slope a minimum of 2% away from structures.

6.13.2 Experience has shown that even with these provisions, subsurface seepage may develop in areas where no such water conditions existed prior to site development. This is particularly true where a substantial increase in surface water infiltration has resulted from an increase in landscape irrigation.

7.0 CULVERT FOUNDATION RECOMMENDATIONS

A bottomless arched culvert or reinforced concrete box culvert is planned for roadway access to the site from Iron Point Road.

7.1 Bottomless Arched Culvert

7.1.1 Arched culvert foundation construction and culvert installation should be performed by contractors experienced with the pre-engineered product used. Installation methods and procedures should conform to manufacturer specifications.

7.1.2 Foundations for the arched culvert should bear entirely on undisturbed weathered rock. Given the presence of existing fill on the north side of the drainage, this may require localized deepening. We recommend that culvert foundations be embedded at least 2 feet into firm, undisturbed soil/rock or 2 feet below the drainage channel invert elevation, whichever is shallower. Our representative should verify footing embedment depth in the field during construction.

7.1.3 Footings meeting the above recommendations may be designed using an allowable bearing capacity of 3,000 psf for dead plus live loads. This value may be increased by one-third when considering transient loads due to wind, seismic forces or vehicle loads. The weight of foundation concrete below grade may be disregarded in sizing computations. Footing reinforcement should be designed by the culvert manufacturer or the project structural engineer, as appropriate.

7.1.4 Foundation excavations will likely require dewatering. Wing-wall back-drainage should conform to manufacturer specifications. Areas behind pre-cast wingwalls shall be sufficiently excavated to allow clear placement of wingwalls and anchors. Backfill material behind wingwalls and above the culvert should conform to manufacturer specifications. It is likely that onsite soil will not meet manufacturer specifications and imported materials will be necessary.

7.2 Box Culverts

7.2.1 Alternatively, planned crossing may consist of pre-cast or cast-in-place box culverts. Recommendations contained in this report are intended to be used to aid in selection of box culvert type and design of associated head walls and wing walls.

7.2.2 Culvert excavation bottoms should be cleaned of loose and saturated materials to expose firm, undisturbed native soil/rock as evaluated by our representative. Where competent soils are not encountered, overexcavation and replacement with engineered fill or stabilization may be required. Specific mitigation recommendations can be provided in the field at the time of construction.

7.2.3 Foundations for box culvert head walls and wing walls may consist of reinforced concrete spread footings comprised of strip footings at least 2 feet wide. Embedment depth of footings should be at least 2 feet into firm, undisturbed soil/rock or 2 feet below the drainage channel invert elevation, whichever is shallower. Our representative should evaluate footing embedment depth in the field on a case-by-case basis during construction. If suitable soils are not encountered within the recommended minimum embedment depth, footings may need to be deepened.

7.2.4 Foundations meeting the above recommendations may be designed using an allowable bearing capacity of 3,000 psf for dead plus live loads. This value may be increased by one-third when considering transient loads due to wind, seismic forces or temporary vehicle loads. The weight of foundation concrete below grade may be disregarded in sizing computations. Footing reinforcement should be designed by the project structural engineer.

7.2.5 Passive pressure used to resist lateral movement of footings may be assumed to be equivalent fluid weight of 350 pcf. The coefficient of friction to resist sliding is 0.35 for concrete against soil. Combined passive resistance and friction may be utilized for design provided frictional resistance is reduced by 50%.

7.2.6 Abutment wall backfill should consist of free-draining crushed rock or sand with less than 10% passing the No. 200 sieve. Geocon should sample, test, and approve proposed backfill materials prior to construction. Provided free-draining crushed rock is used for backfill, abutment walls should be designed using an active lateral earth pressure equal to an equivalent fluid pressure of 40 pcf. This pressure assumes the walls are unrestrained, have a level backfill, no surcharge, and a drained backfill condition. Therefore, wall back-drains or weepholes should be provided. Wall back-drains may consist of a continuous permeable backfill system. This system requires considerable quantities of permeable material but requires less compactive effort for wall backfilling. Alternatively, the use of pre-manufactured drainage composite may be utilized if approved by the project engineer and Geocon.

8.0 FURTHER GEOTECHNICAL SERVICES

8.1 Plan and Specification Review

8.1.1 Geocon should review the foundation and grading plans prior to final design submittal to assess whether our recommendations have been properly implemented and evaluate if additional analysis and/or recommendations are required.

8.2 Testing and Observation Services

8.2.1 The recommendations provided in this report are based on the assumption that we will continue as Geotechnical Engineer of Record throughout the construction phase. It is important to maintain continuity of geotechnical interpretation and confirm that field conditions encountered are similar to those anticipated during design. If we are not retained for these services, we cannot assume any responsibility for other's interpretation of our recommendations.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

The recommendations contained in this report are preliminary until verified during construction by representatives of our firm. Changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. Additionally, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated partially or wholly by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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

10.0 REFERENCES

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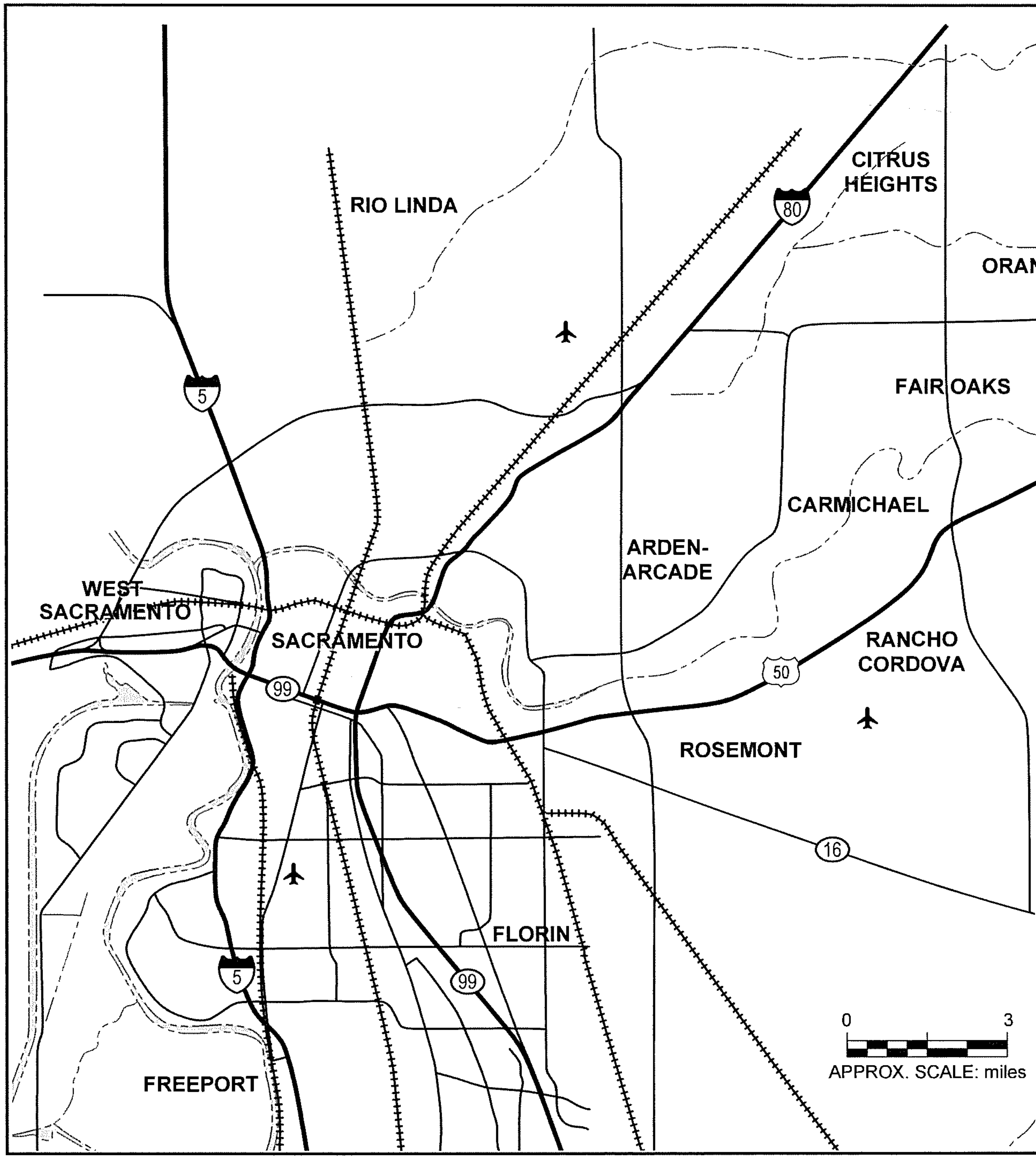

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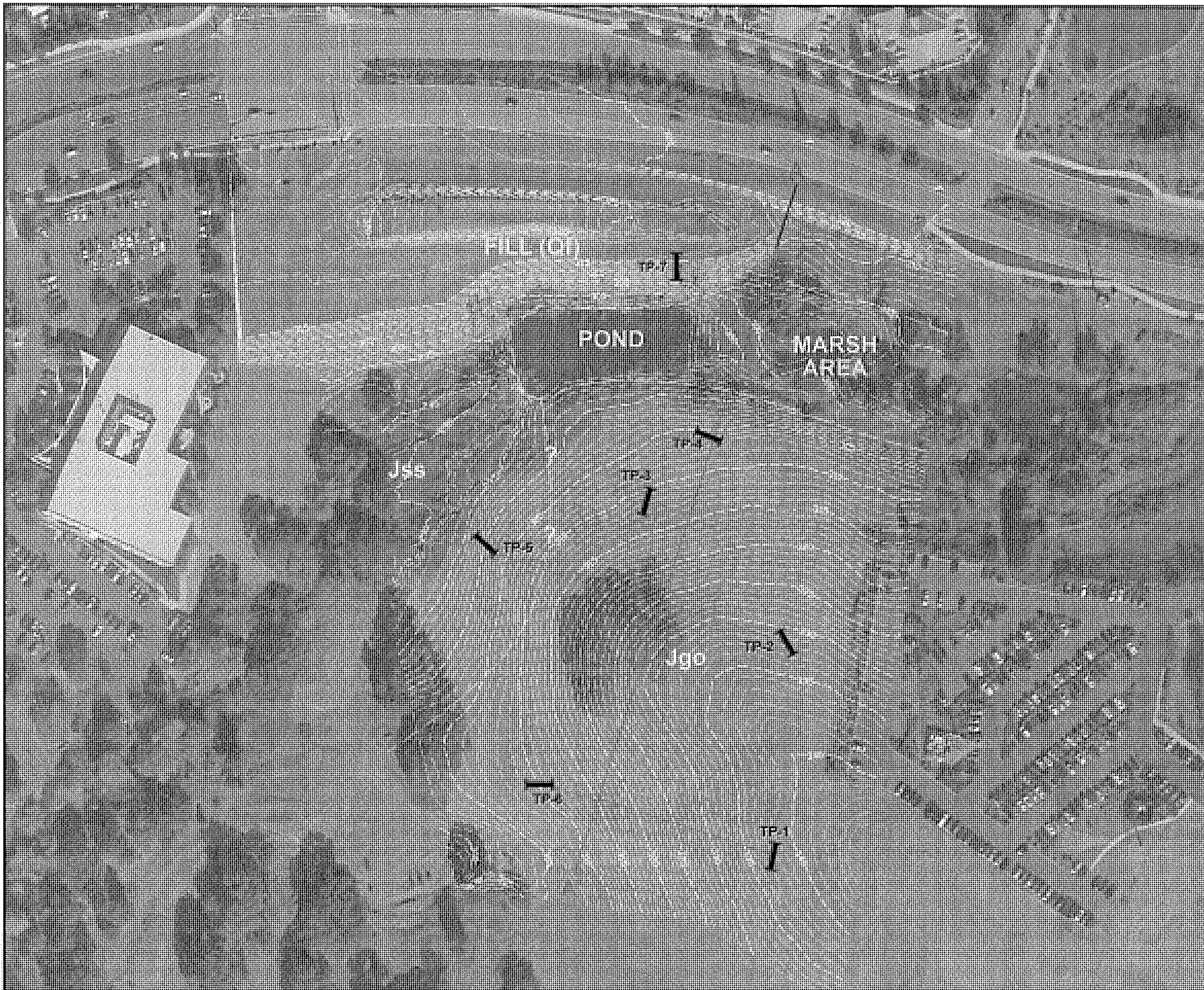
Folsom Senior Living Facility

Iron Point Road,
Folsom, California

VICINITY MAP

GEOCON Proj. No. S1367-05-01	July 2017	Figure 1

SITE0717.DWG



LEGEND:

TP-1 Approximate Test Pit Location

Approximate Geologic Contact

Salt Springs Slate

Gopher Ridge Volcanics

Fill

Note:
Cross-section A-A' is shown on Figure 5.

N

0 60 120

SCALE: feet

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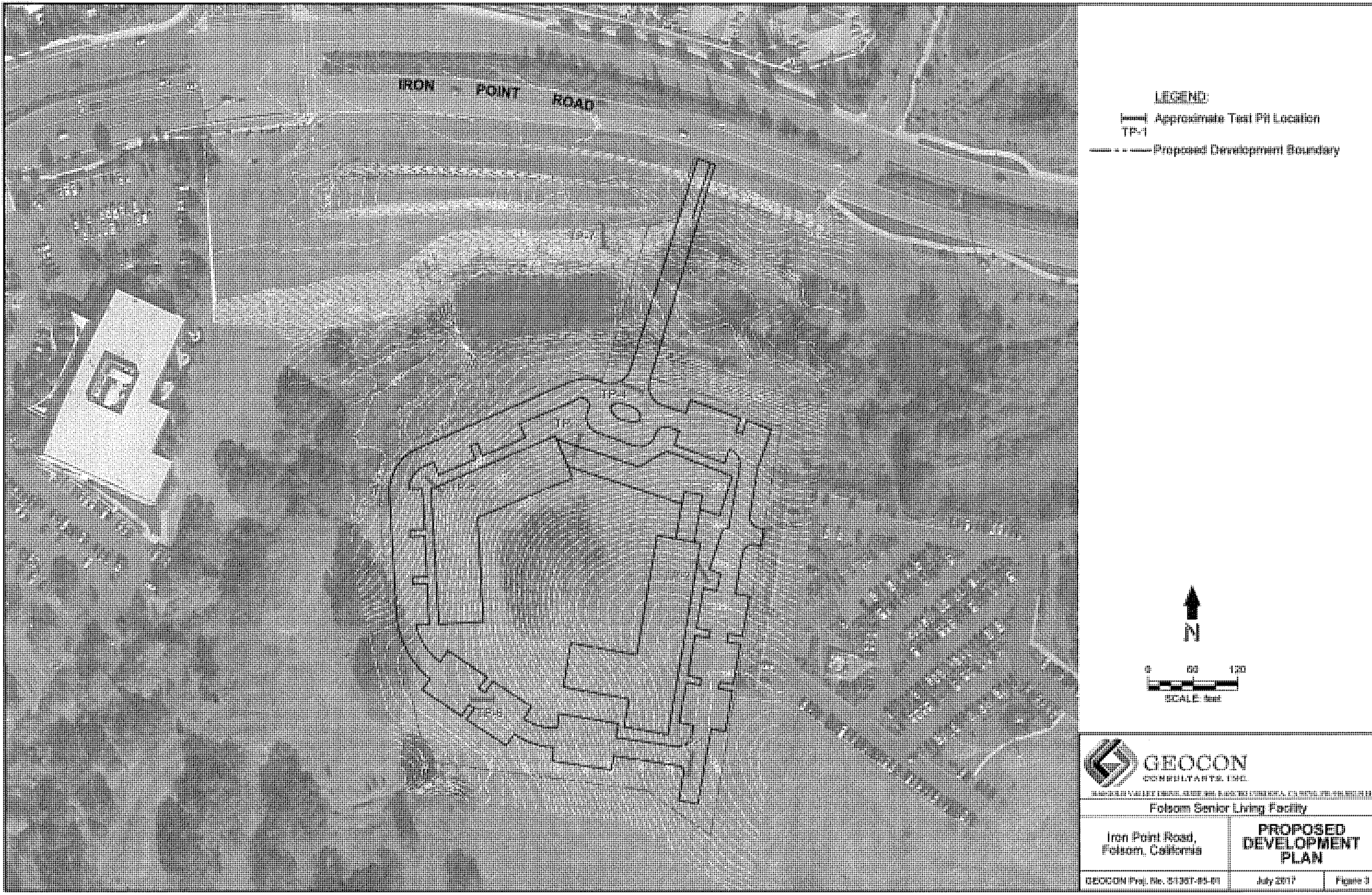
Folsom Senior Living Facility

Iron Point Road, Folsom, California	SITE PLAN/ GEOLOGIC MAP
GEOCON Proj. No. [illegible]	July 2017 · Figure 2

SITE0717.DWG



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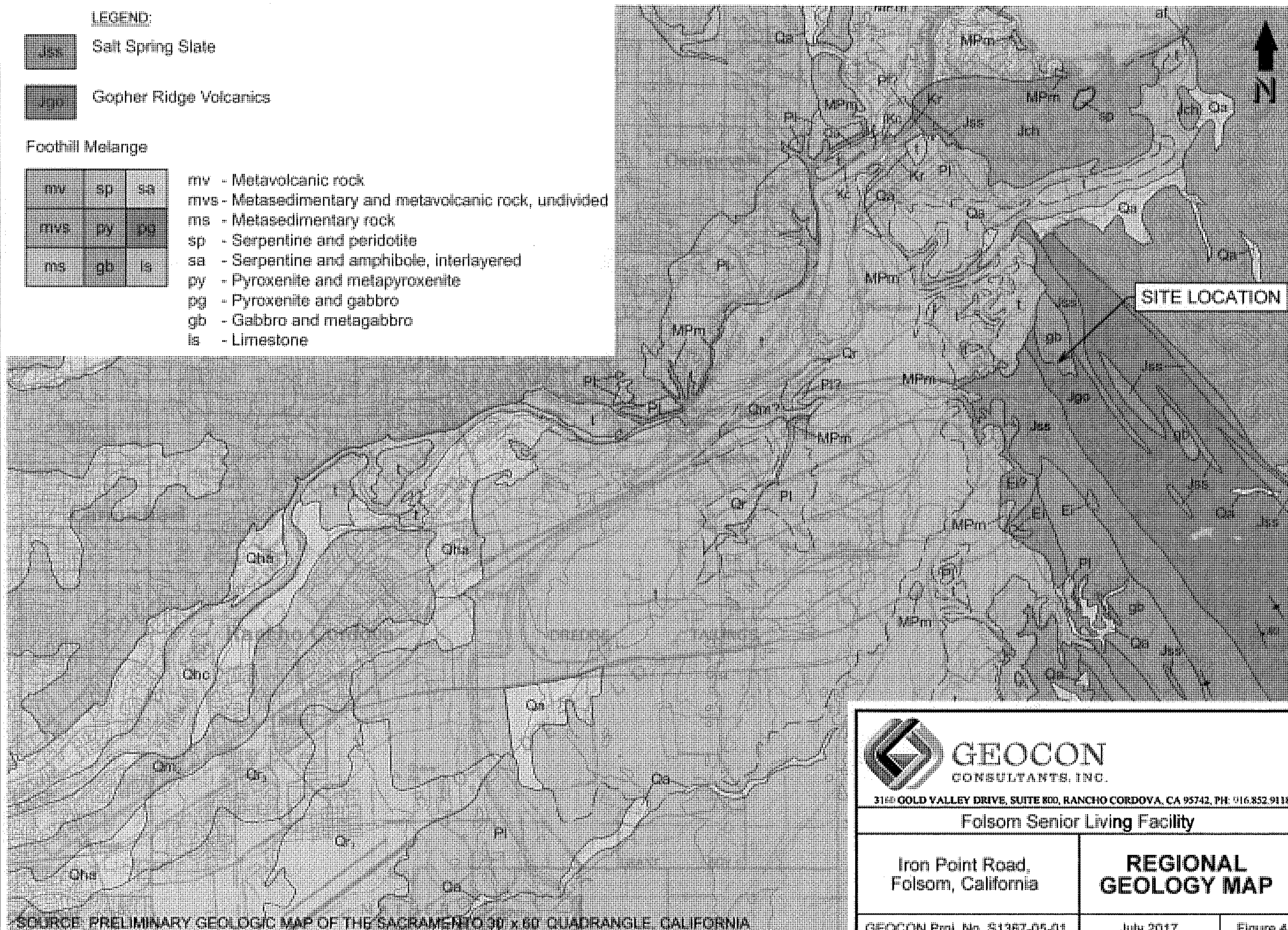
Iron Point Road, Folsom, California

PROPOSED DEVELOPMENT PLAN

GEOCON Proj. No. S1357-05-01	July 2017	Figure 3

SITE0717.DWG



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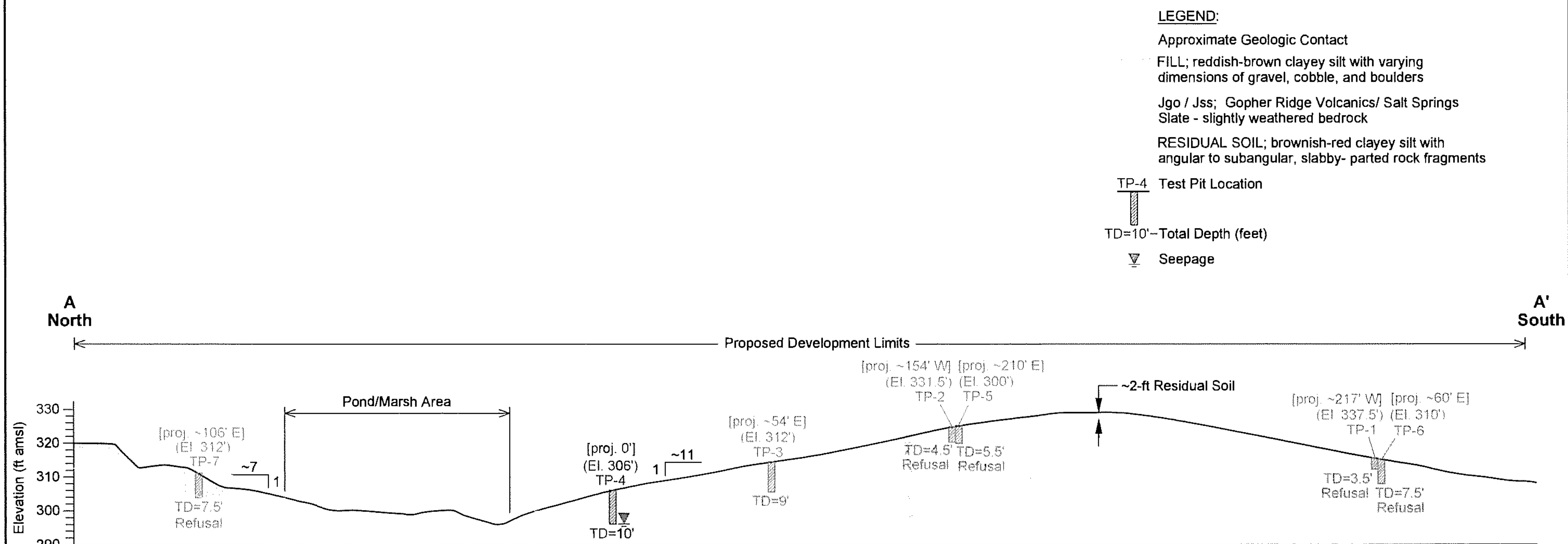
REGIONAL GEOLOGY MAP

GEOCON Proj. No. S1367-05-01 | July 2017 | Figure 4

SITE0717.DWG



Note:
Projected distances are onto the cross-section line.

CROSS-SECTION A-A'
(Vertical Exaggeration: 2X)

0 30 60
SCALE: feet

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CROSS-SECTION A-A'

GEOCON Proj. No. S1367-05-01 | July 2017 | Figure 5

SITE0717.DWG

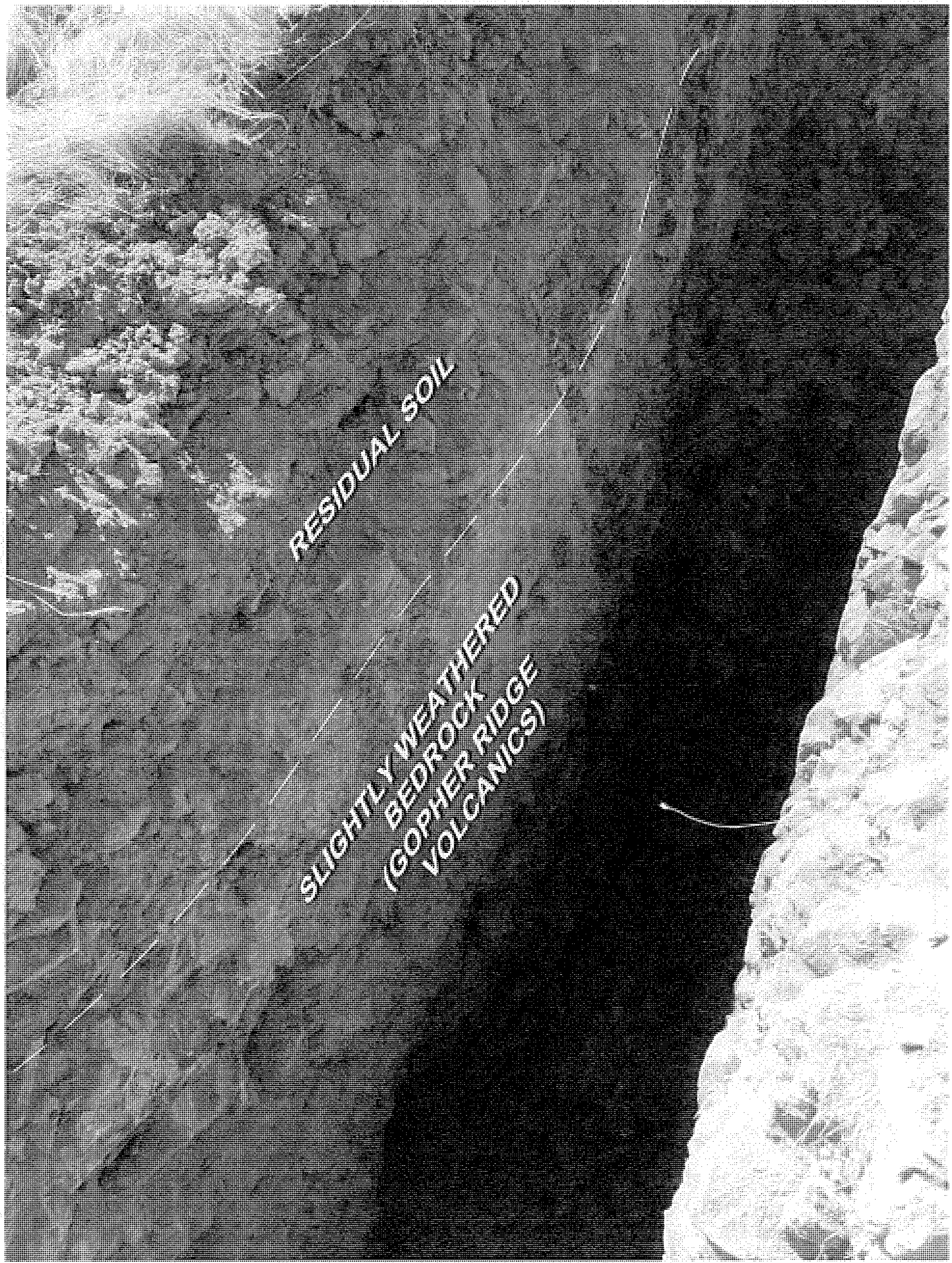

Photo No.1 Typical transition between residual soil and bedrock - Test Pit TP-3

GEOCON CONSULTANTS, INC. 3160 GOLD VALLEY DRIVE, SUITE 800, RANCHO CORDOVA, CA 95742, PH 916.852.9118	Folsom Senior Living Facility	
	Iron Point Road, Folsom, California	**PHOTO No. 1**
	GEOCON Proj. No. S1367-05-01	July 2017

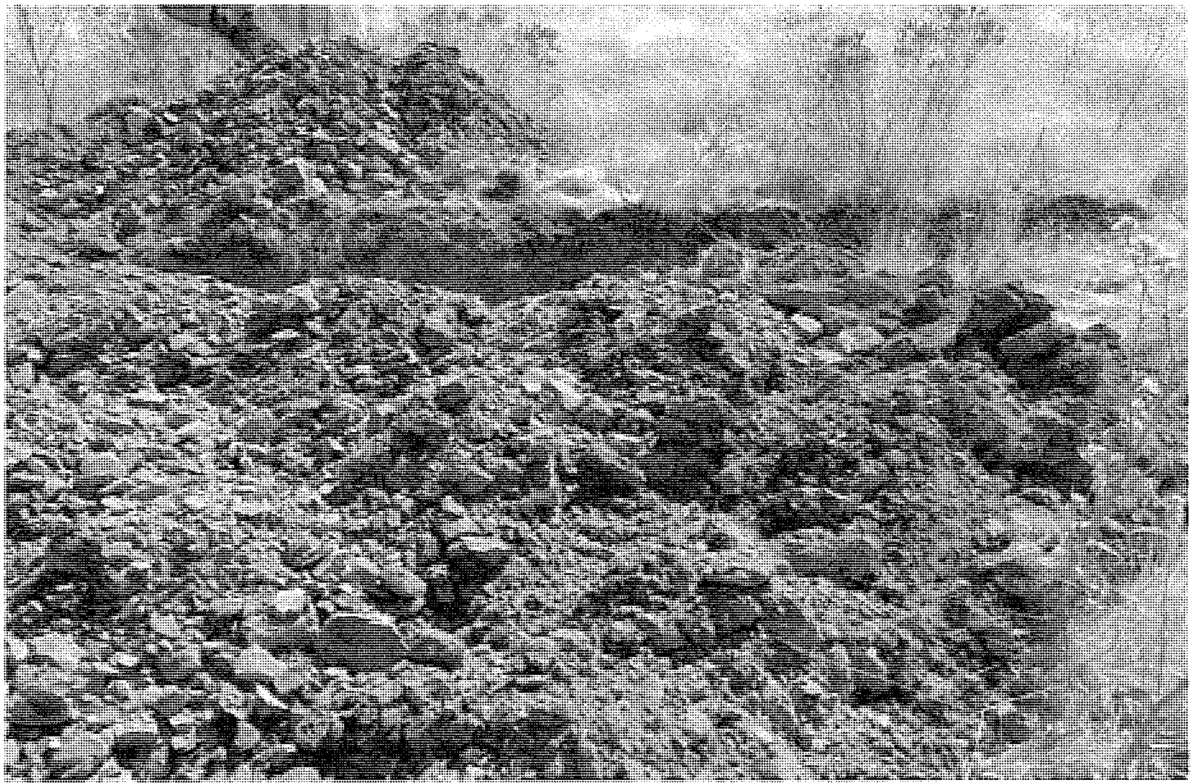

Photo No.2 Typical excavated material - Test Pit TP-3

Photo No.3 Oversized material - dimensions varied across site - Test Pit TP-7

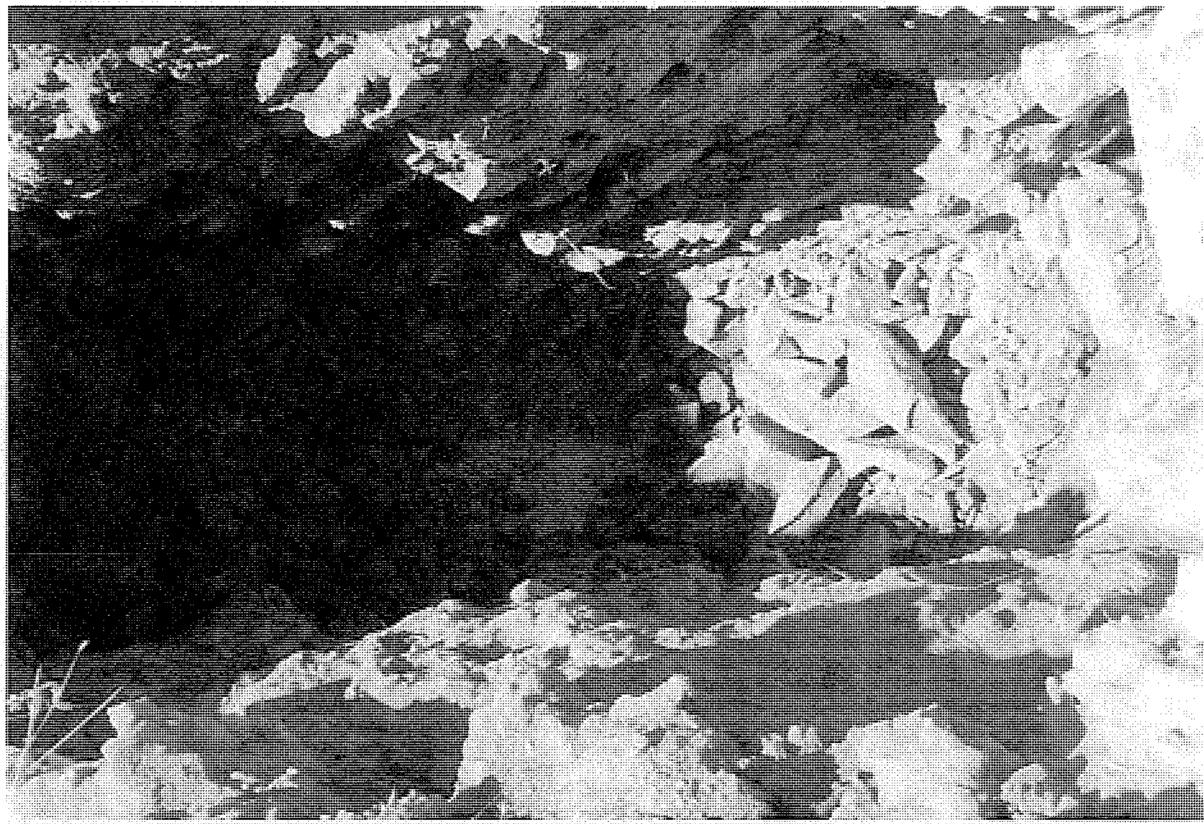

Photo No.4 Seepage in Test Pit TP-4 near pond and marsh area

Photo No.5 Existing pond on northern portion of site (June 2017)

Photo No.6 Excavated salt springs slate - Test Pit TP-5

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GEOCON Proj. No. S1367-05-01	July 2017

APPENDIX A

APPENDIX A

FIELD EXPLORATION

Our geotechnical field exploration program was performed on June 28, 2017, and consisted of excavating seven exploratory test pits (TP-1 through TP-7) at the approximate locations shown on the Site Plan/Geologic Map, Figure 2, and Proposed Development Plan, Figure 3.

Exploratory test pits were performed using a John Deere 310L backhoe equipped with an 18-inch-wide bucket with rock teeth. Bulk samples were obtained from the test pits. Upon completion, the test pits were backfilled with the excavated material and tamped down with the backhoe bucket.

Subsurface conditions encountered in the exploratory test pits were visually examined, classified and logged in general accordance with the ASTM Practice for Description and Identification of Soils (Visual-Manual Procedure D2488-90). This system uses the Unified Soil Classification System (USCS) for soil designations. The logs depict soil and geologic conditions encountered and depths at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, excavation characteristics, and other factors. The transition between materials may be abrupt or gradual. Where applicable, the field logs were revised based on subsequent laboratory testing. A Key to Logs is presented as Figure A1 and logs of test pits (TP-1 through TP-7) are presented as Figures A2 through A8.

UNIFIED SOIL CLASSIFICATION

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

BORING/TRENCH LOG LEGEND

- No Recovery
- Shelby Tube Sample 3" O.D.
- Bulk Sample
- SPT Sample 2" O.D., 1.4" I.D.
- Modified California Sample 3" O.D., 2.4" I.D.
- Groundwater Level (At Completion)
- Groundwater Level (Seepage)

PENETRATION RESISTANCE

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

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

MOISTURE DESCRIPTIONS

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

QUANTITY DESCRIPTIONS

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

GRAVEL/COBBLE/BOULDER DESCRIPTIONS

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

LABORATORY TEST KEY

CP – COMPACTION CURVE (ASTM D1557)
CR – CORROSION ANALYSIS (CTM 422, 643, 417)
DS – DIRECT SHEAR (ASTM D3080)
EI – EXPANSION INDEX (ASTM D4829)
GSA – GRAIN SIZE ANALYSIS (ASTM D422)
MC – MOISTURE CONTENT (ASTM D2216)
PI – PLASTICITY INDEX (ASTM D4318)
R – R-VALUE (CTM 301)
SE – SAND EQUIVALENT (CTM 217)
TXCU – CONSOLIDATED UNDRAINED TRIAXIAL (ASTM D4767)
TXUU – UNCONSOLIDATED UNDRAINED TRIAXIAL (ASTM D2850)
UC – UNCONFINED COMPRESSIVE STRENGTH (ASTM D2166)

BEDDING SPACING DESCRIPTIONS

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

STRUCTURE DESCRIPTIONS

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

CEMENTATION/INDURATION DESCRIPTIONS

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

IGNEOUS/METAMORPHIC ROCK STRENGTH DESCRIPTIONS

FIELD TEST	DESCRIPTION
MATERIAL CRUMBLES WITH BARE HAND	WEAK
MATERIAL CRUMBLES UNDER BLOWS FROM GEOLOGY HAMMER	MODERATELY WEAK
1/4-INCH INDENTATIONS WITH SHARP END FROM GEOLOGY HAMMER	MODERATELY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **ONE** BLOW FROM GEOLOGY HAMMER	STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **COUPLE** BLOWS FROM GEOLOGY HAMMER	VERY STRONG
HAND-HELD SPECIMEN CAN BE BROKEN WITH **MANY** BLOWS FROM GEOLOGY HAMMER	EXTREMELY STRONG

IGNEOUS/METAMORPHIC ROCK WEATHERING DESCRIPTIONS

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

IGNEOUS/METAMORPHIC ROCK JOINT/FRACTURE DESCRIPTIONS

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

GEOCON
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KEY TO LOGS

Figure A1

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP1** ELEV. (MSL.) ______ DATE COMPLETED 6/28/17 ENG./GEO. Victor Guardado DRILLER Bill Thompson EQUIPMENT 310L-Backhoe HAMMER TYPE 18" Bucket w/ rock teeth	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
					MATERIAL DESCRIPTION				
0 1 2	TP-1 Bulk			CL-ML	**RESIDUAL SOIL** Damp, reddish brown, Clayey SILT with some sand and rock fragments - layer of lean to fat clay				
3				GC	**GOPHER RIDGE VOLCANICS** Moderately weathered Metavolcanic Rock: excavates as hard, tannish brown, Clayey GRAVEL and fractured rock with some clay and silt infilling				
					REFUSAL AT 3.5 FEET NO GROUNDWATER ENCOUNTERED BACKFILLED WITH EXCAVATED MATERIAL				

Figure A2, Log of Test Pit, page 1 of 1

IN PROGRESS S1367-05-01 IRON POINT SENIOR LIVING.GPJ 07/13/17

GEOCON

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ☒ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

TEST PIT TP2

ELEV. (MSL.) ____________ DATE COMPLETED **6/28/17**

ENG./GEO. **Victor Guardado** DRILLER **Bill Thompson**

EQUIPMENT **310L-Backhoe** HAMMER TYPE **18" Bucket w/ rock teeth**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	TP-2 Bulk			CL-ML	**RESIDUAL SOIL** Damp, reddish brown, Clayey SILT with some sand and rock fragments				
1									
					- becomes tan to light brown				
2				GC	**GOPHER RIDGE VOLCANICS** Slightly to moderately weathered Metavolcanic Rock: excavates as hard, tannish grayish brown, Clayey GRAVEL and fractured rock with some clay and silt infilling				
3									
4									

EXCAVATION TERMINATED AT 4.5 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH EXCAVATED MATERIAL

Figure A3, Log of Test Pit, page 1 of 1

GEOCON

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

TEST PIT TP3

ELEV. (MSL.) ____________ DATE COMPLETED **6/28/17**

ENG./GEO. **Victor Guardado** DRILLER **Bill Thompson**

EQUIPMENT **310L-Backhoe** HAMMER TYPE **18" Bucket w/ rock teeth**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	TP-3 Bulk			CL-ML	**RESIDUAL SOIL** Damp to moist, brownish red, clayey SILT with some sand and angular to sub-angular rock fragments				
1					- becomes light grayish brown				
2				GC	**GOPHER RIDGE VOLCANICS** Slightly weathered Metavolcanic Rock: excavates as hard, light grayish brown, Clayey GRAVEL and fractured rock with some clay and silt infilling				
3					- becomes moist				
4									
5									
6									
7									
8									
9					TEST PIT TERMINATED AT 9 FEET NO GROUNDWATER ENCOUNTERED BACKFILLED WITH EXCAVATED MATERIAL				

Figure A4, Log of Test Pit, page 1 of 1

GEOCON

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

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

TEST PIT TP4

ELEV. (MSL.) ________ DATE COMPLETED **6/28/17**

ENG./GEO. **Victor Guardado** DRILLER **Bill Thompson**

EQUIPMENT **310L-Backhoe** HAMMER TYPE **18" Bucket w/ rock teeth**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	TP-4 Bulk			CL-ML	**RESIDUAL SOIL** Hard, damp, brownish red, Clayey SILT with small to mediium rock fragments - PP = 4.5				
1									
2					- increasing rock size				
3				GC	**GOPHER RIDGE VOLCANICS** Slightly weathered Metavolcanic Rock: excavates as hard, light grayish brown, Clayey GRAVEL and fractured rock with some clay and silt infilling				
4									
5									
6									
7					- large chunk of quartz encountered				
8									
9					- becomes moist to wet, increasing clayey content				
10			▽		- seepage at 10 feet				
					TEST PIT TERMINATED AT 10 FEET SEEPAGE AT 10 FEET BACKFILLED WITH EXCAVATED MATERIAL				

Figure A5, Log of Test Pit, page 1 of 1

IN PROGRESS S1367-05-01 IRON POINT SENIOR LIVING.GPJ 07/13/17

GEOCON

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

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

TEST PIT TP5

ELEV. (MSL.) ______ DATE COMPLETED **6/28/17**

ENG./GEO. **Victor Guardado** DRILLER **Bill Thompson**

EQUIPMENT **310L-Backhoe** HAMMER TYPE **18" Bucket w/ rock teeth**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	TP-5 Bulk			CL-ML	**RESIDUAL SOIL** Damp, brownish red, clayey SILT with some sand and few angular to sub-angular rock fragments, roots				
1									
					- becomes grayish brown				
2				GC	**SALT SPRINGS SLATE** Slightly weathered Metavolcanic Rock: excavates as hard, grayish brown, Clayey GRAVEL and moderate to abbundant slabby-parted slate with some clay and silt infilling				
3									
4									
5									

REFUSAL AT 5.5 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH EXCAVATED MATERIAL

GEOCON

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

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

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP6** ELEV. (MSL.) ___ DATE COMPLETED 6/28/17 ENG./GEO. Victor Guardado DRILLER Bill Thompson EQUIPMENT 310L-Backhoe HAMMER TYPE 18" Bucket w/ rock teeth	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
					MATERIAL DESCRIPTION				
0 – 2	TP-6 Bulk			CL-ML	**RESIDUAL SOIL** Damp, brownish red, Clayey SILT with some sand and cobbles with max dimension of 22", roots - becomes grayish light brown				
2 – 7.5				GC	**GOPHER RIDGE VOLCANICS** Slightly to moderately weathered Metavolcanic Rock: excavates as hard, grayish light brown, Clayey GRAVEL and fractured rock with some clay and silt infilling - increasing amount of rock fragments				
					REFUSAL AT 7.5 FEET NO GROUNDWATER ENCOUNTERED BACKFILLED WITH EXCAVATED MATERIAL				

Figure A7, Log of Test Pit, page 1 of 1

IN PROGRESS S1367-05-01 IRON POINT SENIOR LIVING.GPJ 07/13/17

GEOCON

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

PROJECT NO. **S1367-05-01** PROJECT NAME **Iron Point Senior Living**

TEST PIT TP7

ELEV. (MSL.) ______ DATE COMPLETED **6/28/17**

ENG./GEO. **Victor Guardado** DRILLER **Bill Thompson**

EQUIPMENT **310L-Backhoe** HAMMER TYPE **18" Bucket w/ rock teeth**

DEPTH IN FEET	SAMPLE INTERVAL & RECOVERY	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	ADDITIONAL TESTS
0	TP-7 Bulk			CL-ML	**FILL** Damp, reddish brown, clayey SILT with varying dimension of gravel, cobbles, and boulders up to 30" - excavation takes place of slope face				
1									
2									
3					- slate fragments - some blue-green clayey soil chunks				
4									
5									
6									
7				GC	- becomes hard				

REFUSAL AT 7.5 FEET
NO GROUNDWATER ENCOUNTERED
BACKFILLED WITH EXCAVATED MATERIAL

Figure A8, Log of Test Pit, page 1 of 1

IN PROGRESS S1367-05-01 IRON POINT SENIOR LIVING.GPJ 07/13/17

GEOCON

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

APPENDIX B

APPENDIX B

LABORATORY TESTING PROGRAM

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their corrosion potential, expansion potential, resistance value (R-value), and moisture-density relationship. Laboratory test results are presented herein.

TABLE B1
SUMMARY OF CORROSION PARAMETERS
CALIFORNIA TEST METHODS 643, 417, AND 422

Sample No.	Sample Depth (ft.)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
TP-2 Bulk	0 – 3	5.69	3,480	0.9	6.6

*Caltrans considers a site corrosive to foundation elements if one or more of the following conditions exist for the representative soil samples at the site:

- The pH is equal to or less than 5.5.
- The resistivity is equal to or less than 1,000 ohm-cm.
- Chloride concentration is equal to or greater than 500 parts per million (ppm).
- Sulfate concentration is equal to or greater than 2,000 ppm.

According to the 2016 California Building Code Section 1904.1 which refers to the durability requirements of American Concrete Institute (ACI) 318 (Chapter 4), Type II cement may be used where soluble sulfate levels in soil are below 2,000 ppm.

TABLE B2
EXPANSION INDEX TEST RESULTS
ASTM D4829

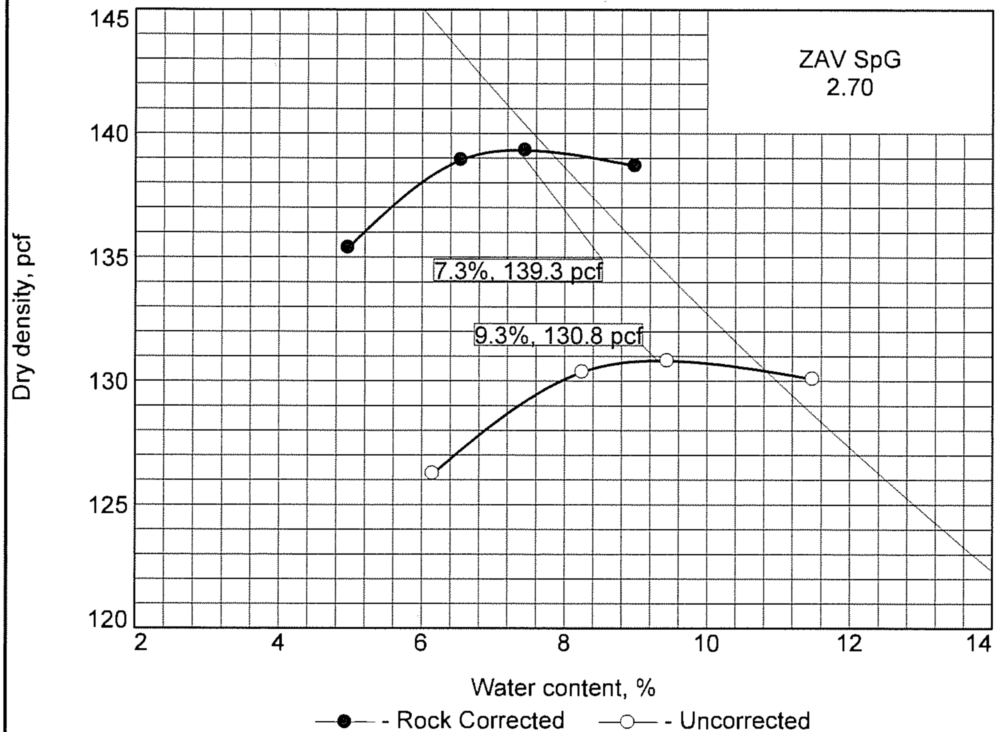
Sample Number	Depth (feet)	Moisture Content (%)		Expansion Index	Classification*
		Before Test	After Test		
TP-3,5,6 Bulk	0-3	9.9	20.7	28	Low

**Expansion Potential Classification per ASTM D4829.*

TABLE B3
SUMMARY OF R-VALUE TEST RESULTS

Sample ID	Depth (feet)	Sample Description	R-Value
TP-1,2 Bulk	0-3	Reddish brown, clayey silt	20

COMPACTION TEST REPORT



Curve No.
1

Test Specification:
ASTM 1557 Method A 2017
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Preparation Method
Hammer Wt. 10.00
Hammer Drop 18
Number of Layers 5
Blows per Layer 25
Mold Size 0.03327 cu. ft.
Test Performed on Material Passing #4 **Sieve**

NM **LL** **PI**
Sp.G. (ASTM D 854) 2.7
%>#4 24.5 **%<No.200**
USCS **AASHTO**
Date Sampled
Date Tested 7/5/17
Tested By JA

TESTING DATA

	1	2	3	4	5	6
WM + WS	4068.0	4175.0	4206.0	4234.0		
WM	2045.4	2045.4	2045.4	2045.4		
WW + T #1	2176.0	2289.0	2648.0	2476.0		
WD + T #1	2058.8	2127.2	2462.0	2250.9		
TARE #1	155.8	165.7	492.4	290.8		
WW + T #2						
WD + T #2						
TARE #2						
MOISTURE	5.0	6.5	7.4	9.0		
DRY DENSITY	135.4	138.9	139.3	138.7		

ROCK CORRECTED TEST RESULTS	UNCORRECTED	Material Description
Maximum dry density = 139.3 pcf	130.8 pcf	Red brown, silty lean clay with gravel
Optimum moisture = 7.3 %	9.3 %	

Remarks:

Project No. S1367-05-01 **Client:** The Wolff Company
Project: Folsom Senior Living Facility - Iron Point Road

○ **Sample Number:** TP-1,2 Bulk

Checked by: Victor Guardado
Title: Staff Engineer

GEOCON CONSULTANTS, INC.

Figure B1