Appendix F: Geological Supporting Information

THIS PAGE INTENTIONALLY LEFT BLANK

F.1 - Soil Corrosivity Evaluation

THIS PAGE INTENTIONALLY LEFT BLANK

Soil Corrosivity Evaluation
&
Recommendations for Corrosion Control

Underground Water Piping Systems
and
Concrete Foundations
Elnoka Village, Santa Rosa, CA

Reese & Associates

Santa Rosa, CA

December 9, 2016

Prepared By

jdh corrosion consultants, inc.

1100 Willow Pass Court, Concord, CA 94520
Tel. No. 925.927.6630 Fax No. 925.927.6634

December 9, 2016

Reese & Associates
134 Lystra Court, Suite C
Santa Rosa, CA 95403

Attention: **Mr. Jeffrey K. Reese**
President

Subject: **Soil Corrosivity Evaluation & Recommendations for Corrosion Control Underground Water Piping Systems and Concrete Foundations**
Elnoka Village
Santa Rosa, CA

Dear Mr. Reese,

Pursuant to your request, **JDH Corrosion Consultants, Inc**. has conducted a site corrosivity evaluation for the above referenced project site and we have provided herein recommendations for long-term corrosion control for the proposed materials of construction for the underground water lines and concrete foundations at this site.
.

Purpose

The purpose for this evaluation is to determine the corrosion potential, resulting from the soils along the project site and to provide recommendations for long-term corrosion control for the buried metallic utilities and concrete foundations.

Background

This project involves the construction of up to 778 units of housing for seniors and employees in a series of one-, two- and three-story buildings in a gated community setting. The structures are assumed to be slab-on-grade and there will be buried utilities associated with this development.

Soil Testing and Analysis

Soil Testing Results

Fourteen (14) soil samples were collected from the site by **Reese Associates** and they were transported to a state certified testing laboratory, **CERCO Analytical, Inc.** (certificate no. 2153) located in Concord, CA for chemical analysis. Each sample was analyzed for pH, chlorides, resistivity (@ 100% saturation), sulfates and Redox potential using ASTM test methods as detailed in the table below. The preparation of the soil samples for chemical analysis was in accordance with the applicable specifications.

Soil Analysis Test Methods

Chemical Analysis	ASTM Method
Chlorides	D4327
pH	D4972
Resistivity	G57
Sulfate	D4327
Redox Potential	D1498

The results of the chemical analysis are provided in the CERCO Analytical, Inc. reports dated November 4, 2016. The results are summarized as follows:

CERCO Analytical, Inc.
Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	N.D. – 17 (mg/kg)	Non-corrosive*
pH	6.03 – 7.65	Mildly Corrosive to Non-corrosive*
Resistivity (100% saturation)	1,100 – 4,900 ohms-cm	Corrosive to Moderately Corrosive*
Sulfate	N.D. (mg/kg)	Non-corrosive**
Redox Potential	410 - 480 mV	Non-corrosive*

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Chemical Testing Analysis

The chemical analysis provided by **CERCO Analytical, Inc.** indicates that the soils are generally classified as “corrosive to moderately corrosive”. The chloride levels indicate “non-corrosive” conditions to steel and ductile iron and the sulfate levels indicate “non-corrosive” conditions for concrete structures placed into these soils with regard to sulfate attack. The pH of the soils is alkaline which classifies them as “non-corrosive” to buried steel and concrete structures.

In-Situ Soil Resistivity Measurements

The in-situ resistivity of the soil was measured at ten (10) locations along the project site by **JDH Corrosion Consultants, Inc.** field personnel at the locations shown on the attached map. Resistance measurements were conducted with probe spacing of 2.5, 5, 7.5, 10, and 15-feet at each location. For analysis purposes we have calculated the resistivity of soil layers 0-2.5, 2.5-5, 5-7.5, 7.5-10, and 10-15' using the Barnes Method as follows:

$$\rho_{b-a} = KR_{(b-a)}$$

Where;

ρ_{b-a}	=	soil resistivity of layer depth b-a (ohm-cm)
a	=	soil depth to top layer (ft)
b	=	soil depth to bottom layer (ft)
R_a	=	soil resistance read at depth a (ohms)
R_b	=	soil resistance read at depth b (ohms)
R_{b-a}	=	resistance of soil layer from a to b (ft)
K	=	layer constant = $60.96\pi(b-a)$ (cm)

and $$\frac{1}{R_{b-a}} = \frac{1}{R_a} - \frac{1}{R_b}$$

The visual diagrams below describe the Wenner 4-pin testing configuration.

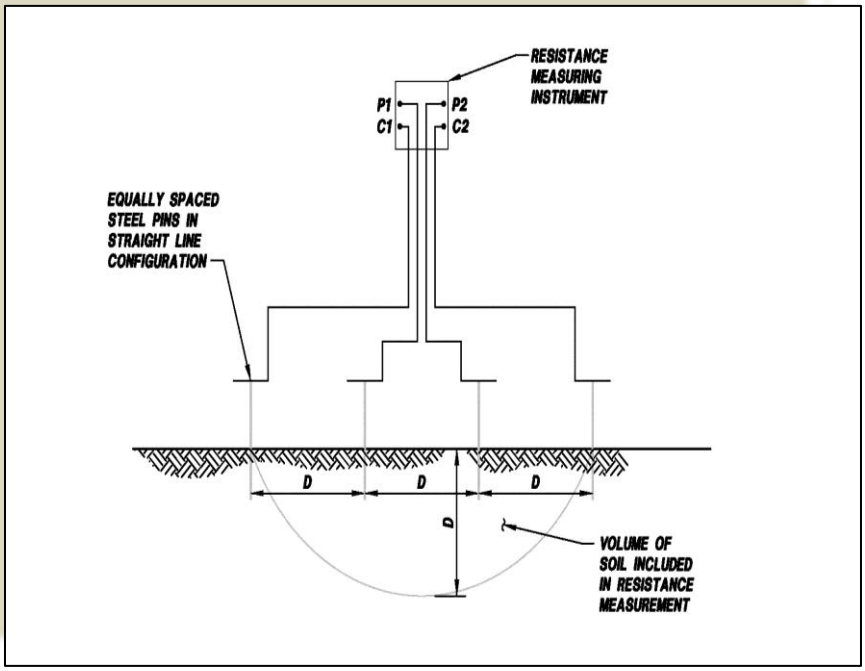


Fig 1: Wenner 4-Pin Resistivity Schematic No.1

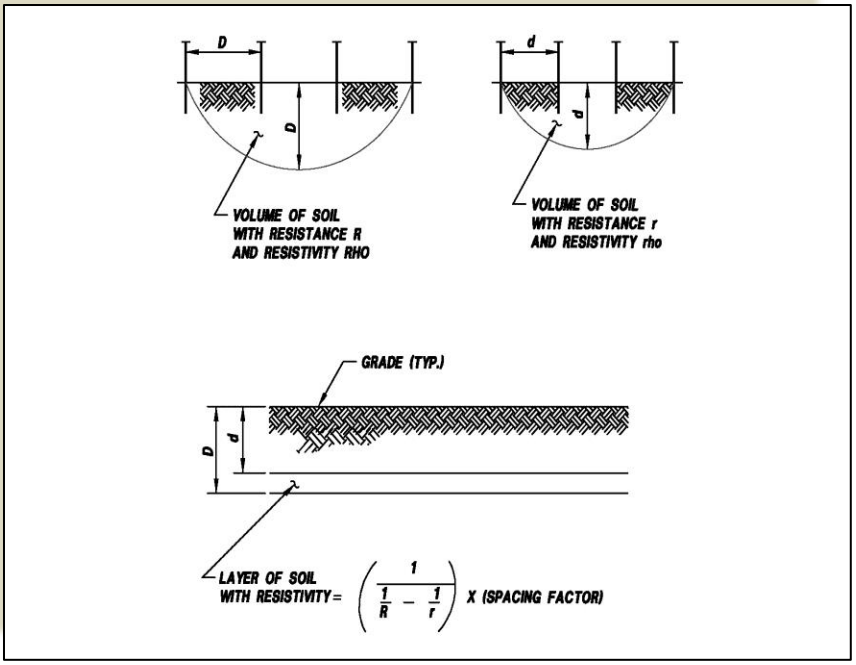


Fig 2: Illustration of Barnes Layer Calculations

In-Situ Soil Resistivity Analysis

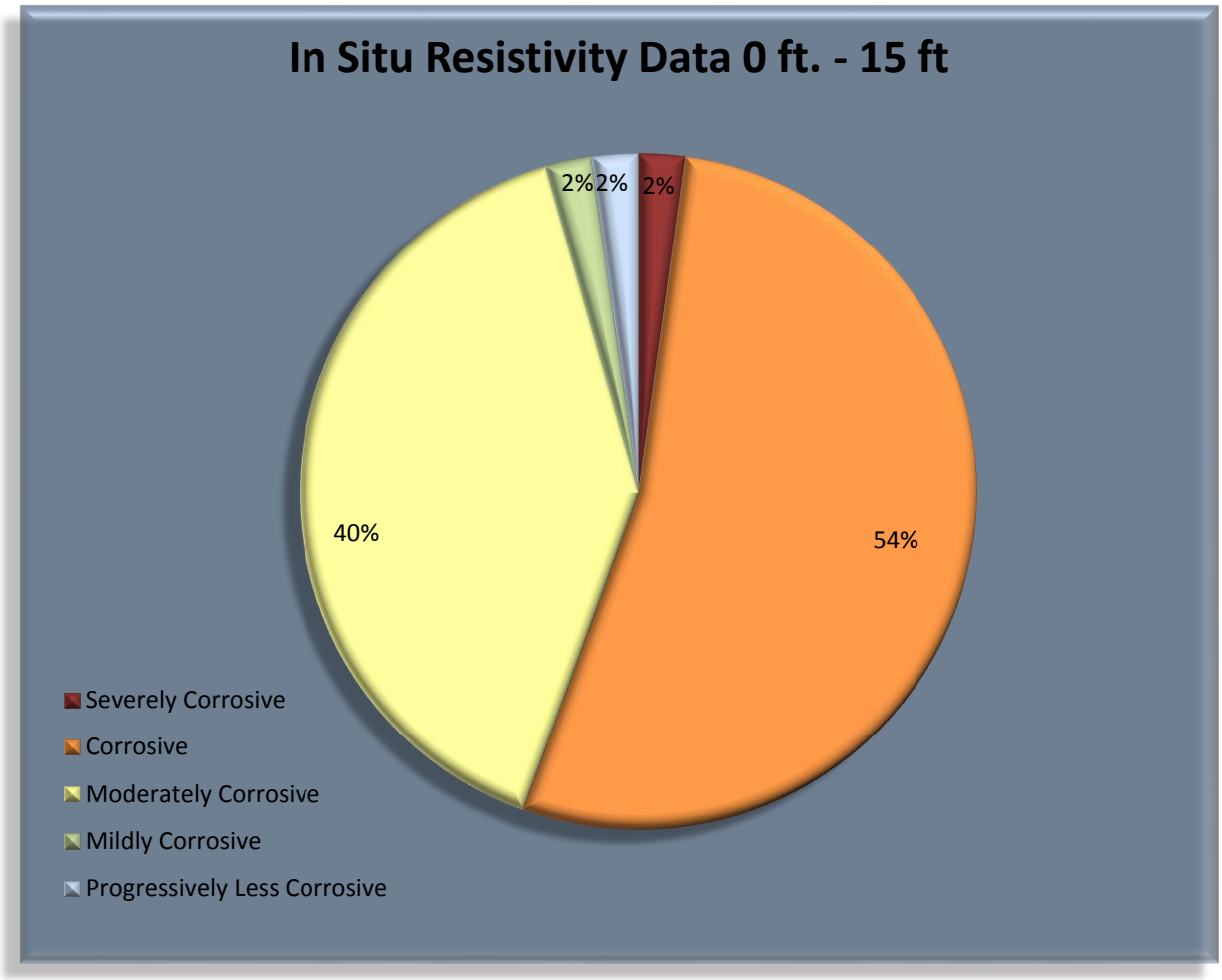
Corrosion of a metal is an electro-chemical process and is accompanied by the flow of electric current. Resistivity is a measure of the ability of a soil to conduct an electric current and is, therefore, an important parameter in consideration of corrosion data. Soil resistivity is primarily dependent upon the chemical content and moisture content of the soil mass.

The greater the amount of chemical constituents present in the soil, the lower the resistivity will be. As moisture content increases, resistivity decreases until maximum solubility of dissolved chemicals is attained. Beyond this point, an increase in moisture content results in dilution of the chemical concentration and resistivity increases. The corrosion rate of steel in soil normally increases as resistivity decreases. Therefore, in any particular group of soils, maximum corrosion will generally occur in the lowest resistivity areas. The following classification of soil corrosivity, developed by William J. Ellis[1], is used for the analysis of the soil data for the project site.

Resistivity (Ohm-cm)	Corrosivity Classification
0 – 500	Very Corrosive
501 – 2,000	Corrosive
2,001 – 8,000	Moderately Corrosive
8,001 – 32,000	Mildly Corrosive
> 32,000	Progressively Less Corrosive

The above classifications are appropriate for the project site and the results are presented in the graphs below. In general, the soils are classified as “corrosive to mildly corrosive” with respect to corrosion of buried steel structures throughout the top 0 to 15 feet of the site.

The chart of the in-situ soil resistivity data for the soil layers 0 to 15 feet indicate that 2% of the soils are classified as “severely corrosive”, 54% of the soils are classified as “corrosive”, 40% of the soils are classified as “moderately corrosive”, 2% of the soils are classified as “mildly corrosive” and 2% of the soils are classified as “progressively less corrosive”.



Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be "corrosive" to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

Recommendations

Reinforced Concrete Slab Foundations

1. We recommend using a Type II concrete mix with a water-to-cement ratio as specified in the California Building Code (CBC) for soils containing less than 0.10% water soluble sulfate by weight. Adhering to the minimum depth of cover for the reinforcing steel in the foundations as specified in the CBC is recommended for the subject structures as well.

Ductile Iron Pipe (Pressure Piping such as Domestic Water and Fire)

1. Direct buried ductile iron pipe should be encased in 8-mil polyethylene as specified in AWWA specification C-105. Epoxy coatings are also an acceptable alternative type of coating system for the pipe and/or fittings such as valves.

2. All rubber gasket joints, fusion-bonded epoxy coated flanges and flexible couplings on ductile iron pipelines should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

3. Insulating flanges and/or couplings should be installed to electrically isolate the buried portion of pipeline from other metallic pipelines, reinforced concrete structures and above grade buildings or structures.

4. Test stations shall be installed on all ductile iron pipelines at a spacing of 800 to 1,000 feet. Bonding and test stations shall comply with NACE Standards.

5. A sacrificial type of cathodic protection utilizing ***magnesium*** anodes should be installed to protect the entire length of buried metallic pipeline. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

6. As an alternate, non-metallic piping may be used in lieu of ductile iron piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures. However, all metallic valves, fittings and appurtenances on non-metallic piping will require protection as specified below.

Ductile Iron Fittings & Metallic Valves (On Plastic Pressure Piping)

1. All direct buried ductile iron fittings installed on non-metallic piping shall be provided with a bituminous coating from the factory and encased in an 8-mil polyethylene bag in the field in accordance with AWWA Specification C-105. All bolts, restraining rods, etc. shall be coated with bitumastic prior to encasement in the polyethylene bag.

2. All metallic valves shall be coated from the factory (i.e. using powdered epoxy or equivalent type of coating system) and all bolts shall be coated with bitumastic in the field and the entire valve shall be encased in an 8-mil polyethylene bag in accordance with AWWA Specification C-105.

3. A sacrificial type of cathodic protection utilizing ***magnesium*** anodes should be installed to protect the valves and fittings. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the pipeline.

Cast Iron (Gravity Sewer and Storm Drain Lines)

1. No special corrosion considerations are required for the gravity sewer and storm drain lines.

Steel Pipelines (Natural Gas Pipelines & Risers)

1. A fusion-bonded epoxy coating system or a suitable tape coating should be applied to all buried steel pipelines in accordance with ANSI/AWWA C214-95, “AWWA Standard for Tape Coating Systems for the Exterior of Steel Water Pipelines.” Also, a tape coating per AWWA Standard C209-95 is recommended for special sections, connections and fittings.

2. Insulating flanges and/or couplings should be installed to electrically isolate the buried portions of steel pipelines from other metallic pipelines, reinforced concrete structures and above grade structures.

3. All rubber gasket joints, fusion epoxy coated flanges and flexible couplings should be bonded with insulated copper cable to insure electrical continuity of the pipeline and fittings.

4. A sacrificial type of cathodic protection using ***magnesium*** anodes should be installed to protect the buried portions of steel pipelines used for the natural gas piping systems. Cathodic protection should be designed in accordance with NACE Standard SP0169-13 and applicable local standards and included with the contract documents to permit installation along with the subject pipeline.

5. As an alternate, non-metallic piping may be used in lieu of steel piping as allowed by State and local codes. Non-metallic piping does not require the implementation of any special type of corrosion prevention measures.

Copper Water Pipelines (Service Lines)

1. All copper water laterals shall be provided with a polyethylene sleeve to effectively isolate the copper piping from the earth.

2. All copper water laterals shall be electrically isolated from metallic water mains via the use of insulating type corporation stops installed at the water main.

LIMITATIONS

The conclusions and recommendations contained in this report reflect the opinion of the author of this report and are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees either expressed or implied are provided.

We thank you for the opportunity to be of assistance on this important project. If you have any questions concerning this report or the recommendations provided herein, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Field Technician

Mohammed Ali

Mohammed Ali, P.E.
JDH Corrosion Consultants, Inc.
Principal

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED ALI
No. CR 1035
Exp. June 30, 2017
CORROSION
STATE OF CALIFORNIA

CC: File 162i8

REFERENCES

1. *Ellis, William J., Corrosion of Concrete Pipelines, Western States Corrosion Seminar, 1978*

2. *AWWA Manual of Water Supply Practices - M27, First Edition, External Corrosion - Introduction to Chemistry and Control (Denver, CO: 1987)*

3. *National Association of Corrosion Engineers, Standard Recommended Practice, SP 01-69-13, Control of External Corrosion on Underground or Submerged Pipeline*

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: JDH Corrosion Consultants, Inc.
Client's Project No.: 16218
Client's Project Name: Santa Rosa - El Noka
Date Sampled: Not Indicated
Date Received: 27-Oct-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 4-Nov-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Chloride (mg/kg)*	Sulfate (mg/kg)*
1610240-001	Pit 1 @ 4'	450	6.03	-	9,400	2,400	N.D.	N.D.
1610240-002	Pit 2 @ 5'	450	6.91	-	3,100	2,700	N.D.	N.D.
1610240-003	Pit 3 @ 1'	460	6.42	-	12,000	1,400	N.D.	N.D.
1610240-004	Pit 3 @ 3'	460	7.65	-	4,000	1,100	N.D.	N.D.
1610240-005	Pit 4 @ 5'	440	6.71	-	17,000	5,400	N.D.	N.D.
1610240-006	Pit 5 @ 3'	460	7.16	-	6,500	2,000	N.D.	N.D.
1610240-007	Pit 6 @ 3.5'	440	6.16	-	2,500	1,100	17	N.D.
1610240-008	Pit 7 @ 3'	430	6.92	-	3,700	1,500	17	N.D.
1610240-009	Pit 8 @ 5'	410	6.10	-	20,000	4,900	N.D.	N.D.
1610240-010	Pit 9 @ 6'	440	6.16	-	2,700	1,600	N.D.	N.D.
1610240-011	Pit 10 @ 5'	480	6.28	-	2,400	1,600	N.D.	N.D.
1610240-012	Pit 11 @ 5'	470	6.62	-	2,700	2,500	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM G57	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	-	15	15
Date Analyzed:	3-Nov-2016	3-Nov-2016	-	3-Nov-2016	3-Nov-2016	3-Nov-2016	3-Nov-2016

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: JDH Corrosion Consultants, Inc.
Client's Project No.: 16218
Client's Project Name: Santa Rosa - El Noka
Date Sampled: Not Indicated
Date Received: 27-Oct-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 4-Nov-2016

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Chloride (mg/kg)*	Sulfate (mg/kg)*
1610240-013	Pit 12 @ 4'	480	7.03	-	2,900	1,700	N.D.	N.D.
1610240-014	Pit 13 @ 3.5'	460	6.65	-	4,300	2,400	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM G57	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	-	15	15
Date Analyzed:	3-Nov-2016	3-Nov-2016	-	3-Nov-2016	3-Nov-2016	3-Nov-2016	3-Nov-2016

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

Client: *Reese & Associates*
Project: *Elnoka Village*
Location: *Santa Rosa, CA*
Date: *11/22/2016*
Subject: *In-Situ Soil Resistivity Data*

Severely Corrosive
Corrosive
Moderately Corrosive
Mildly Corrosive
Progressively Less Corrosive

*Test	Location	Resistance Data From AEMC Meter					Soil Resistivities (ohm-cm)					Barnes Layer Analysis (ohm-cm)				
#	Description	2.5	5	7.5	10	15	2.5	5	7.5	10	15	0-2.5'	2.5-5'	5-7.5'	7.5-10'	10-15'
1	Position 1	3.24	1.22	0.61	0.45	0.36	1551	1168	876	862	1034	1551	937	584	821	1724
2	Position 2	6.84	1.84	0.95	0.69	0.68	3275	1762	1364	1321	1953	3275	1205	940	1207	44926
3	Position 3	6.18	2.36	0.93	0.69	0.63	2959	2260	1336	1321	1810	2959	1828	735	1280	6937
4	Position 4	6.05	2.32	1.12	0.47	0.74	2896	2221	1609	900	2126	2896	1802	1037	388	NA
5	Position 5	6.78	2.34	0.91	0.62	0.66	3246	2241	1307	1187	1896	3246	1711	713	931	NA
6	Position 6	12.40	3.97	1.92	1.64	1.64	5937	3801	2758	3141	4711	5937	2796	1780	5384	NA
7	Position 7	17.30	5.28	2.61	2.04	3.37	8282	5056	3749	3907	9680	8282	3638	2471	4472	NA
8	Position 8	4.89	2.43	1.21	0.84	0.73	2341	2327	1738	1609	2097	2341	2313	1154	1315	5338
9	Position 9	5.58	2.52	1.18	0.85	1.14	2671	2413	1695	1628	3275	2671	2200	1062	1455	NA
10	Position 10	8.62	2.72	1.53	1.12	0.82	4127	2604	2197	2145	2355	4127	1903	1674	1455	2931

In-situ Locations

F.2 - Soil Investigation Report

THIS PAGE INTENTIONALLY LEFT BLANK

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS
134 LYSTRA COURT SANTA ROSA, CA 95403
TELEPHONE (707) 528-3078 FACSIMILE (707) 528-2837

Report
Soil Investigation
Elnoka Village
Santa Rosa, California

Prepared for
Oakmont Senior Living
9240 Old Redwood Highway, Suite 200
Windsor, CA 95472

By

REESE & ASSOCIATES
Consulting Geotechnical Engineers

Joseph M. Mauney
Civil Engineer No. 85560

Jeffrey K. Reese
Civil Engineer No. 47753

Job No. 202.6.1
January 19, 2017

TABLE OF CONTENTS

INTRODUCTION **1**

SCOPE **2**

PREVIOUS INVESTIGATIONS **2**

WORK PERFORMED **3**

SURFACE AND SUBSURFACE **5**

CONCLUSIONS **8**

SEISMIC DESIGN **14**

RECOMMENDATIONS **15**

Site Grading 15

Foundations 23

Spread Footings 23

Drilled Piers 25

Post-Tensioned Slabs 26

Conventional Slab-on-Grade 28

Retaining Walls 29

Pavement Construction 31

Soil Engineering Drainage 33

Supplemental Services 35

LIMITATIONS **35**

LIST OF PLATES **37**

DISTRIBUTION **38**

REFERENCES **39**

APPENDIX **58**

INTRODUCTION

This report presents the results of our soil investigation for the proposed Elnoka Village development to be constructed in Santa Rosa, California. The site is located southwest of the intersection of State Highway 12 and Melita Road and consists of 16 parcels that cover a total area of approximately 60 acres. The site location is indicated on Plate 1, Site Vicinity Map.

Preliminary plans prepared by Brelje & Race Consulting Engineers indicate that the currently proposed development will consist of a two-story care center building, a two-story recreation center building, 14 three-story apartment buildings, 2 two-story employee housing buildings and 65 one- and/or two-story cottages for single family living. The recreation center, care center and apartment buildings will all include below-grade parking, with concrete floor slabs on the lower (parking) level. The cottages and worker housing structures could have either concrete slab-on-grade floors or wood floors supported on joist above grade.

Asphalt- or concrete-paved roadways with underground utilities will serve the development internally. Two new vehicle bridges are planned: one spanning over Oakmont Creek to provide access to buildings on the southwest side of the creek, and a second bridge over Melita Creek near the north entrance to the site from Melita Road. Concrete slab-on-grade flatwork areas and walkways are also planned throughout the development.

We understand that currently proposed grading will include either placement of fills to buttress existing near-vertical cuts and/or flattening of the existing slopes to avoid construction of tall retaining walls. Because the site is sloping, we anticipate that low site retaining walls (less than about 4 feet in height) will be needed throughout the development.

SCOPE

The scope of our investigation, as outlined in our proposal dated September 29, 2016, was to review previous work by others at the site in addition to selected geologic references in our files, explore subsurface conditions, measure depth to groundwater, if encountered, and determine physical properties of the soils encountered. We then performed engineering analyses to develop conclusions and recommendations concerning:

1. Proximity of the project to active faults.
2. Site preparation and grading.
3. Foundation support and design criteria, including criteria for design of post-tensioned slabs-on-grade.
4. Support of concrete slab-on-grade floors.
5. Retaining wall design criteria.
6. Quality and compaction criteria for development of asphalt-paved roadways.
7. Soil engineering drainage.
8. Supplemental soil engineering services.

PREVIOUS INVESTIGATIONS

Three separate geotechnical investigations have been performed at the site since the early 1990s. We reviewed two investigations performed for development of the entire site: "Preliminary Geotechnical Investigation, Proposed Life Care Community Project, Highway 12 and Elnoka Lane, Santa Rosa, California," prepared by John H. Dailey, Consulting Geotechnical Engineer dated April

2, 1991 and; "Geotechnical Investigation Report, Santa Rosa Life Care Center, Santa Rosa, California," prepared by Kleinfelder, Inc., dated November 8, 1993. Giblin Associates, Consulting Geotechnical Engineers, also performed a soil investigation focusing on the northeast portion of the project site and summarized the results of their investigation in a report dated October 8, 2007.

In total, 40 test borings and 45 test pits were performed during the three geotechnical investigations. The test borings varied in depth between about 8½ and 41½ feet, and the test pits were excavated to depths of about 4 to 10½ feet. The subsurface information from these investigations was correlated with our subsurface investigation to develop conclusions and recommendations for the entire site. The approximate locations of the previous test pits and borings are presented on Plate 2, Test Pit Location Plan and Geotechnical Planning Map.

In addition to the previous investigation reports, we also reviewed a final report entitled "Summary of Earthwork Observation and Testing, Phase A & Phase 1-1996 Improvements, Pacific Lifecare/Three Bridges, Santa Rosa, California" prepared by Kleinfelder, Inc., dated November 22, 1996. The report summarizes their observation and testing services during site grading work on the northeast portion of the property, denoted as Zone 2 on Plate 2.

WORK PERFORMED

In addition to the above-mentioned geotechnical reports, we review selected published, geologic and geotechnical information in our files pertinent to the project. Aerial photography of the project site was also reviewed as part of our investigation. The literature reviewed is listed in the References section of this report.

On October 11 and 12, 2016, we were on-site to observe the surface conditions and explore subsurface conditions to the extent of 14 test pits. The approximate test pit locations are shown on Plate 2. The test pits were excavated to depths ranging from about 3 to 10½ feet with track-mounted, excavator equipment. Our project geologist and project engineer located the pits, observed the excavations, logged the conditions encountered and obtained samples for laboratory classification testing. In-place strength indicator determinations were conducted in the walls of the pits with a penetrometer. Logs of the test pits are presented on Plates 3 and 4. A description of the soils presented on the logs is provided on Plate 5. The soils are classified in general accordance with the Unified Soil Classification System explained on Plate 6. Physical characteristics for the rock descriptions included on the pit logs are explained on Plate 7.

Selected samples were tested in our laboratory to determine moisture content and classification (Atterberg Limits, percent passing the No. 200 sieve and percent free swell). The test results, including the field penetrometer data, are shown Plates 8 through 10. Detailed results of the Atterberg Limits tests are presented on Plates 11 and 12.

The test pit locations are approximate and were established by visually estimating from existing surface features. The locations of the test pits should be considered no more accurate than implied by the methods used to establish the data. At the completion of the exploration, the pits were backfilled with the excavated soils, without compaction.

Concurrent to our soil investigation, JDH Corrosion Consultants, Inc. performed an evaluation of the corrosivity of the soils at the site. Their report entitled "Soil Corrosivity Evaluation & Recommendations for Corrosion Control," dated December 9, 2016 is included herein as Appendix A.

SURFACE AND SUBSURFACE CONDITIONS

The approximately 60-acre project site is located about 5 miles east of downtown Santa Rosa, California. The site is bordered to the north by State Highway 12 and the Stonegate neighborhood. Channel Drive (within Annadel State Park) and single-family residential properties form the south border of the project. The Oakmont Subdivision Nos. 15-C and 15-D1 and a City of Santa Rosa wastewater treatment plant establish the east border. Melita Road and residential properties create the west perimeter.

The site is dominated by a northwest-southeast trending ridgeline that extends through the site just south of its center. The difference in elevation from the base of the slope to the crest of the ridge varies between about 70 and 100 feet. On the southwest–facing slope, the gradient varies between about five horizontal to one vertical (5:1) and 3:1. The south portion of the northeast-facing slope was terraced during previous site grading operations, with unretained, near-vertical cuts varying between about 10 and 25 feet in height. The north portion of the northeast-facing slope retains its natural inclination between about 8:1 and 4:1. The top of the ridge is relatively rounded, with very gently sloping areas at the crest. Beyond the base of the ridge, relatively gently sloping terrain persists throughout the remainder of the site.

Three creeks extend through the property: Annadel Creek and Oakmont Creek in the south portion and Melita Creek in the north. The creek banks are relatively steep (between about 2:1 and near vertical) and vary in height from about 2 to 16 feet. Oakmont Creek is the most substantial of the creeks. The other two creeks serve as seasonal drainages. An existing concrete vehicle bridge spans over Melita Creek in the central portion of the northeast quadrant of the site, which was constructed in the mid-1990s under the observation services of Kleinfelder, Inc.

Several existing one- and two-story residences with various outbuildings are located atop the ridge, which will be removed as part of the proposed construction. The residences are served by overhead power, underground utilities and water wells, and the houses likely have leach field septic systems. The existing residences are accessed by the asphalt-paved private roads, which will also be removed. An abandoned water well was observed in the low area at the far west extent of the site, near proposed Building No. 65. Several utility easements extend through the north and south portions of the site, which will remain in-place.

At the time of our exploration, the ground surface across most of the site was covered with moderate to dense growths of grass and weeds with occasional mature trees. Dense growths of brush and thick clusters of mature trees are located near creeks and atop the ridge.

The previous explorations, our test pits and laboratory tests indicate that the site is underlain by variable soil conditions, with discontinuous layers of fill materials and natural silty, clayey, sandy and gravelly soils overlying firm bedrock materials. In general, the site soils can be divided into five general soil zones, demarcated as Zones 1 through 5 on Plate 2. In Zones 1 and 2, the upper approximate 1 to 13 feet of soil consists of existing fill materials of variable

strength and consistency. Laboratory tests indicate that the fill materials are low to high in expansion potential. That is, the soils would tend to undergo low to high strength and volume changes with seasonal variations in moisture content. Natural soils are present underlying the fill. In Zone 1, the natural soils were relatively soft and judge to be compressible under anticipated loading conditions.

In Zone 3, deep cuts on the order of about 10 to 25 feet were performed during previous site work. The soils exposed in the level terraces generally consists of minor fills of low expansion potential and weathered bedrock materials (subsequently described). In Zone 4, the upper approximate 1 to 3 feet consists of relatively weak and compressible, low expansive sandy silt and silty sand topsoils, dense clayey gravels and highly expansive clayey soil overlying relatively shallow bedrock. The expansive clays were generally encountered within the upper 2½ feet of the pits, and shrinkage cracks were observed to a depth of about 2 feet in the clay. Surface topography suggest localized areas of existing fill in Zone 4. The approximate locations of existing fill materials at the entire site are shown on Plate 2. In Zone 5 adjacent to Annadel and Oakmont Creeks, the estimated upper 8½ to 14 feet consists of weak and porous sandy, silty and clayey flood deposits of low expansion potential. Highly expansive clayey flood deposits were locally encountered in Test Pit 12. In general, the upper 12 inches to 3 feet of soil at the site contains abundant roots and root fibers, and most of the soils encountered contained a significant amount of rounded gravel.

Previous explorations and our test pits generally bottomed in deeply weathered bedrock materials of the Glen Ellen Formation. While the Glen Ellen Formation is a bedrock formation,

the overall physical characteristics of the materials are mostly that of very firm soil or soft rock. In the test pits, sand and gravel conglomerate, sandstone and siltstone/mudstone bedrock materials were encountered. The bedrock materials were observed to be low to moderate hardness, friable to moderately strong and weakly cemented. Bedrock materials of the Sonoma Voncanics Group are exposed in outcroppings at the south border of the site and have been encountered during pervious investigations in the south area.

Groundwater was not encountered in the test pits during our exploration. However, groundwater has been encountered as shallow as about 3 feet during previous investigations. Groundwater conditions and seepage levels can vary seasonally and could rise and fall several feet annually. In addition, free water in the form of shallow seepage can commonly migrate downslope through porous soils and permeable rock materials. Determination of the precise depth to groundwater, extent of seasonal water level fluctuations or the existence of perched groundwater conditions is beyond the scope of this investigation.

CONCLUSIONS

Based on the results of our field exploration, laboratory tests and engineering analyses, we conclude that, from a soil engineering standpoint, the site can be used for the proposed construction. The most significant soil engineering factors that must be considered in design and construction are:

1) The presence of existing fill materials.
2) Weak, compressible natural soils.

3) Local areas of near-surface clayey soils that exhibit a high expansion potential.

4) The presence of the above-mentioned soils overlying firm bedrock materials on sloping terrain.

5) Differential supporting conditions beneath buildings.

6) The potential for scour in creek channels.

7) Potentially high groundwater levels with respect to the proposed below-grade construction.

The previously-referenced final report by Kleinfelder, Inc. indicates that the existing fills present in Zone 2 were properly placed and compacted under soil engineering observation and testing services. Therefore, we judge that the existing fill materials in Zone 2 will be suitable for support of the planned improvements. However, we anticipate that the upper portion of the soils in Zone 2 will need to be improved, as grading in the area was performed more than 20 years ago and the area has since been covered with a relatively dense growth of grass and weeds.

We could find no evidence during our investigation or in our review of the previous investigations by others to indicate that the existing fills in Zones 1, 3, 4 and 5 were properly placed and compacted under soil engineering observation and testing services. We judge that such fills could undergo significant total and/or differential settlements under the anticipated loading conditions. Therefore, we conclude that the existing fill materials in these areas are not suitable for fill, foundation or slab support. It will be necessary to remove (overexcavate) the existing fills, where present, within proposed building areas and replace the materials as properly compacted fill, as subsequently recommended.

Our experience indicates that weak, compressible soils can undergo considerable strength loss and settlement when loaded in a saturated condition. Where evaporation is inhibited by footings, slabs or fill, eventual saturation of the underlying soils can occur. Expansive soils can shrink and swell with seasonal changes in moisture content and can heave and/or distress lightly loaded footings or slabs. Accordingly, we conclude that the weak, compressible and expansive upper soils are not suitable for foundation or slab support in their present condition. To reduce the risk of total and/or differential settlements, we conclude that it will be necessary to upgrade the weak upper soils by removal and replacement as properly compacted fill. As an alternative, foundations could be extended through the weak and expansive upper soils to gain support from the underlying firm bedrock materials (e.g. deepened spread footings, drilled piers, etc.). Foundations designed to tolerate compressible and expansive soil movements could also be utilized (e.g. post-tensioned slabs, mat slabs, grid foundations, etc.). Because of the potential for differential supporting conditions to be encountered at pad grade level, we judge that some of the post-tensioned or mat slab-on-grade foundations, if used, would need to be designed so as to recognize the presence of both expansive and compressible soils.

Where existing fills, weak, compressible and/or expansive soils are overlying firm bedrock materials on sloping terrain, the soils generally undergo a phenomenon known as soil creep. Soil creep is a long-term, gradual downhill migration of soil under the influence of gravity and is generally on the order of a fraction of an inch per year. Creep soil movements can exert lateral forces on the upslope sides of foundations. In addition, the soils can creep away downslope sides of foundations, reducing lateral support. Accordingly, we conclude that the

foundations for the proposed structures in sloping areas would need to be designed to account for lateral forces from soil creep, unless site grading is performed to remove and replace the creep-affected soils as properly compacted fill.

Total and/or differential settlement and resultant distress to structures can occur in areas underlain by significant differential thicknesses of fill, or in areas that transition from cut to fill. Based on our review of preliminary development plans, we anticipate that a number of structures in Zone 4 could be positioned in areas that transition from cut on the uphill side to fill on the downhill side. To provide more uniform support and reduce the risk of distress, we conclude that it will be necessary to verify that all slab-on-grade floor areas are underlain with a sufficient thickness of properly compacted fill, even those located in areas of planned cut. Differential fill thicknesses should also be limited to help control differential settlement, as subsequently recommended.

Liquefaction, a loss of shear strength, and densification, a reduction in void ratio, are phenomena associated with loose, cohesionless, sands and gravels subjected to ground shaking during earthquakes and can result in unacceptable total and/or differential settlements. Phenomena associated with strong ground shaking at creek banks include lateral spreading and soil lurching. Lateral spreading is a liquefaction-induced horizontal displacement of a soil mass that generally occurs down a slope or toward an open slope face. Soil lurching is a virtually instantaneous lateral displacement of a soil mass out of a slope. Whether such phenomena would actually occur or not depends on complicated factors such as intensity and duration of ground shaking at the site, response characteristics of the underlying materials and groundwater

conditions. Because the sandy and gravelly soils encountered in our test pits and in the previous test borings by others are relatively dense and/or contain a sufficient amount of clayey or silty fines, we judge that the risk of liquefaction at the site can be considered low. Therefore, we also judge that the risk of lateral spread can be considered low. However, the risk of other earthquake-induced bank instabilities and soil lurching cannot be disregarded. The bridge foundation systems recommended herein are intended to reduce potential distress should such bank instabilities occur.

The risk of soil loss from erosion processes (such as scour) and resultant bank instability must be considered for structures positioned near creek banks. Therefore, we judge that buildings should be sufficiently set back from creek channels, as planned. In addition, we believe that the bridge foundations would need to be bottomed below an imaginary 5:1 line extended up from the bottom of the creek channels. Bridge abutments could also be setback from the top of banks, or rock riprap could be placed to improve bank stability.

In driveway and parking areas, we believe that pavements consisting of asphalt concrete or Portland cement concrete and aggregate base can be placed directly on properly prepared site soils. To help reduce the risks of future heave, settlement and resultant pavement distress, it will be necessary to verify that expansive soils, where exposed at subgrade, have not dried and cracked.

We judge that slab-on-grade floors in cottage garage areas and exterior concrete flatwork not adjacent to buildings can be supported directly on properly prepared on-site soils, provided the risk of future settlement and heave and possibly more than normal cracking are acceptable.

Such slabs would need to be structurally separated from adjacent foundations and allowed to float. In areas where minor garage/exterior slab movement and potential resultant distress would be considered undesirable, the subsequent recommendations for building area slab-on-grade floors could be implemented.

Because of potentially high groundwater levels, dewatering measures could be needed to facilitate construction in below-grade parking areas. During previous investigations, groundwater was encountered as shallow as about 3 feet below the adjacent ground. We anticipate that in some winter months, the groundwater could possibly rise to about this level. We conclude that design measures should be implemented to mitigate potential detrimental effects from high groundwater in Zones 1 and 5.

It should be understood that the factors that influence slope stability are complex, and there is an inherent risk on any hillside site. We judge that properly designed and constructed improvements will be stable, and, provided the recommendations presented herein are implemented, the risk of future instability will be within the range generally associated with construction on such hillsides in Sonoma County area.

For site preparation and building foundation design and installation in accordance with our recommendations, we judge that total settlements would be less than about 1-inch. We believe that post-construction settlements should be about 1/2-inch, or less.

SEISMIC DESIGN

The geologic maps reviewed did not indicate the presence of active faults at the site, and the project is not located within a presently designated Alquist-Priolo Earthquake Fault Zone. Therefore, we judge that there is little risk of fault-related ground rupture at the site during earthquakes. In a seismically active region such as Northern California, there is always some possibility for future faulting at any site. However, historical occurrences of surface faulting have generally closely followed the trace of the more recently active faults. The closest faults generally considered active are the Rodgers Creek fault zone located approximately 3 miles to the southwest, the Maacama fault zone, south extension, located approximately 10 miles to the northwest and the San Andreas fault zone located approximately 24 miles to the southwest.

Strong ground shaking will occur during earthquakes. The intensity at the site will depend on the distance to the earthquake epicenter, depth and magnitude of the shock and the response characteristics of the materials beneath the site. Because of the proximity of active faults in the region and the potential for strong ground shaking, it will be necessary to design and construct the project in strict accordance with current standards for earthquake-resistant construction.

We have determined the seismic ground motion values in accordance with procedures outlined in Section 1613 of the 2016 California Building Code (CBC). Mapped acceleration parameters, S_S and S_1, were obtained by inputting approximate site coordinates (latitude and longitude) into earthquake ground motion software developed by the United States Geological Survey (USGS). Based on our review of available geologic maps and knowledge of the

subsurface conditions, we judge that the site can be classified as Site Class C, as described in Table 20.3-1 of American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard ASCE/SEI 7-10. Using corresponding values of site coefficients for Site Class C and procedures outlined in the CBC, the mapped acceleration parameters were adjusted to yield the design spectral response acceleration parameters, S_{DS} and S_{D1}. The following earthquake design data summarize the results of the procedures outlined above.

2016 CBC Ground Motion Parameters

Site Class	C
Mapped Spectral Response Accelerations:	
S_S	1.756 g
S_1	0.698 g
Design Spectral Response Accelerations:	
S_{DS}	1.171 g
S_{D1}	0.605 g

RECOMMENDATIONS

Site Grading

Areas to be developed should be cleared of existing structures, debris, grass and vegetation, where encountered. Debris resulting from demolition of the existing structures, including foundations and associated subsurface utilities, should be removed from the site. Designated trees should be removed, and the root systems excavated. The resultant voids from demolition and tree removal should be backfilled with compacted soil, as subsequently described.

The surfaces then should be stripped of upper soils containing abundant root growth and organic matter. We anticipate that the depth of stripping needed will average about 3 inches. The strippings should be removed from the site, stockpiled for reuse as topsoil or mixed with at least five parts of soil and used as fill at least 10 feet away from the structures, walkways or paved areas.

Wells, septic tanks, leach fields, or other underground obstructions encountered within improvement areas should be removed, filled with soil or granular material that is properly compacted or be capped with concrete, as determined by the appropriate regulatory agency or the soil engineer.

After clearing and stripping, excavations can be performed as necessary. Undocumented existing fills and soil stockpiles in Zones 1, 3, 4 and 5 encountered within building, improvement and pavement areas should be removed (overexcavated) for their full depth. The existing fills in Zone 2 can remain in-place, as previously discussed. However, because of the current surface treatment and seasonal weather exposure over the last 21+ years, we recommend that the upper 2 feet of existing fill in Zone 2 be removed and replaced as properly compacted fill.

Overexcavations to remove existing fills should occur within the building areas and extending to at least 5 feet beyond the perimeters, and to at least 3 feet beyond any adjacent exterior concrete slab areas (i.e. building envelopes). Based on the data from our test pits, topographic information from old surveys and the subsurface information from previous investigations, we estimate that the depth of overexcavation to remove the existing fills at the site will locally vary between about 1 and 13 feet below the adjacent ground. However, localized

deeper overexcavation could be needed to remove deeper existing fills, if encountered. Because the actual depth of excavation to remove existing fills could vary, we suggest contract documents contain provisions to account for such variations.

If it is desired to use spread footings bottomed at minimum depth on compacted fill or conventional slab-on-grade floors, the weak, compressible natural soils should also be overexcavated for their full depth within the building envelopes. Based on our test pits and previous work by others, we anticipate that the depths to remove the weak, compressible natural soils would likely vary as follows:

Zone	Approximate Depth to Remove Weak, Compressible Soils (below existing grade)
1	2 to 14 feet
2	2 feet
3	2 to 4 feet
4	1 to 3 feet
5	4 to 13 feet*

* Actual depth of overexcavation in Zone 5 should be further evaluated by the soil engineer when actual foundation loading conditions have been determined.

The depths of overexcavations should also be adjusted, as needed, to provide space for at least 12 inches of properly compacted fill of low expansion potential beneath minimum depth footings and floor slabs. In expansive soil areas, the depth of the overexcavation should be

further adjusted, as needed, to provide space for a blanket of at least 30 inches of approved, on-site or imported fill of low expansion potential over the expansive soils, where encountered.

If mat or post-tensioned slab foundations designed to tolerate compressible and expansive soil movements are utilized for foundation support, the above recommendations for overexcavation of weak, compressible upper soils can be reduced to provide at least 12 inches of compacted fill below planned pad grade elevation in Zones 1 through 4 and at least 2 feet of compacted fill beneath mat or post-tensioned slabs in Zone 5. Any existing fills beneath post-tensioned or mat slabs should still be removed for their full depth. Expansive soils encountered beneath post-tensioned or mat slab foundations should be fully prewelled, as subsequently described. If drilled piers or deepened spread footings bottomed in bedrock are used in conjunction with raised-wood floors, no removal of existing fills or weak upper soils would be necessary.

To reduce the risk of total and/or differential settlement, we recommend that differential fill thickness under a building foundation or floor slab be limited to 6 feet or less. This may result in the need for overexcavation within planned cut or minor fill areas and refilling with compacted fill. The indicated variation in fill thickness could be increased with increased compaction of the fills. We can provide specific recommendations, if desired. We recommend contract documents contain provisions to cover the costs for additional overexcavation and fill placement to reduce differential fills, if needed.

We anticipate that, with the exception of organic matter and rocks or hard fragments larger than 4 inches in diameter, the majority of excavated weak, compressible upper soils and

existing fill materials will be suitable for reuse as compacted fill. However, the expansive clayey soils encountered across the site should not be reused as fill within the upper 30 inches of planned pad grade elevation where conventional floor slabs and minimum depth footings are used.

Imported fill, if needed, should be low in expansion potential and have a Plasticity Index of 15 or less. Imported fill material should also be free of organic matter and rocks or hard fragments larger than 4 inches in diameter. Imported materials should be tested and approved by the soil engineer prior to delivery to the site.

The surfaces exposed by stripping or overexcavation should be scarified to a depth of at least 6 inches, moisture conditioned to near optimum (at least 4 percent above optimum for on-site highly expansive clayey soils and to close any shrinkage cracks for their full depth) and compacted to at least 90 percent relative compaction.[1] Approved on-site or imported fill then should be spread in 8-inch-thick loose lifts, moisture conditioned and similarly compacted.

Where fills are placed on slopes inclined at about 6:1 or steeper, level keyways at least 10 feet wide should be excavated along the toe of the planned new fills. The keyway should bottom into firm underlying soil or bedrock below existing fills, weak upper soils and creep-affected materials. For estimating purposes, the depth of keyways can be assumed to extend at least 3 feet below the original ground surface, as measured on the downhill side, but could be deeper if

1 Relative compaction refers to the in-place dry density of fill expressed as a percentage of maximum dry density of the same material determined in accordance with the American Society for Testing and Materials (ASTM) Standard ASTM D 1557-12 laboratory compaction test procedure. Optimum moisture content refers to the moisture content at maximum dry density.

deeper weak upper soils are encountered. Subsurface drainage facilities will be needed at the rear of the keyways and could be needed at intermediate bench levels, depending on seepage conditions, as determined in the field by the soil engineer. Cleanout risers should be installed at the uphill ends of subdrains. The cleanouts should be extended to the final ground surface and be capped with a suitable cover. As fill placement continues upslope, level benches should be excavated into firm underlying soil or bedrock to key the new fills into the hillside and remove the existing old fills and creep-affected materials, where encountered. Plate 13 shows a typical cross-section of our general recommendations for hillside fill placement. It should be understood that the cross-section is a typical representation of hillside grading and keyways. The actual location depth and extent of keyways and subdrains should be determined in the field during grading when actual conditions are exposed. Therefore, we recommend that the contract contain provisions for variations in keyway excavations and additional subdrainage quantities, if appropriate.

Areas mapped as heavy creep and landslides on Plate 2 that interfere with building envelopes or are located upslope of planned structures should be repaired by removing the unstable soils and replacing the excavated materials as compacted fill buttresses. The buttresses should be keyed and benched into firm underlying bedrock with internal subdrainage, as discussed above for general hillside fill placement. However, the initial keyways at the toe of the buttress fills should be at least 12 feet wide, and an additional subdrain blanket should be installed at the uphill edge of the compacted fill buttress. Normally, such uphill subdrains must

blanket the potential slide zone and, therefore, contain two or three times more drainrock than the lower subdrain.

We anticipate that the depth of the initial keyways at the landslide areas will bottom about 7 feet below the adjacent ground for the north landslide and about 4½ feet below the existing ground surface for the south landslide. The initial keyway at the areas of heavy creep should be planned to be about 4 feet deep. The slope failures in Zone 3 are relatively shallow and can be removed without buttressing, as the near-vertical slopes are planned to be retained or regraded. The actual location, depth and extent of the buttress keyways, benches and subdrains will vary and must be determined in the field by the soil engineer when the actual conditions are exposed during the grading. We recommend that the landslide and heavy creep areas be potholed during site grading to establish the extent and depth of the slide debris and heavy creep soil zones.

It is our experience that weak upper soils can tend to trap considerable amounts of water into the late spring or early summer. Accordingly, we believe that grading, particularly if performed early in the construction season, would likely require more than normal effort to satisfactorily excavate and/or compact the materials. In addition, the upper 8½ to 14 feet of soils in Zone 5 exhibit relatively low strength and are likely highly susceptible to instability under the weight of conventional grading equipment, particularly when saturated. Because of planned underground basements and the possibility for deep overexcavation to remove the weak, compressible soils, local, soft and saturated soil conditions at the bottom of excavations should be anticipated. The need for additional overexcavation to remove unstable soils, imported

granular working pads, geotextile fabrics, dewatering systems, lime- or cement-treating techniques or other measures could be needed to complete the building pads and develop subgrade. Accordingly, we suggest that the possible need for such measures be accounted for in the contract documents.

The test pits were backfilled with the excavated soils, but the materials were not compacted. Therefore, the test pit backfills constitute local zones of highly compressible materials. Where encountered in planned improvement areas, the pit backfills should be removed for their entire depth, and the soils be replaced as properly compacted fill. As an alternative, foundation elements could be deepened accordingly to extend below the test pit backfills.

Finished slopes should be trimmed to expose firm material and should be no steeper than 2:1. Slopes over 3 feet high should be planted with fast-growing, deep-rooted ground cover to help reduce erosion. The face of finished fill slopes should be thoroughly compacted by slope rolling and trimming or constructed wider than planned and then trimmed to expose dense, well compacted material. Sloughing, erosion and sliding are common on newly graded slopes, especially during the first few winters. Therefore, supplemental erosion inhibitors may be prudent to apply, such as jute mesh or other commercially available materials. Any such sloughing, erosion or sliding that does occur should be repaired promptly before it can enlarge.

Foundations

Spread Footings

Provided site grading is performed as indicated above, spread footings should be underlain by at least 12 inches of properly compacted fill of low expansion potential. Footings underlain by compacted fill should be bottomed a minimum of 12 inches below lowest adjacent grade. Foundation support could also be achieved from footings bottomed on firm underlying bedrock. Footings bottomed on bedrock materials footings should be planned to be a minimum of 18 inches deep. On slopes, footings should penetrate at least 12 inches into firm bedrock materials, extend at least 8 horizontal feet from the face of the nearest downslope and be stepped, as necessary, to provide level and up to 10 percent sloping bottoms. We estimate that the depth of deepened spread footings in Zones 3 and 4 will likely vary between 18 inches (where bedrock is currently exposed) and 3½ feet below the existing ground.

Spread footings can be designed to impose dead plus code live loads and total design loads (including wind or seismic forces) bearing pressures of 2,000 and 3,000 pounds per square foot (psf), respectively. In sloping areas inclined 6:1 or steeper in Zone 4, footings parallel to contour should be designed to withstand creep soil forces. We anticipate that the depth of the creep soil zone will average about 2 feet. The creep force can be assumed as an at-rest equivalent fluid pressure of 55 pounds per cubic (pcf).

Resistance to lateral loads can be obtained from passive pressure and soil friction. We recommend the following criteria for design:

Passive Earth Pressure	=	300 pcf, equivalent fluid, neglect the upper 12 inches unless confined by pavement or slab
Soil Friction Factor	=	0.30

On slopes steeper than 4:1, the spread footing foundations should be well reinforced and well tied together with grade or tie beams. Tie beams, if used, should be at least 12 inches square and be reinforced with at least two No. 5 (or three No. 4) bars. Upslope/downslope and cross-slope oriented footings or tie beams should be no farther apart than about 20 feet.

Deepened spread footings could be used for foundation support of the bridges. Spread footings should be bottomed on firm, natural soil or weathered bedrock materials below an imaginary 5:1 line extended up from the bottom of the creek channel. Based on cross sections developed during our investigation, we judge that such footings could vary in depth from about 3 to 10 feet, or more, depending on the positioning of the footings relative to the creek bank. The spread footings should be designed to resist an active equivalent fluid pressure of 55 pcf above the imaginary 5:1 line. If footings are planned to be positioned closer than about 5 feet to the top of the banks, we should be consulted to evaluate the need for deepening of footings below soils subject to future scour.

Resistance to lateral loads for the bridge foundations can be obtain using the values described above, but should be neglected above the 5:1 line or within 8 horizontal feet of the bank face, whichever is deeper.

Drilled Pier Foundations

Drilled piers used in conjunction with raised wood floors could also be used for foundation support of the cottages and employee housing buildings. In areas flatter than 6:1 in Zone 4, the drilled piers should be at least 12 inches in diameter and bottomed at least 10 feet below the existing ground surface. In areas steeper than 6:1 in Zone 4 and at the employee housing buildings in Zone 1, the piers should be bottomed at least 12 feet below the adjacent grade. The foundation piers should extend at least 7 feet into firm underlying material, as determined in the field by the soil engineer.

Vertical loads on the piers can be carried below the upper 3 feet in skin friction using a value of 700 psf. End bearing should be neglected because of the difficulty of cleaning out small diameter holes and the uncertainty of mobilizing end bearing and skin friction simultaneously. Resistance to lateral loads on piers can be obtained from a passive equivalent fluid pressure of 300 pcf applied over two pier diameters. The passive pressure can be assumed to commence at the ground surface, but should be neglected in the upper 24 inches, unless confined by pavement or slab. On slopes steeper than about 6:1, the piers should be designed to resist the creep soil forces outlined above for spread footings.

Piers beneath perimeter and bearing walls should be interconnected with grade beams designed to support the calculated structural loads. In lieu of grade beams under bearing walls, the framing must be sufficient to carry the loads, as required by the CBC. Piers should be reinforced as determined by the structural design engineer. In general, piers should be spaced no closer than three diameters, center to center.

To retard wet concrete from settling, pier holes should not contain more than 3 inches of slough. It may be necessary to tamp the slough with a heavy timber prior to concrete placement, as determined in the field by the soil engineer.

Groundwater and caving soils were not encountered within the anticipated pier depths during our exploration. However, caving soils and/or perched groundwater could be encountered during pier drilling operations, particularly if drilled in the winter and early spring months or near creek channels. If caving soils are encountered, it may be necessary to case the holes. If groundwater is encountered, it may be necessary to dewater the holes or place the concrete by an approved pumping or tremie method.

Post-Tensioned Slab Foundations

The following presents preliminary design parameters for post-tension slab foundations. We can provide specific recommendations for post-tensioned slabs as planning progresses and exact building locations are established. Provided the site is graded as outlined above, post-tensioned slabs should be underlain by at least 12 inches of properly compacted fill in Zones 1 through 4 and 2 feet of fill in Zone 5. We recommend that the post-tensioned slabs be designed in accordance with *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* (PTI DC10.5), current edition, and the criteria in the latest adopted edition of the CBC. Uniform thickness post-tensioned slabs should be at least 10 inches thick with at least an additional 2-inch thickened edge for stiffening.

Based on our field and laboratory data, we judge that expansive soil conditions would likely control foundation design in Zone 4 where no cuts are planned. For slabs positioned in expansive soil areas, we recommend the following parameters be used for design:

Center Lift	Edge Lift
$E_m = 7.0$ feet	$E_m = 5.0$ feet
$Y_m = 2$ inches	$Y_m = 1$ inch

In the other Zones, areas where excavations remove the expansive clays or at building pads that transition from cut to fill, compressible soil movements will likely govern the post-tensioned foundation design. Accordingly, we recommend that the slab design be performed using the following criteria for the following compressible soil conditions:

Anticipated differential settlement between the center and edge of slabs = 2½ inches

For design, an allowable bearing value of 1,000 psf and an effective friction factor of 0.30 can be used. The slab subgrade soils should be thoroughly moistened and be smooth, firm and uniform. Slab subgrade should not be allowed to dry prior to concrete placement. A capillary moisture break should be provided, as subsequently recommended

From our experience, we have observed that where trees or slopes are located near foundations at expansive soil sites, an increased risk of foundation distress resulting from significant loss of moisture in the soils adjacent to and beneath the slab foundations can occur. Accordingly, to reduce the risk of future foundation distress as a result of possible future

differential shrink/swell soil movements, we recommend that a 36-inch deep thickened edge be provided at the slab perimeter where: 1) the distance from buildings to trees is less than one-half the trees maximum mature height; or, 2) where the slab perimeter is within about 8 feet of the face of a slope. The actual need for and location of the thickened edges should be determined during final design.

Conventional Slab-on-Grade Floors

Provided the site is prepared as recommended above, at least 12 inches of properly compacted, approved on-site or imported fill materials of low expansion potential should underlie floor slab areas. In addition, any expansive soils in conventional slab-on-grade areas should be covered by at least 30 inches of fill of low expansion potential. Basement level slabs-on-grade should be at least 5 inches thick. Single-family garage or other slabs should be at least 4 inches thick. Slabs should be reinforced with bars to reduce cracking. Actual slab thickness and reinforcing used should be determined by the structural design engineer or architect based on anticipated use and performance. Slabs should be underlain by a capillary moisture break and cushion layer consisting of at least 4 inches of free-draining, crushed rock or gravel (i.e. slab rock) at least 1/4-inch and no larger than 3/4-inch in size. Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moisture conditioned and be smooth, firm and uniform. Slab-on-grade subgrade should not be allowed to dry prior to concrete placement

Moisture vapor will condense on the underside of slabs. Where migration of moisture vapor through the slabs would be detrimental, a vapor retarder should be provided between the

supporting base material and the slab. Two inches of moist, clean sand could be placed on top of the membrane to aid in curing and to help provide puncture protection. However, the architect or design engineer should determine the actual use of sand. The use of a less permeable and stronger membrane should be considered if sand is not placed for puncture protection, or where the flooring manufacturer requires a vapor barrier. Concrete design and curing specifications should recognize the potential adverse affects associated with placement of concrete directly on the membrane.

Retaining Walls

Retaining walls that are free to rotate slightly and support level and up to 3:1 sloping backfill should be designed to resist an active equivalent fluid pressure of 40 pcf acting in a triangular pressure distribution. Where the backfill slope is steeper than 3:1, the pressure should be increased to 55 pcf. If the wall is constrained at the top and cannot tilt, the design pressures for level and sloping backfill should be increased to 55 and 70 pcf, respectively. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an added surcharge pressure equivalent to 1½ feet of additional backfill. Where an imaginary 1½:1 line projected down from any adjacent foundations intersects retaining walls, the portion of the wall below the intersection should be designed for an additional uniform surcharge of load of 100 psf.

Retaining wall foundations can be designed in accordance with the recommendations above for building foundations. Where foundations for retaining walls are located on or near slopes, footings should be deepened to provide at least 8 horizontal feet of confinement from the

edge of the footing to an adjacent slope face. Where deepening of footings is needed, foundation wall heights and corresponding footing widths may vary beyond those indicated on the plans, and should be determined in the field in consultation with the project structural engineer. Accordingly, we suggest contract documents contain provisions to account for such variations. In addition, to avoid construction delays, schedules could be provided by the structural design engineer for increased wall heights, footing widths and reinforcing.

In general, retaining walls should be full backdrained. The backdrains should consist of 4-inch-diameter, perforated rigid plastic pipes sloped to drain to outlets by gravity and free-draining gravel or crushed rock (i.e. drainrock). The drainrock should consist of Class 2 Permeable Material per current Caltrans Standard Specifications. As an alternative, any clean washed durable rock product containing less than 1 percent soil fines , by weight, could be used if the rock is separated from the adjacent soil and covered by a nonwoven, geotextile fabric weighing at least 4 ounces per square yard (such as Mirafi 140N, or equivalent). The drainrock should extend to within 12 inches of the surface. The upper 12 inches should be backfilled with compacted soil to inhibit surface water infiltration, unless capped with a concrete slab. Where interior slabs-on-grade are placed inside retaining walls, the flow-line of the perforated pipe in the wall backdrain should be at least 8 inches lower than the level of the floor slab. Where migration of moisture through retaining walls would be detrimental, the walls should be waterproofed. The ground surface behind retaining walls should be sloped to drain.

Retaining wall backfills should be placed in lifts and compacted to at least 90 percent relative compaction. Light compaction equipment should be used to avoid overstressing the

walls. Retaining walls can yield slightly during backfilling. Therefore, backfill operations should be completed prior to building on or adjacent to the walls.

Because of the potential for high groundwater levels, below-grade walls in Zones I and 5 should be designed to resist additional hydrostatic lateral loads, unless a retaining wall backdrain system with a gravity outlet is provided. If the walls are not provided with a backdrain system, the above active pressures for cantilever and constrained conditions should be increased to 85 and 100 psf, respectively.

As outlined in the 2016 CBC, it may be necessary to design retaining walls to resist additional lateral soil loads imposed during seismic shaking. Accordingly, based on the Mononobe-Okabe Method, we have computed the following dynamic component of total thrust induced on the wall for varying backslope inclinations. A peak ground acceleration (PGA) of 0.678 g was used for the analysis.

Backslope Inclination (β)	Dynamic Component of Total Thrust* (lbs/ft)
$0 \leq \beta \leq 8:1$	$9\ H^2$
$8:1 < \beta \leq 4:1$	$13\ H^2$
$4:1 < \beta \leq 3:1$	$19\ H^2$

* The dynamic component of total thrust should be applied as a line load at a height of 0.6H above the base of the retaining wall, where H is height of the retaining wall.

Pavement Construction

For preliminary planning purposes, based on our experience with similar projects and soils, we recommend that the following minimum pavement sections for driveways and parking areas:

Material	Auto Parking	Automobile and Pickup Truck Driveways
Class 2 Aggregate Base	6"	8"
Asphalt Concrete	2½"	2½"

Such pavements should be suitable for auto and light pickup truck traffic. Heavy delivery truck traffic and/or increased wheel loads from garbage trucks could reduce the useful life of such pavement sections and cause premature distress and increased maintenance. Longer pavement life and lower maintenance can be achieved by thickening driveway sections to at least 3 inches of asphalt and at least 10 to 16 inches of aggregate base. Because of concentrated heavy wheel loads at dumpster lift points, reinforced concrete slabs should be considered at those locations.

Prior to subgrade preparation, underground utilities in the paved areas should be installed and properly backfilled. Pavement subgrades should be prepared by scarifying to a depth of at least 6 inches, moisture conditioning to slightly above optimum (at least 3 percentage points above optimum for on-site clayey soils) and compacting to at least 95 percent relative compaction. Finished subgrade should be smooth, firm, uniform and nonyielding. Approved aggregate base materials should be spread in layers, moisture conditioned and compacted to at least 95 percent relative compaction. The aggregate base surface should also be firm and nonyielding.

The flexible pavement materials should conform to the quality requirements of Caltrans Standard Specifications, current edition, and the requirements of the City of Santa Rosa.

Soil Engineering Drainage

Ponding water will soften site soils and would be detrimental to foundations. It is important that the areas adjacent to the buildings be sloped to drain away from foundations. We recommend that good, positive surface drainage away from the buildings consisting of at least 1/2-inch per foot extending at least 4 feet out be provided around the buildings. All surface drainage should be diverted away from cut and fill slope faces by means of top-of-slope ditches, berms or approved equivalents.

The roofs should be provided with gutters, and the downspouts should discharge onto paved areas, splash blocks draining at least 30 inches away from foundations or be connected to rigid plastic pipelines that discharge into planned storm drain facilities.

Where irrigated landscape areas abut the buildings, excess water can be introduced into soil layers along the edges of the building, tending to soften soils in the foundation areas and increase the risk of potential heave or settlement of the floor slab and increase the risk of infiltration of seepage water into underfloor areas. We believe that the installation of the recommended compacted fill pad that extends to at least 5 feet beyond a building perimeter should provide an effective barrier to the infiltration of excess waters from landscape areas positioned outside the building envelope. However, as an added precaution, landscape planters abutting the building could be lined with a plastic membrane (6-mil Visqueen, or equivalent) and

be provided with a subdrain that outlets into planned site drainage systems (gutters, storm drains, etc.). In addition, any cold joints in the perimeter foundations below grade should be hot-mopped or water-proofed on the exterior side in some manner where a landscape planter abuts the building.

To help provide and outlet for water that could accumulate beneath slab floors, we recommend that perforated rigid plastic pipes be embedded below the underslab rock. The underslab subdrain system should be configured to drain each bay created by interior and/or perimeter foundations. A typical cross-section of the recommended underslab subdrain is shown in Plate 15. The underslab subdrain systems should be connected to nonperforated outlet pipes that extend through or beneath the perimeter foundations to suitable discharge points. We could provide additional consultation concerning the configuration and location of the underslab subdrain systems during final design.

In sloping areas 6:1 or steeper, foundation subdrains may need to be provided along the uphill building foundations and at intermediate grade beam levels. Foundation subdrains should consist of trenches about 12 inches wide and about 18 inches deep that are filled with drainrock. A 3-inch-diameter, perforated, rigid-wall plastic pipe should be installed in the trench on a bed of about 1 to 2 inches of drainrock. The drainrock should conform to the quality requirements for Class 2 Permeable Material per Caltrans Standard Specifications. The rock should extend to within 6 inches of the surface. The upper 6 inches should consist of compacted, excavated soil to inhibit surface water infiltration. A nonperforated outlet pipe should be provided that discharges by gravity to a suitable outlet location. A typical cross-section of a foundation

subdrain detail is shown on Plate 16. Roof downspouts and surface drains must be maintained entirely separate from foundation and underslab subdrains.

Supplemental Services

We should review the grading and foundation plans for conformance with the intent of our recommendations and to establish preliminary location and extent of recommended keyways, subdrains and buttresses. During site grading operations, we should provide intermittent soil engineering observation and testing to determine the conditions encountered and modify our recommendations, if warranted. Field and laboratory tests should be performed to ascertain that the specified moisture content and degree of compaction are being attained.

We should observe footing excavations and pier drilling operations to verify that the conditions are as anticipated and to modify our recommendations, if warranted. Concrete placement and reinforcing should be checked as stipulated on the project plans or as required by the Building Department. It is our understanding that approval from the Building Department must be obtained prior to the placement of concrete in foundation elements.

LIMITATIONS

We have performed the investigation and prepared this report in accordance with generally accepted standards of the soil engineering profession. No warranty, either express or implied, is given. It should be understood that our services were limited to the scope of work outlined above and specifically excluded other services including, but not limited to, an

evaluation or analysis of soil chemistry, corrosion potential, mold and soil/groundwater contamination.

Subsurface conditions are complex and may differ from those indicated by surface features or encountered at test pit locations. Therefore, variations in subsurface conditions not indicated on the logs could be encountered. If the project is revised, or if conditions different from those described in this report are encountered during construction, we should be notified immediately so that we can take timely action to modify our recommendations, if warranted.

Supplemental services as recommended herein are in addition to this investigation and are performed on an hourly basis in accordance with our Standard Schedule of Charges. Such supplemental services are performed on an as-requested basis, and we accept no responsibility for items we are not notified to check, or for use and/or interpretation by others of the information contained herein.

Site conditions and standards of practice change. Therefore, we should be notified to update this report if construction is not performed within 24 months.

LIST OF PLATES

Plate 1	Site Vicinity Map
Plate 2	Test Pit Location Plan and Geotechnical Planning Map
Plate 3	Logs of Test Pits 1 through 8
Plate 4	Logs of Test Pits 9 through 14
Plate 5	Explanation of Test Pit Logs
Plate 6	Soil Classification System and Key to Test Data
Plate 7	Physical Properties for Rock Descriptions
Plates 8 through 10	Laboratory Test Results
Plates 11 and 12	Atterberg Limits Test Results
Plate 13	Typical Cross Section Hillside Grading
Plate 14	Typical Cross Section Compacted Fill Buttress
Plate 15	Typical Cross Section Underslab Subdrain
Plate 16	Typical Cross Section Foundation Subdrain

APPENDIX A

Report
Soil Corrosivity Evaluation and
Recommendations for Corrosion Control
Elnoka Village, Santa Rosa, CA
By JDH Corrosion Consultants, Inc.
Dated December 9, 2016

DISTRIBUTION

Copies submitted:	5	Oakmont Senior Living 9240 Old Redwood Highway, Suite 200 Windsor, CA 95492
	1	Brelje & Race Consulting Engineers 475 Aviation Boulevard, Suite 120 Santa Rosa, CA 95403

JM/JKR:nay/ra//Job No. 202.6.1

REFERENCES

Cartwright Aerial Surveys, Nos. 3088-131 and 132 [Aerial Photos], Scale: 1/24,000, April 21, 1971.

"City of Santa Rosa Citywide Creek Master Plan," City of Santa Rosa, Stormwater and Creeks Division, dated May 24, 2013.

"Geologic and Geophysical Framework of the Santa Rosa 7.5' Quadrangle, Sonoma County, California," Open-File Report 2008-1009, by R. J. McLaughlin et al., USGS, dated 2008.

"Geologic Map of the Kenwood 7.5' Quadrangle, Sonoma and Napa Counties, California: A Digital Database," by M. P. Delattre et al., California Geological Survey, dated 2007.

"Geology for Planning in Sonoma County," Special Report 120, by M. E. Huffman & C.F. Armstrong, California Division of Mines and Geology, dated 1980.

Geotechnical Investigation Report, "Santa Rosa Life Care Community, Santa Rosa, California," by Kleinfelder, Inc., dated November 8, 1993.

"Liquefaction Susceptibility Map," USGS, dated 2006, accessed via the Association of Bay Area Governments website (www.abag.ca.gov).

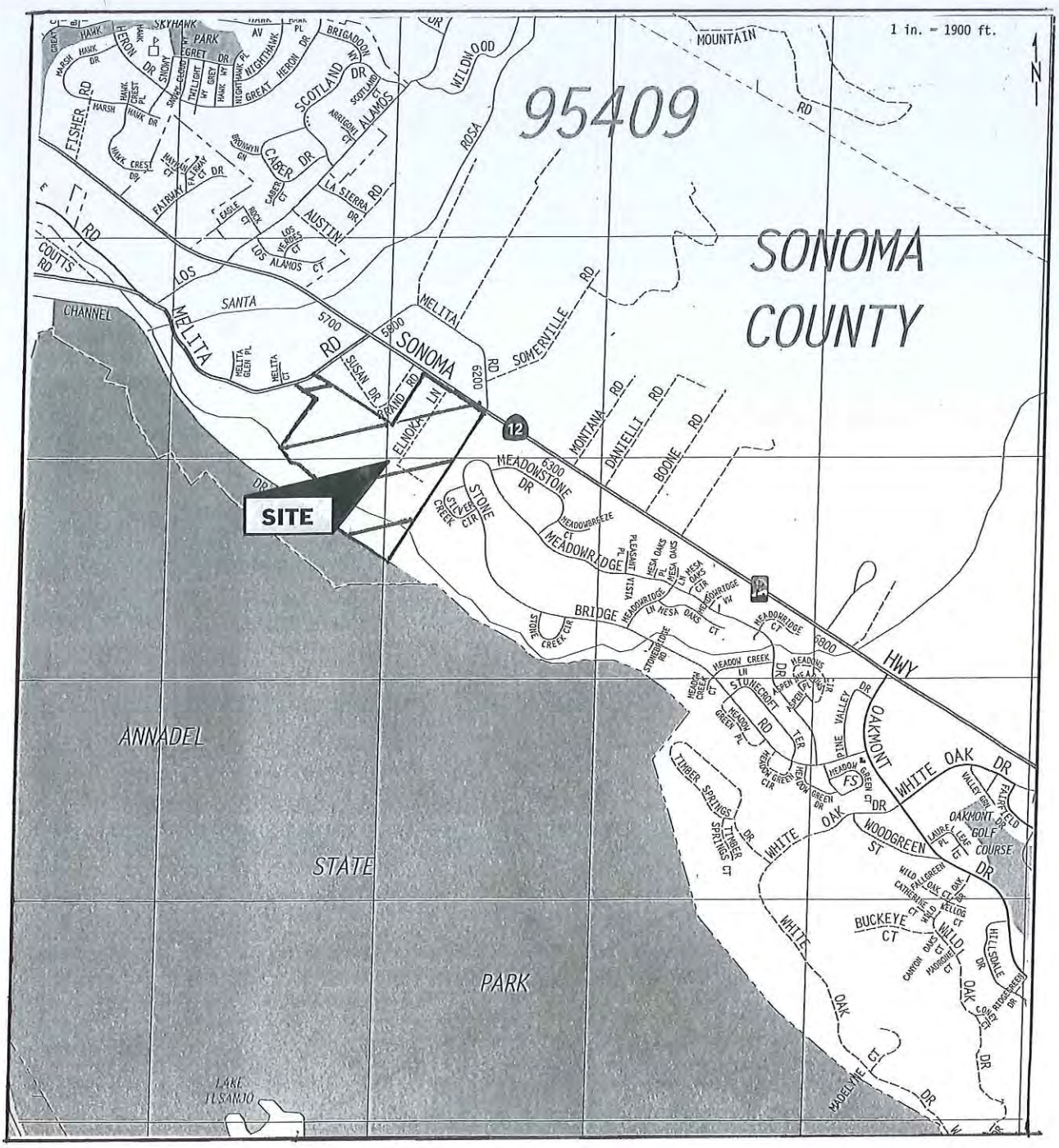
Report, "Preliminary Geotechnical Investigation, Proposed Life Care Community Project, Highway 12 and Elnoka Lane, Santa Rosa, California," by John H. Dailey, Consulting Geotechnical Engineer, dated April 2, 1991.

Report, "Soil Investigation, Elnoka Village, Santa Rosa, California," by Giblin Associates, Consulting Geotechnical Engineers, dated October 8, 2007.

Report, "Soil Investigation, Oakmont Subdivision Unit 15-D, Santa Rosa, California," by Giblin Associates, Consulting Geotechnical Engineers, dated June 23, 1988.

Report, "Summary of Earthwork Observation and Testing, Phase A & Phase 1-1996 Improvements, Pacific Lifecare/Three Bridges, Santa Rosa, California," by Kleinfelder, Inc., dated November 22, 1996.

The Santa Rosa Quadrangle Sheet of the Alquist-Priolo Special Studies Zone maps, California Division of Mines and Geology, dated 1983.



REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-06-17 Appr: Jm	**SITE VICINITY MAP** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **1**

SOIL DESCRIPTIONS

A = BROWN SANDY SILT (ML), medium stiff, dry, porous, with fine roots (Topsoil)

A_1 = dark gray in color, with strong organic odor

B = DARK BROWN SANDY CLAY (CH), very stiff, moist, plastic, with roots and root fibers

B_1 = with abundant subrounded gravel

C = MOTTLED YELLOW AND DARK BROWN CLAYEY SUBROUNDED GRAVEL (GC), medium dense, moist, with roots and cobbles (Residual Soil from sand and gravel conglomerate)

D = BEDROCK MATERIALS OF THE GLEN ELLEN FORMATION (QTge)

D_1 = MOTTLED DARK BROWN, ORANGE AND YELLOW SAND AND GRAVEL CONGLOMERATE, deeply weathered, low hardness, friable, weakly cemented

D_2 = YELLOW AND BROWN SANDSTONE, deeply weathered, low hardness to moderately hard, friable to moderately strong

D_3 = LIGHT BROWN, LIGHT GRAY AND BROWN SILTSTONE/MUDSTONE, deeply weathered, low hardness, friable

E = VERY DARK BROWN SANDY CLAY (CL), soft to medium stiff, dry to moist, porous, with abundant roots (Flood Deposits)

E_1 = dark red-brown in color

E_2 = with some orange and gray mottling, nonporous

F = DARK GRAY CLAYEY COARSE SAND (SC), loose, moist, porous, with abundant fine gravel (Flood Deposits)

G = GRAY-BROWN VERY SILTY FINE SAND (SM), medium dense, dry, porous (Flood Deposits)

H = GRAY VERY SANDY CLAY (CL), stiff, dry, slightly plastic (Residual Soil from sandstone/siltstone)

I = EXISTING FILL MATERIALS

I_1 = MOTTLED YELLOW AND BROWN SANDY COARSE GRAVEL (GP), very dense, dry, with rounded gravel up to 3 inches in size

I_2 = DARK BROWN SANDY CLAY (CL), medium stiff, moist, with gravel and some plastic clay (CH)

I_3 = DARK ORANGE-BROWN SANDY SILT (ML), stiff, dry (existing soil stockpile)

J = ORANGE-BROWN CLAYEY GRAVEL (GC), medium dense to dense, moist

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			TYPICAL NAMES	
COARSE GRAINED SOILS (MORE THAN HALF IS LARGER THAN No. 200 SIEVE)	GRAVEL — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN No. 4 SIEVE SIZE	CLEAN GRAVEL WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVEL, GRAVEL-SAND MIXTURE
			GP	POORLY GRADED GRAVEL, GRAVEL-SAND MIXTURE
		GRAVEL WITH OVER 12% FINES	GM	SILTY GRAVEL, GRAVEL-SAND-SILT MIXTURE
			GC	CLAYEY GRAVEL, GRAVEL-SAND-CLAY MIXTURE
	SAND — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN No. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 5% FINES	SW	WELL GRADED SAND, GRAVELLY SAND
			SP	POORLY GRADED SAND, GRAVELLY SAND
		SAND WITH OVER 12% FINES	SM	SILTY SAND, GRAVEL-SAND-SILT MIXTURE
			SC	CLAYEY SAND, GRAVEL-SAND-CLAY MIXTURE
FINE GRAINED SOILS (MORE THAN HALF IS SMALLER THAN No. 200 SIEVE)	SILT AND CLAY — LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILT, ROCK FLOUR, SANDY OR CLAYEY SILT WITH LOW PLASTICITY
			CL	INORGANIC CLAY OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAY (LEAN)
			OL	ORGANIC CLAY AND ORGANIC SILTY CLAY OF LOW PLASTICITY
	SILT AND CLAY — LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILT, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOIL, ELASTIC SILT
			CH	INORGANIC CLAY OF HIGH PLASTICITY, GRAVELLY, SANDY OR SILTY CLAY (FAT)
			OH	ORGANIC CLAY OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

KEY TO TEST DATA

						Shear Strength, psf	Confining Pressure, psf
EI	–	Expansion Index	TxUU	–	Unconsolidated Undrained Triaxial	320	(2600)
Consol	–	Consolidation	TxCU	–	Consolidated Undrained Triaxial	320	(2600)
LL	–	Liquid Limit (in %)	DSCD	–	Consolidated Drained Direct Shear	2750	(2000)
PL	–	Plastic Limit (in %)	FVS	–	Field Vane Shear	470	
PI	–	Plasticity Index	LVS	–	Laboratory Vane Shear	700	
SA	–	Sieve Analysis	UC	–	Unconfined Compression	2000	*
G_s	–	Specific Gravity	UC(P)	–	Laboratory Penetrometer	700	*
■		"Undisturbed" Sample					
□		Bulk Sample					

Notes: (1) All strength tests on 2.8" or 2.4" diameter samples unless otherwise indicated. * Compressive Strength

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 1-17-17 Appr:	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA ELNOKA VILLAGE SANTA ROSA, CALIFORNIA	PLATE **6**

A: CONSOLIDATION OF SEDIMENTARY ROCKS; usually determined from unweathered samples; largely dependent on cementation

1. U = unconsolidated
2. P = poorly consolidated
3. M = moderately consolidated
4. W = well consolidated

B: BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness (in feet)	Stratification
1. Massive	Greater than 4.0 ft	very thick bedded
2. Blocky	2.0 to 4.0 ft	thick bedded
3. Slabby	0.2 to 2.0 ft	thin bedded
4. Flaggy	0.05 to 0.2 ft	very thin bedded
5. Shaly or platy	0.01 to 0.05 ft	laminated
6. Papery	Less than 0.01 ft	thinly laminated

C: FRACTURING

Intensity	Size of Pieces (in feet)
1. Very little fractured	Greater than 4.0 ft
2. Occasionally fractured	1.0 to 4.0 ft
3. Moderately fractured	0.5 to 1.0 ft
4. Closely fractured	0.1 to 0.5 ft
5. Intensely fractured	0.05 to 0.1 ft
6. Crushed	Less than 0.05 ft

D: HARDNESS

1. **Soft** - Reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blow away.
4. **Hard** - can be scratched with difficulty; scratch produces little powder and is often faintly visible
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak

E: STRENGTH

1. **Plastic** - of very low strength.
2. **Friable** - Crumbles easily by rubbing with fingers.
3. **Weak** - An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

F: WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing

1. **Deep** - Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
2. **Moderate** - Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected; moderate to occasional intense discoloration; moderately coated fractures.
3. **Little** - No megascopic decomposition of minerals; little or no effect on normal cementation; slight, intermittent and/or localized discoloration; few stains on fracture surfaces.
4. **Fresh** - Unaffected by weathering agents. No disintegration or discoloration.

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEER	Job No: 202.6.1 Date: 01-16-17 Appr: Jm	**PHYSICAL PROPERTIES FOR ROCK DESCRIPTIONS** **ELNOKA VILLAGE SANTA ROSA, CALIFORNIA**	**PLATE 7**

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
1	0.5	M	4.9
	0.5	FS	30
	1.0	UC(P)	4500+
	2.0	M	16.3
	2.0	FS	90
	2.0	UC(P)	4500+
	2.5	M	9.6
	2.5	FS	70
	3.0	UC(P)	4500+
	4.0	UC(P)	4500+
2	0.5	M	14.1
	0.5	FS	50
	1.0	M	19.5
	1.0	FS	70
	1.0	UC(P)	4500+
	2.0	UC(P)	4500+
	3.0	UC(P)	4500+
	3.5	-200	66.3
3	0.5	UC(P)	4500+
	1.5	FS	40
	1.5	UC(P)	4500+
	2.5	UC(P)	4500+
4	1.0	M	13.8
	1.0	FS	50
	3.0	M	9.5
	3.0	-200	16.1
5	0.5	M	12.4
	0.5	FS	80

*Test Type

M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEER	Job No: 202.6.1 Date: 01-16-17 Appr: Jm	**LABORATORY TEST DATA** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **8**

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
6	0.0	M	5.5
	0.0	FS	30
	1.0	UC(P)	4500+
	1.5	M	17.4
	1.5	FS	90
	2.0	UC(P)	4500+
	3.0	UC(P)	4500+
7	1.0	FS	55
	1.0	UC(P)	4500+
	2.0	UC(P)	4500+
8	1.0	UC(P)	4500+
	1.5	M	7.5
	1.5	FS	20
	2.0	UC(P)	4500+
	3.0	UC(P)	4500+
	4.0	UC(P)	4500+
9	0.0	M	4.5
	0.0	FS	50
	1.0	UC(P)	4500+
	2.0	UC(P)	4500+
	3.0	UC(P)	4500+
10	1.0	M	15.4
	1.0	FS	60
	2.0	UC(P)	4500+
	4.0	FS	100
	4.0	UC(P)	3000
	6.0	UC(P)	4500+

*Test Type

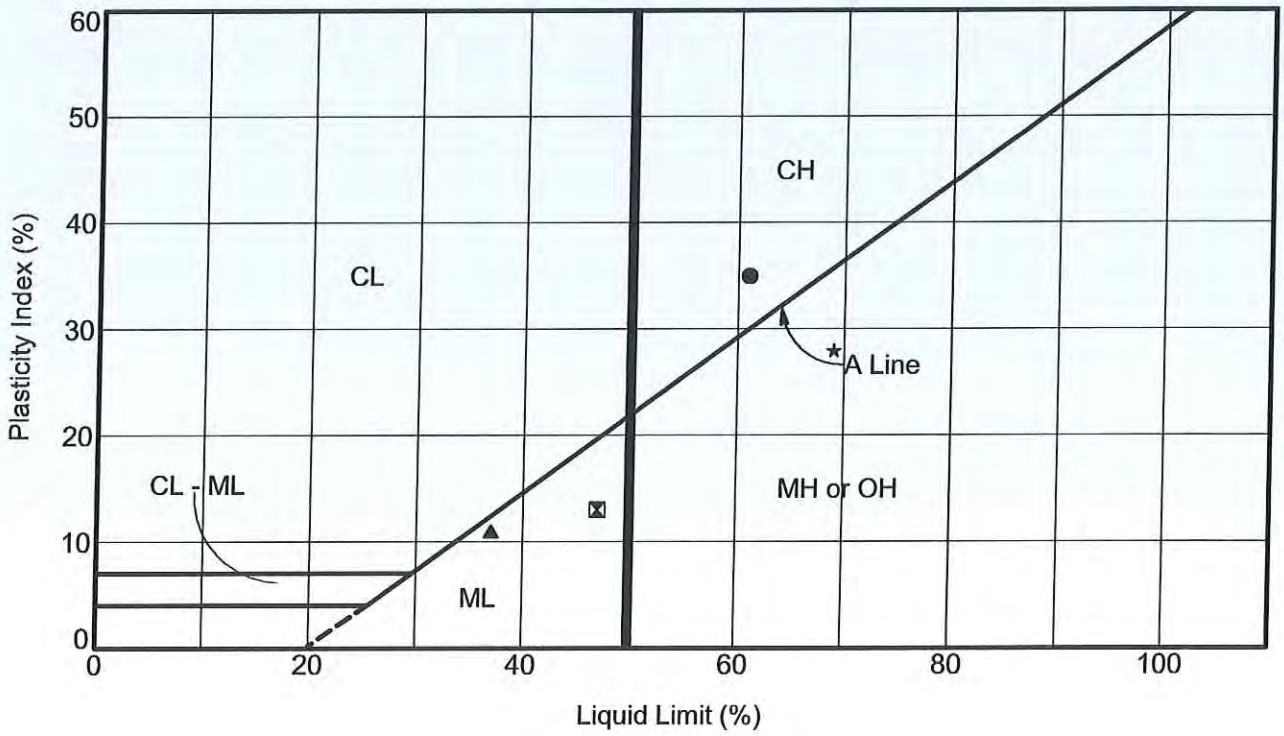
M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEER	Job No: 202.6.1 Date: 01-16-17 Appr:	**LABORATORY TEST DATA** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **9**

PIT NUMBER	DEPTH	TEST TYPE*	TEST RESULTS
11	0.5	M	18.2
	0.5	FS	55
	0.5	UC(P)	4500+
	1.5	UC(P)	4500+
	2.0	M	21.1
	2.0	FS	60
	3.5	UC(P)	4000
	4.5	UC(P)	3750
12	1.0	M	14.3
	1.0	FS	80
	1.0	UC(P)	4500+
	2.0	-200	38.8
	2.0	FS	80
	2.0	UC(P)	4500+
	3.0	UC(P)	4500+
	4.0	UC(P)	4000
	5.0	UC(P)	4000
13	0.5	M	13.6
	0.5	FS	55
	0.5	UC(P)	4500+
	1.5	M	15.1
	1.5	FS	65
	2.0	UC(P)	4500+
	3.5	UC(P)	4500+
14	2.0	M	13.3
	2.0	FS	50

*Test Type

M	Moisture Content (percent of dry weight)
MD	Moisture Content (percent of dry weight)/dry density (pounds per cubic foot)
UC(P)	Penetrometer - strength indicator (pounds per square foot)
UC	Unconfined Compression (pounds per square foot)
-200	Percent Passing No. 200 sieve by weight
FS	Percent Free Swell



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**DARK BROWN SANDY CLAY (CH)** **Test Pit 1 at 2.0 feet**	**61**	**26**	**35**	**90**
⊠	**DARK GRAY-BROWN SANDY SILT (ML)** **Test Pit 3 at 1.5 feet**	**47**	**34**	**13**	**40**
▲	**BROWN SANDY SILT (ML)** **Test Pit 6 at 0.0 feet**	**37**	**26**	**11**	**30**
★	**MOTTLED YELLOW AND BROWN SANDY COARSE GRAVEL (GM)** **Test Pit 7 at 1.0 feet**	**69**	**41**	**28**	**55**

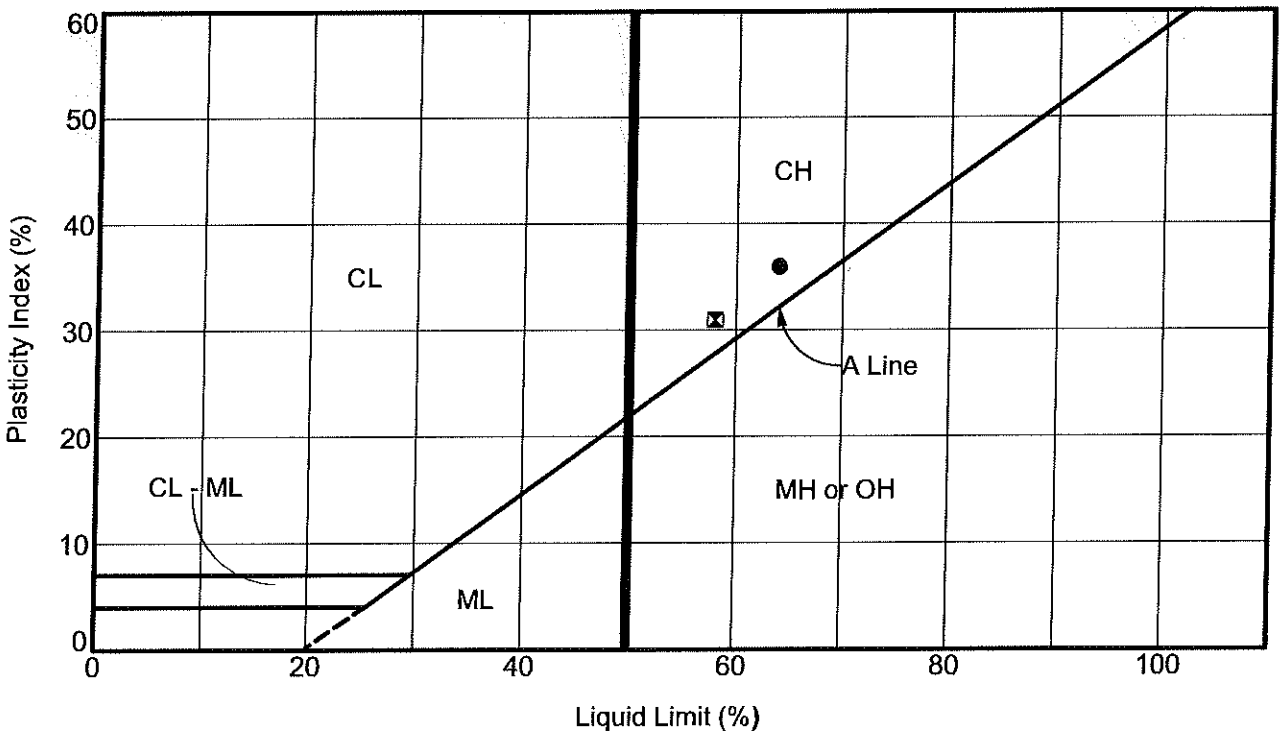
REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 202.6.1
Date: 1-17-17
Appr: JM

ATTERBERG LIMITS TEST RESULTS

ELNOKA VILLAGE
SANTA ROSA, CALIFORNIA

PLATE
11



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**DARK BROWN CLAY (CH)** **Test Pit 10 at 4.0 feet**	**64**	**28**	**36**	**100**
⊠	**DARK BROWN SANDY CLAY (CH)** **Test Pit 12 at 1.0 feet**	**58**	**27**	**31**	**80**

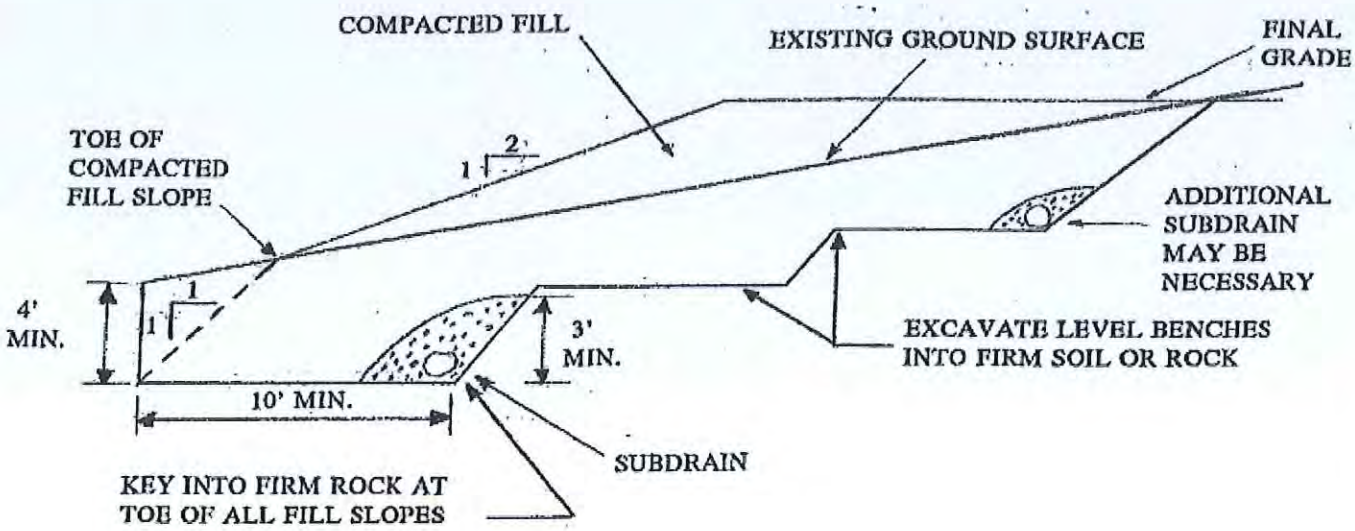
REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 202.6.1
Date: 1-17-17
Appr:

ATTERBERG LIMITS TEST RESULTS

ELNOKA VILLAGE
SANTA ROSA, CALIFORNIA

PLATE
12



(NOT TO SCALE)

Notes

1. Actual dimensions and extent of keyways, benches and subdrains will be determined in the field by the soil engineer.

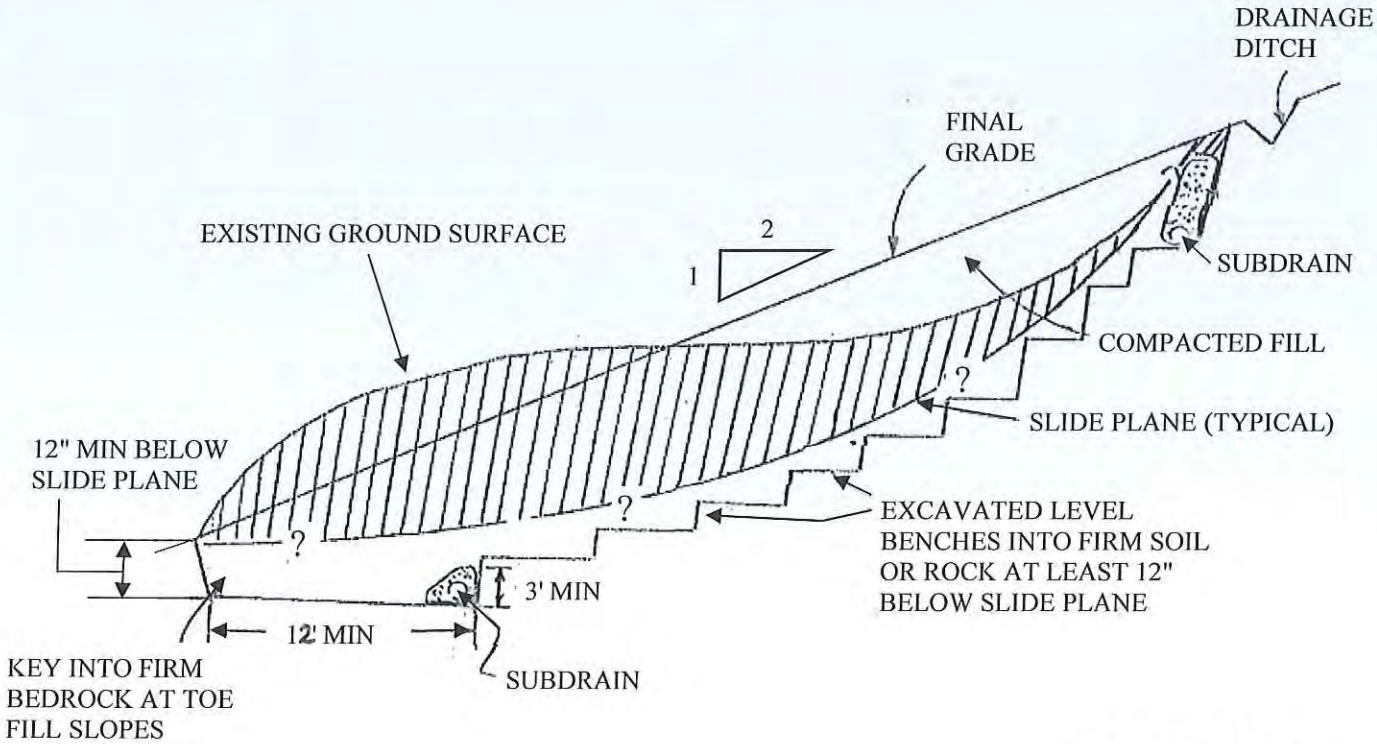
2. The upper 6 inches of soil exposed by excavation should be scarified, moisture conditioned and compacted to at least 90 percent relative compaction.

3. Fills should be placed in thin lifts and similarly compacted.

4. Slopes should be planted with deep-rooted vegetation (or protected by other suitable means) to reduce erosion.

5. Subdrains should consist of 4-inch diameter perforated, rigid plastic pipe (SDR-35 or equivalent) with gravity outlet and Class II Permeable Material, or any drainrock encased in a non-woven geotextile fabric weighing at least four ounces per square yard.

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-17-17 Appr: Jm	**TYPICAL CROSS SECTION** **HILLSIDE GRADING** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **13**



(NOT TO SCALE)

Notes

1. Actual dimensions and extent of keyways, benches and subdrains will be determined in the field by the soil engineer.

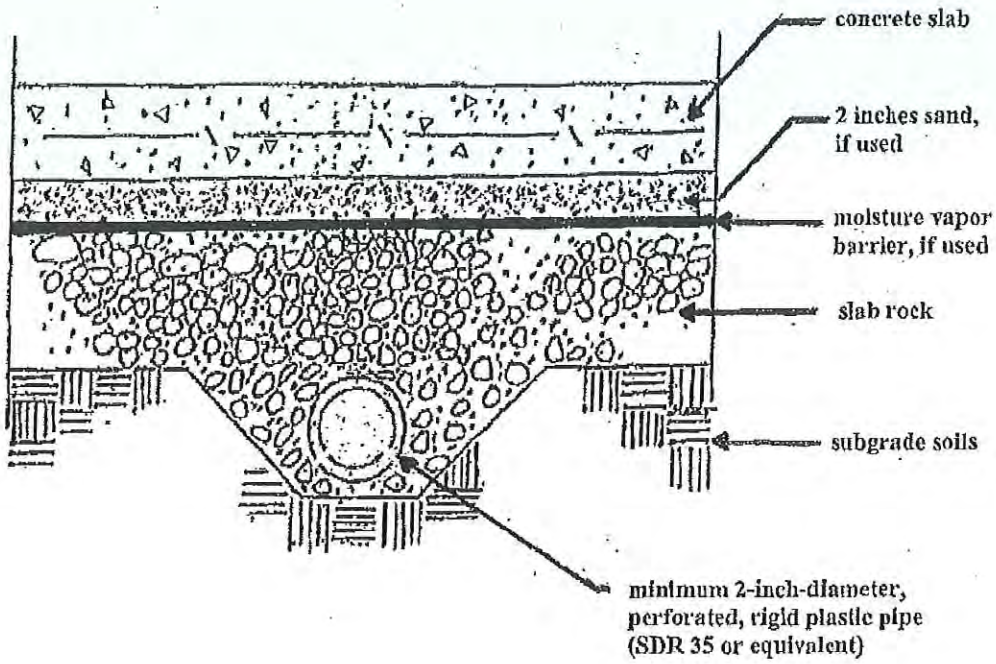
2. The upper 6 inches of soil exposed by excavation should be scarified, moisture conditioned and compacted to at least 90 percent relative compaction, as determined in the field by the soil engineer.

3. Fills should be placed in thin lifts and similarly compacted.

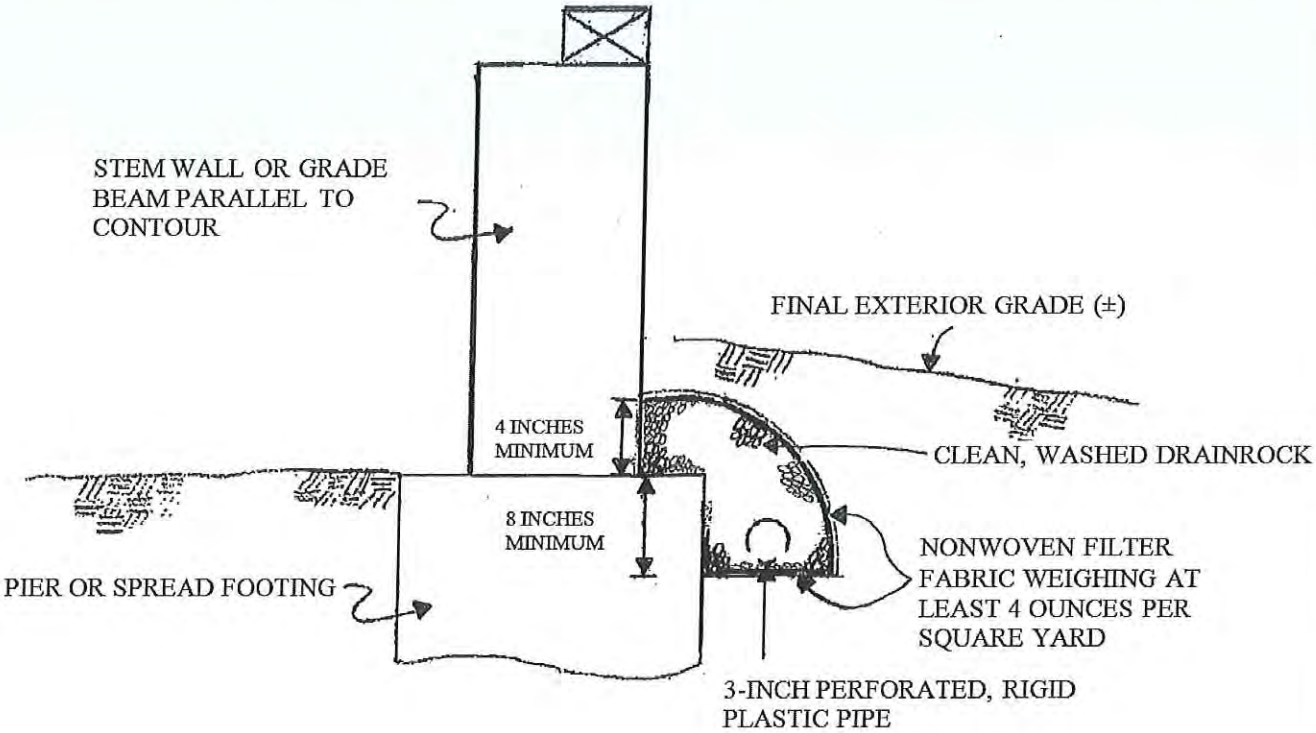
4. Slopes should be planted with deep-rooted vegetation (or protected by other suitable means) to reduce erosion.

5. Subdrains should consist of 4-inch perforated pipe with gravity outlet and Class II Permeable Material, or any drainrock encased in a non-woven geotextile fabric weighing at least four ounces per square yard.

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-16-17 Appr:	**TYPICAL CROSS SECTION** **COMPACTED FILL BUTTRESS** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **14**

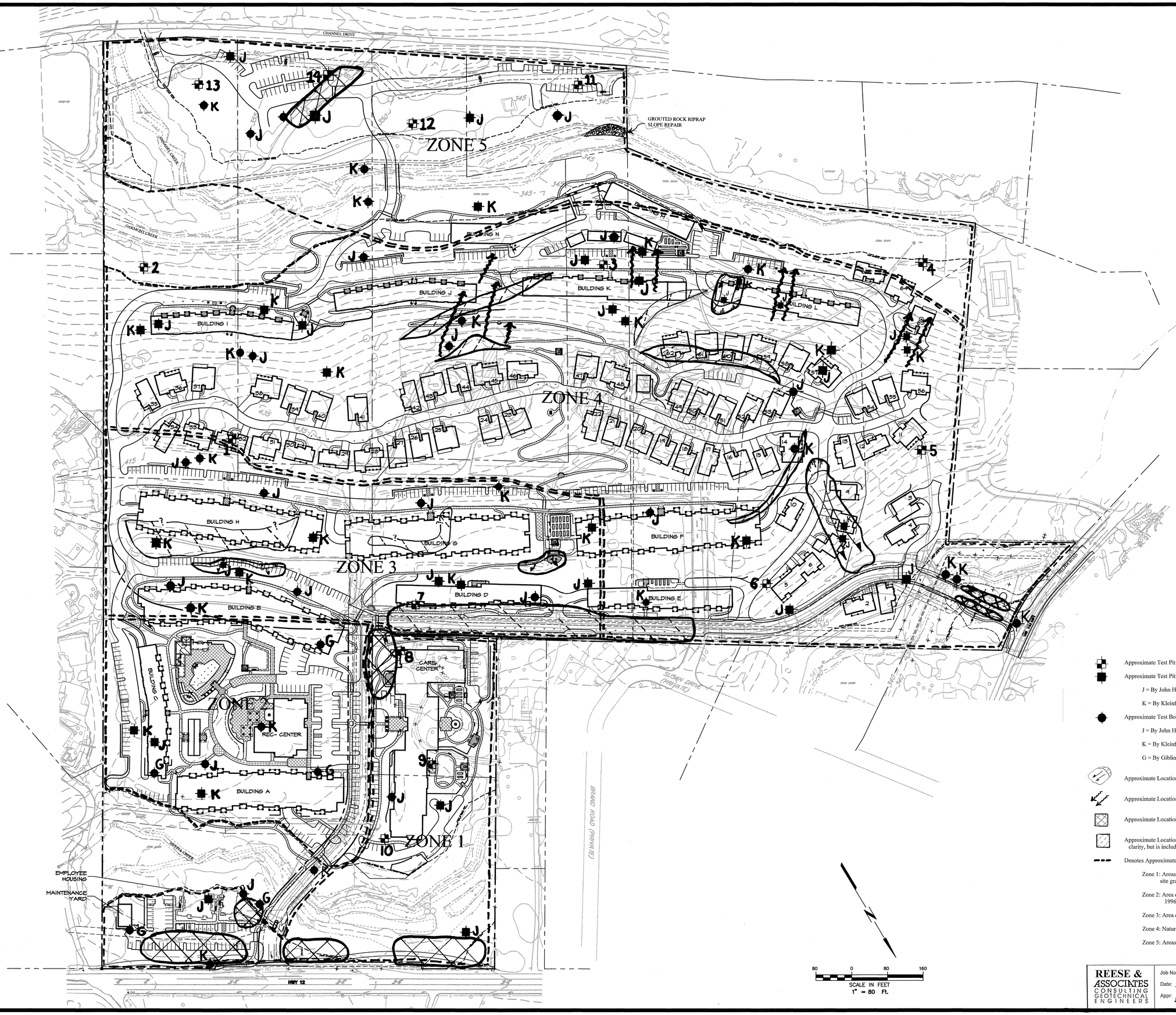


REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-17-17 Appr: gm	**TYPICAL CROSS SECTION** **UNDERSLAB SUBDRAIN** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **15**



REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-17-17 Appr: JM	**TYPICAL CROSS SECTION** **FOUNDATION SUBDRAIN** **ELNOKA VILLAGE** **SANTA ROSA, CALIFORNIA**	**PLATE** **16**

APPENDIX A



KEY

- Approximate Test Pit Location
- Approximate Test Pit Location by Others
  - J = By John H. Daily, Consulting Geotechnical Engineer (1991)
  - K = By Kleinfelder, Inc. (1993)
- Approximate Test Boring Location by Others
  - J = By John H. Daily, Consulting Geotechnical Engineer (1991)
  - K = By Kleinfelder, Inc. (1993)
  - G = By Giblin Associates (2007)
- Approximate Location of Landslide Areas
- Approximate Location of Areas of Heavy Creep
- Approximate Location of Soil Stockpiles
- Approximate Location of Undocumented Existing Fills (Zone 1 was left undashed for clarity, but is included)
- Denotes Approximate Limits of Soil Zones 1 through 5
  - Zone 1: Areas of existing fills that were not observed and tested during previous site grading operations.
  - Zone 2: Area of approved fill materials observed and tested by Kleinfelder, Inc. in 1996
  - Zone 3: Area of deep cuts and undocumented fills
  - Zone 4: Natural hillside terrain along ridgeline
  - Zone 5: Areas of deep weak soils

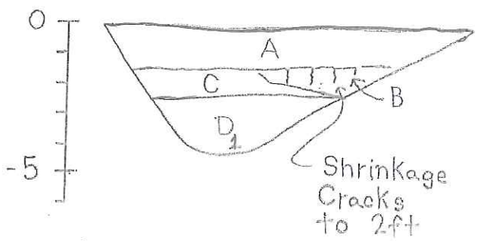
REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-17-17 Appr: JM	TEST PIT LOCATION PLAN AND GEOTECHNICAL PLANNING MAP ELNOKA VILLAGE SANTA ROSA, CALIFORNIA	PLATE 2

12-15-16 bartholow \2854\DWG\2854 01\2854-04-BASE.dwg TAB: 80acsle

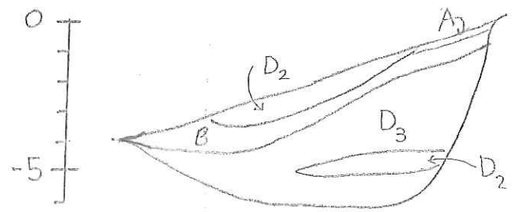
Scale: 1 in = 5 ft
Horizontal and Vertical

0 — 5

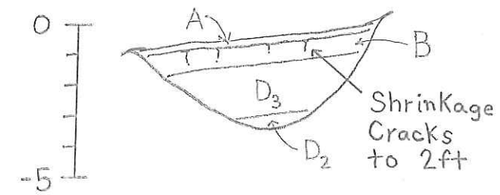
LOG OF TEST PIT 1



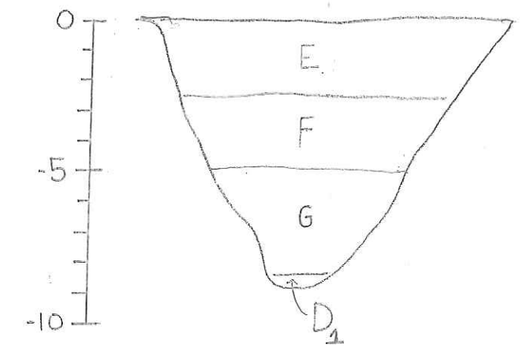
LOG OF TEST PIT 2



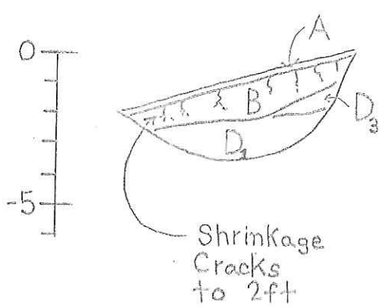
LOG OF TEST PIT 3



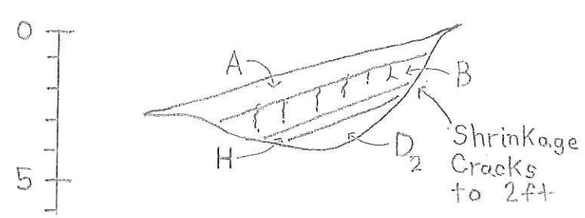
LOG OF TEST PIT 4



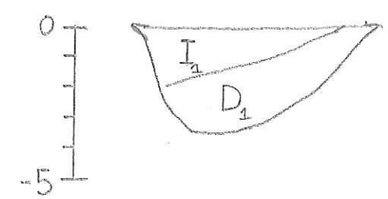
LOG OF TEST PIT 5



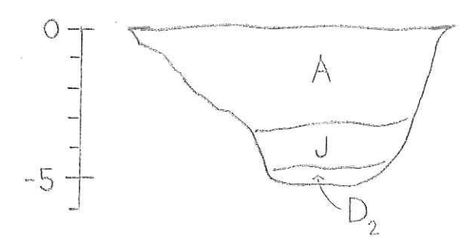
LOG OF TEST PIT 6



LOG OF TEST PIT 7



LOG OF TEST PIT 8



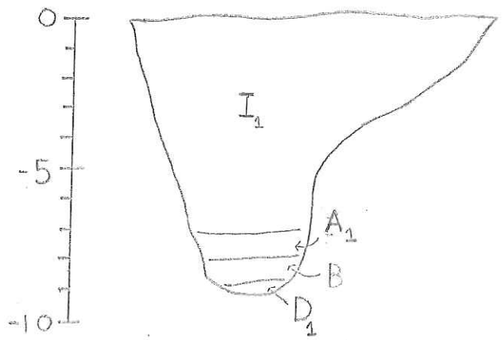
*See Plate 5 for Soil Descriptions

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-06-17 Appr: Jm	**LOG OF TEST PITS 1 THROUGH 8** **ELNOKA VILLAGE** **SANTA ROSA, CALIFONIA**	**PLATE** **3**

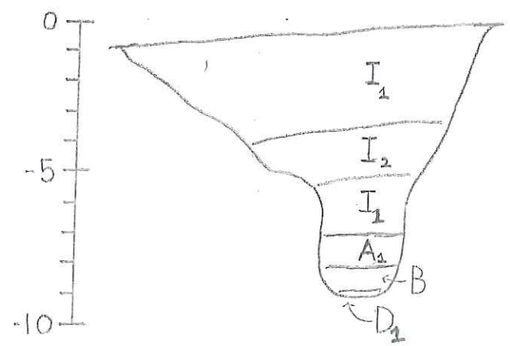
Scale: 1 in = 5 ft
Horizontal and Vertical

0 — 5

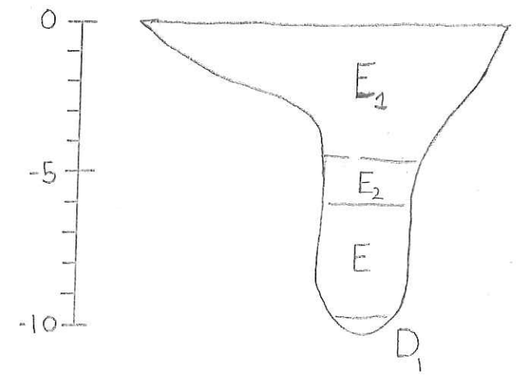
LOG OF TEST PIT 9



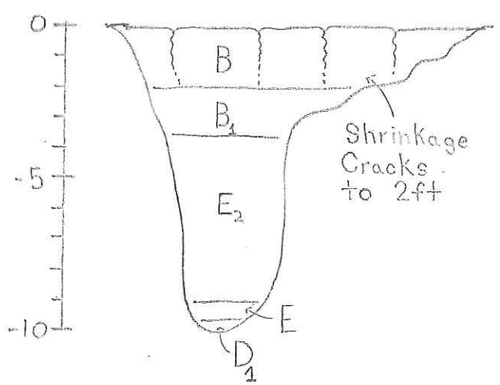
LOG OF TEST PIT 10



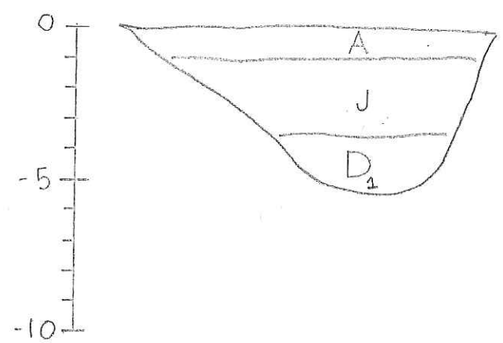
LOG OF TEST PIT 11



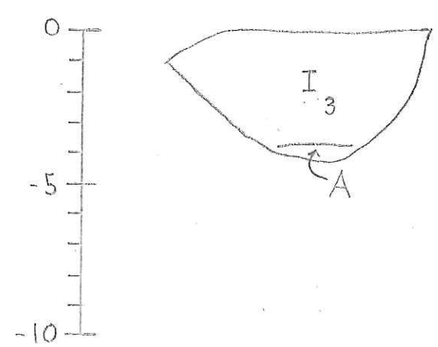
LOG OF TEST PIT 12



LOG OF TEST PIT 13



LOG OF TEST PIT 14



*See Plate 5 for Soil Descriptions

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 202.6.1 Date: 01-06-17 Appr: Jm	**LOG OF TEST PITS 9 THROUGH 14** **ELNOKA VILLAGE** **SANTA ROSA, CALIFONIA**	**PLATE** **4**

F.3 - Paleontological Report

THIS PAGE INTENTIONALLY LEFT BLANK

Kenneth L. Finger, Ph.D.
Consulting Paleontologist

18208 Judy St., Castro Valley, CA 94546-2306 510.305.1080 klfpaleo@comcast.net

June 26, 2017

Liza Baskir
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

Re: Paleontological Records Search for the Einoka CCRC Project (2498.0008), Sonoma County

Dear Ms. Bazkir:

As per your request, I have performed a records search on the University of California Museum of Paleontology (UCMP) database for the Einoka CCRC Project that is to the southeast of Melita and bordered by Annabel State Park to the southwest and the Sonoma Hwy 12 to the northeast. The project site location is in the Santa Rosa and Kenwood quadrangles (1980 USGS 7.5-series topographic maps). Google Earth imagery suggests that the surface of the site probably has been disturbed by agricultural use; in addition, there are a few structures with adjacent landscaping.

Geologic Units

Key to Mapped Units

Qha	Holocene alluvium
Qhc	Holocene colluvium
Qhpf	Holocene alluvial fan deposits
Qf	Holocene fan deposits
QTge	Glen Ellen Fm (Pliocene to Pleistocene)
QTu	Unnamed fluvial deposits (Pliocene to Pleistocene)
Tgp?	Fluvial & lacustrine deposits of Humbug Creek (Pliocene)
Tsb	Andesite, basaltic andesite & basalt
Tsbb	Basaltic andesite flows & flow breccias distinguished by phenocrysts of olivine & augite
Tsrb	Rhyolite of Bennett Mountain
Tsbt	Andesitic to dacitic tuff, breccia, & minor flows
Tsvt	Pumiceous ash-flow & minor air-fall tuff
Tsvm	Mafic flows, undivided

According to the conjoined parts of the geologic maps for the Santa Rosa quadrangle (McLaughlin et al., 2008) and the Kenwood quadrangle (Delattre et al. 2007) shown here, four geologic units are within the project site (outlined in red): Holocene alluvium (Qf), Holocene alluvial fan deposits (Qhpf), undifferentiated Pliocene to Pleistocene fluvial deposits (QTu), and questionably Pliocene fluvial & lacustrine deposits (Tgp?). Eight of the other nine units shown on this map are within the surrounding half-mile search area (dashed line) and six of them are volcanic. Only the two onsite pre-Holocene sedimentary units (QTu and Tgp?) are of paleontological concern; the Glen Ellen Formation, which is mapped nearly a half-mile from the site, is the only other another unit with paleontological potential here, and it may occur in the subsurface of the site.

UCMP Records Search

The UCMP database lists 6 vertebrate and 2 plant fossil localities in the Santa Rosa quadrangle, and 1 vertebrate and 0 plant fossil localities in the Kenwood quadrangle. Three of the vertebrate localities are recorded as being in the Petaluma Formation, one locality is questionably ascribed to the Glen Ellen Formation, and the other three are unassigned, but all are in the Pliocene–Pleistocene interval. In the Santa Rosa and Kenwood quadrangles, the Pleistocene is represented by 3 vertebrate specimens in the UCMP collection: *Glossotherium* cf. *G. robustus* (robust ground sloth), *Bison antiquus* (ancient bison), and *Bison* sp. that indicate a terrestrial environment of deposition. Other Pleistocene localities in Sonoma County have also yielded *Clemmys* (western pond turtle), *Glossotherium harlani* (Harlan's ground sloth), and *Mammut americanum* (American mastodon). The only Pleistocene plant locality recorded from Sonoma County is in a marine terrace along the Pacific coast.

Five Pliocene vertebrate localities are also recorded in the two quadrangles. Two localities in the Petaluma Formation yielded *Equus* (horse). The other three localities are in the Merced Formation and are represented by 49 specimens including eagle ray (*Myliobatus*), dolphins (*Delphinus*, cf. *Stenella*, and cf. *Stenodelphis sternbergi*), porpoise (*Pontoporia sternbergi*), baleen whale (*Balaemula*), northern fur seal (*Caliorhinus ursinus*), and extinct fur seal (cf. *Thalassoleon mexicanus*). There also are 13 Pliocene plant localities in the county, but 9 are in the Sonoma Volcanics and 2 are in the Merced Formation. Neither of these units are mapped in the vicinity of the Einoka CCRC project site, but there are two localities east of Petaluma (no specimens entered into database) that are in an unnamed unit that could be equivalent to those at the project site.

In summary, the recovery of 12 Pleistocene vertebrates from 10 localities in unnamed units in Sonoma County indicates that Einoka CCRC project excavations could impact significant paleontological resources, but the potential is low.

Remarks and Recommendations

About 60% of the project site is mapped as unnamed Pliocene–Pleistocene deposits of low paleontological potential, and these or adjacent deposits of similar age probably underlie the Holocene deposits mapped on the other 40% of the surface. Occurrences of terrestrial fossils tend to be unpredictable and spottily distributed, so most Pleistocene alluvial and lacustrine sediments are have a low potential but high sensitivity for significant paleontological resources. A preconstruction paleontological survey of the Einoka CCRC project site is not recommended because the site's entire surface appears to be disturbed. It would be prudent to have paleontological

monitoring of construction-related excavations because paleontologically sensitive units are on the surface and in the subsurface of the site. At the very least, a professional paleontologist should periodically inspect the excavations. Although unlikely, should any vertebrate elements (e.g., teeth, bones) or plants (e.g., leaves, tree trunks) be found, all construction activities are to be diverted away from the find until the monitor has assessed the find and, if deemed appropriate, salvaged it in a timely manner. Salvaged fossils should be deposited in an appropriate repository, such as the UCMP, where they will be properly curated and made available for future research.

If I can be of further assistance on this project, please don't hesitate to contact me.

Sincerely,

References Cited

Delattre, M.P., Wagner, D.L., Higgins, C.T., Witter, R.C., and Sowers, J.M., 2007, Geologic map of the Kenwood 7.5' quadrangle, Sonoma and Napa counties, California: A digital database by Gutierrez, C.I., and Toman-Sager, K., version 1.0. California Geological Survey.

McLaughlin, R.J., Langenheim, V.E., Sarna-Wojcicki, A.M., Fleck, R.J., McPhee, D.K., Roberts, C.W., McCabe, C.A., and Wan, E., 2008, Geologic and geophysical framework of the Santa Rosa 7.5' quadrangle, Sonoma County, California. U.S. Geological Survey Open-File Report 2008-1009 .

THIS PAGE INTENTIONALLY LEFT BLANK