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Report
Soil Investigation
Pink Viking Subdivision
8841 Old Redwood Highway
Cotati, California

Prepared for
Pink Viking, LLC
435 E Street
Santa Rosa, CA 95404
Attention: Keith Christopherson

By

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INTRODUCTION

This report presents the results of our soil investigation for the proposed Pink Viking Subdivision in Cotati, California. The site address is 8841 Old Redwood Highway. We understand that the proposed development will consist of subdividing the property into 21 lots for single-family, residential construction. The lots would be served by a new asphalt-paved roadway and underground utilities. Site grading is expected to include cuts and fills on the order of about 3 to 8 feet to create level building pads. Retaining walls are proposed between some of the lots. From our conversations with the developer, we understand the use of post-tensioned concrete slabs-on-grade are desired for foundation support. Accordingly, the balance of this report is oriented towards the use of post-tensioned slabs for foundation support.

The object of our investigation, as outlined in our proposal dated May 19, 2014, was to review selected published geologic information in our files, explore subsurface conditions, measure depth to groundwater, if encountered, and determine physical properties of the soils sampled. We then performed engineering analyses to develop conclusions and recommendations concerning:

1. Proximity of the site to active faults.
2. Site preparation and grading.
3. Foundation support and design criteria.
4. Support of concrete slab-on-grade floors.
5. Retaining wall design criteria.

6. The results of two laboratory stabilometer (R-value) tests for pavement thickness determination by others.

7. Quality and compaction criteria for development of asphalt-paved roadways.

8. Soil engineering drainage.

9. Supplemental soil engineering services.

WORK PERFORMED

We reviewed selected published geologic information in our files including:

1. The Cotati Quadrangle Sheet of the Alquist-Priolo Earthquake Fault Zone maps, California Division of Mines and Geology, dated 1983.

2. The Fault Activity Map of California, by C.W. Jennings and W.A. Bryant, California Geological Survey, dated 2010.

3. Flood Insurance Rate Map, Sonoma County, California, Map No. 06097C0551E, Federal Emergency Management Agency (FEMA), dated December 2, 2008.

4. The "Geologic Map of the Santa Rosa Quadrangle, California," by D. L. Wagner and E. J. Bortugno, California Division of Mines and Geology, dated 1982.

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

6. Liquefaction Susceptibility Map, Association of Bay Area Governments website (www.abag.ca.gov), dated March 2007.

On August 7, 2014, we observed surface features and explored subsurface conditions to the extent of five test borings at the approximate locations indicated on Plate 1. The borings were drilled to depths of about 10 to 14 feet with track-mounted auger equipment. Our field engineer located the borings, observed the drilling, logged the conditions encountered and obtained samples for visual classification and laboratory testing. Relatively undisturbed samples were obtained with a 2.5-inch (inside-diameter), split-spoon sampler driven with a 140-pound drop hammer. A 2.0-inch (outside-diameter), Standard Penetration Test (SPT), split-spoon sampler was used at selected depths where granular materials were encountered. The stroke during driving was about 30 inches. The blows required to drive the samplers were recorded and converted to equivalent SPT blow counts for correlation with empirical data. Logs of the borings showing soil classifications, sample depths and SPT blow counts are presented on Plates 2 through 6. The soils are classified in accordance with the Unified Soil Classification System explained on Plate 7.

Selected samples were tested in our laboratory to determine moisture content, dry density, classification (percent free swell and Atterberg Limits) and strength characteristics. The test results are shown on the logs, with the strength data shown in the manner described by the Key to Test Data on Plate 7. Detailed results of the Atterberg Limits tests are presented on Plate 8. The results of two R-value tests are presented on Plates 9 and 10.

The boring locations shown on Plate 1 were determined by visually estimating from existing surface features. The locations should be considered no more accurate than implied by the methods used to establish the data. At completion of the exploration, the boreholes were backfilled with the soil cuttings.

SURFACE AND SUBSURFACE CONDITIONS

The approximately 7-acre project site is located about 1½ miles south of downtown Cotati. The site is bordered by Old Redwood Highway on the east, the Hunter's Ridge/Lasker Knolls subdivision on the north and rural residential properties on the west and south. In general, the site slopes upward to the west from Old Redwood Highway and varies in inclination from about 25 horizontal to 1 vertical (25:1) in the eastern portion to about 10:1 in the western portion. At the time of our exploration, the ground surface was covered with tall grass and weeds, bushes and scattered trees. A line of large eucalyptus trees is located at the western property line. A dense growth of berry bushes is located along the north property line in the central portion of the site. Single-family residences with several outbuildings are located on the eastern portion of the site and abandoned wooden chicken coops and sheds in various stages of disrepair are located throughout the west portion of the property. Surface topography suggest some minor fills on site, including aggregate baserock driveways that serve the on-site residences.

The borings and laboratory tests indicate that the site is underlain by silty and sandy topsoil underlain by clayey and sandy soils and highly weathered rock materials. Fine silty sand topsoil was observed in about the upper 2 feet in Test Borings 1 and 2. Sandy silt topsoil was observed to depths of about 1½ to 3½ feet in the remaining borings. The topsoils were observed to be porous from prior decomposition of organic materials in the upper about 1 to 2 feet. Laboratory tests indicate that the topsoils are low in expansion potential. That is, the materials would tend to undergo low strength and volume changes with seasonal variations in moisture content.

Low expansive, dense to very dense silty sands were observed underlying the topsoils in Test Borings 1, 4 and 5. The silty sands extended to depths of about 4 to 5½ feet below the existing ground surface. Highly expansive, plastic clays were observed underlying the silty sands in Test Borings 1 and 4 and the topsoils in Test Borings 2 and 3. All of the test borings bottomed into deeply weathered rock materials of the Petaluma Formation. The rock materials consisted of low hardness, friable siltstone, claystone and sandstone. In general, the total thickness of soil overlying bedrock materials ranged from about 5½ to 12 feet below the existing ground surface.

CONCLUSIONS

Based on the results of our field exploration, laboratory testing and engineering analyses, we conclude that, from a soil engineering standpoint, the site can be used for the proposed residential construction. The most significant soil engineering factors that must be considered in design and construction are the presence of existing fills, weak, porous, upper natural topsoils and underlying highly expansive soils.

Existing fill materials, if not properly placed and compacted, could be subject to significant amounts of total and/or differential settlement. Also, our experience indicates that weak, porous upper soils, such as the topsoils encountered at the site, can undergo considerable strength loss and settlement when loaded in a saturated condition. Where evaporation is inhibited by footings, slabs, or fills, eventual saturation of the underlying soils can occur. Therefore, we conclude that existing fills and upper porous soils are not suitable for foundation support in their present condition. To reduce the risk of total and/or differential settlements, we judge that it will be necessary to remove (overexcavate) the existing fills for their full depth and a portion of the weak upper soils so as to provide a pad of properly compacted fill beneath foundation elements. The compacted fill pad would need to be sufficiently thick so as to help redistribute foundation loads and provide more uniform supporting conditions. Specific recommendations for the depth and extent of the compacted fill pad beneath the building foundations are discussed in subsequent sections of this report.

Expansive soils can shrink and swell with seasonal changes in moisture content and can heave lightly loaded footings and slabs. Future shrink and swell of expansive soils can be substantially reduced by controlling soil moisture content. We have observed that significant seasonal changes in moisture content generally occur in the upper 2 to 3 feet. However, depending on factors such as seasonal rainfall totals, summer weather conditions and surface treatments, significant moisture variations in the soils can occur to substantially deeper depths. The risk of future distress to structures resulting from shrinking and swelling of the expansive clays can be considerably reduced by initially moisture conditioning the soils to cause preswelling and then covering the soils with a moisture confining and protecting blanket of nonexpansive fill or utilizing foundation systems designed to tolerate expansive soil movements.

We understand that it is desired to use post-tensioned slab-on-grade foundation systems with perimeter and interior stiffening beams. We conclude that post-tensioned slab-on-grade foundations could be used for support of the residences as proposed. We anticipate that the graded pads for the residences would likely include cuts exposing firm bedrock material and/or highly expansive soils on the upslope side transitioning to planned fill placed over compressible natural soils along the downslope side. Because of the potential for such differential supporting conditions to be encountered at pad grade level, we judge that post-tensioned slab-on-grade foundations would need to be designed to tolerate both expansive and compressible soil movements. To help provide more uniform supporting conditions and reduce potential foundation heave and/or settlement, we judge that it will be necessary to verify that expansive soils have not dried and cracked before being covered with fill and to underlie post-tensioned slab-on-grade areas with a pad of properly compacted fill, as subsequently recommended.

We judge that exterior concrete flat work can be supported directly on the weak upper and/or expansive clay soils, provided the slabs are allowed to float, and some minor heave and/or settlement and resultant distress are acceptable.

For foundations designed and installed in accordance with our subsequent recommendations, we judge that settlements will be small, less than about 1-inch. Post-construction settlements should be about one-half this amount.

SEISMIC DESIGN PARAMETERS

The geologic maps reviewed did not indicate the presence of active faults at the site and the property 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 Tolay fault is the nearest recognized fault located approximately 1 mile southwest of the project area. However, this fault is considered to have only experienced displacement in the past 1.6 million years. In order to be considered an active fault within the State Seismic Hazards mapping project, a fault would have to have shown evidence of displacement in the last 11,000 years. However, some geologists consider that the southern section of the Tolay fault has shown evidence of displacement in the last 11,000 years. Although the Tolay fault zone is not included in the list of active faults, even inactive faults may be capable of some minor displacement in the event of a large earthquake occurring on nearby active faults. The closest faults generally considered active are the Rodgers Creek fault zone located approximately 4½ miles to the northeast, the San Andreas fault zone located approximately 15 miles to the southwest and the West Napa fault zone located approximately 20 miles to the east.

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 2013 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. Based on our review of available geologic maps and our knowledge of the subsurface conditions, we judge that the site can be classified as Site Class C, as described in Chapter 20 of American Society of Civil Engineers (ASCE) publication ASCE 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.

2013 CBC Ground Motion Parameters

Site Class	C
Mapped Spectral Response Accelerations:	
S_S	1.519 g
S_1	0.600 g
Design Spectral Response Accelerations:	
S_{DS}	1.012 g
S_{D1}	0.520 g

RECOMMENDATIONS

Site Grading

The areas to be developed should be cleared of debris, brush and other obstructions, where encountered. Debris resulting from demolition of the existing structures, including foundations and associated subsurface utilities, should be removed from the site. Designated trees, their root systems and dense growths of grass and vegetation should be removed within planned improvement areas. The resultant voids should be backfilled with compacted soil as subsequently described.

Wells, septic tanks, leach fields and other voids encountered or created should be removed, filled with compacted soil or granular material or capped with concrete, as determined by the appropriate regulatory agency or the soil engineer.

Areas to be graded then should be stripped of the upper few inches of soil containing root growth and organic matter. We anticipate that the depth of stripping needed would 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 structures, walkways and paved areas.

After stripping, excavations can be performed as necessary. Existing fills encountered within building or improvement areas should be removed for their full depth. Within planned building areas and extending to at least 5 and 3 feet beyond the building perimeter and adjacent concrete walkways, respectively, excavations should be performed so as to provide space for at least 12 inches of properly compacted fill below planned finished pad grade elevation. We anticipate that, with the exception of organic matter and rocks or hard fragments larger than 4 inches in diameter, the on-site excavated materials will be suitable for reuse as compacted fill.

The surfaces exposed by stripping and/or excavation should be scarified at least 6 inches deep, moisture conditioned to near optimum (at least 4 percentage points above optimum for on-site clayey soils and so as to close any shrinkage cracks for their full depth) and compacted to at least 87 percent relative compaction.[1] Approved on-site and/or imported fill should be placed in layers, similarly moisture conditioned and compacted to at least 90 percent. Exposed expansive soils should be fully moisture conditioned as discussed above. The building pads should be maintained in a moist condition (at least 4 percent above optimum for on-site clayey soils) through the completion of the designed improvements. As an alternative, building pad and/or other planned improvement areas could be remoisture conditioned just prior to construction. This may require several cycles of watering over a period of days or weeks, depending on the time of year and construction scheduling. Therefore, if foundation construction is not completed within 1 month after the rough grading, we should be notified to evaluate the need for further moisture conditioning.

For grading performed in the driest time of the year, especially after winters of significantly less than normal total or springtime rainfall, shrinkage cracks in the expansive soils may be deep. Prolonged watering or controlled flooding with the possible use of wetting agents may be necessary to moisture condition the expansive soils to the high initial moisture content

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 ASTM D 1557 laboratory compaction test procedure. Optimum moisture content refers to the moisture content at maximum dry density.

needed to close shrinkage cracks for their full depth. As a construction expediency, the grading contractor could elect to overexcavate a portion of the expansive soils to reduce the amount of moisture conditioning time needed. The overexcavated soils then could be moisture conditioned and replaced as properly compacted fill.

For grading performed in the rainy season (late fall, winter, early spring), the soils may become fully expanded naturally and not require increased moisture conditioning. However, with winter grading there are risks that include: 1) the site becoming too wet and soft to support construction equipment; 2) normally suitable imported fill becoming too wet to compact (requiring more expensive rocky fill); 3) excavation bottoms becoming unstable, requiring overexcavation and/or use of geotextile fabrics or placement of granular working pads; and 4) procedures being required to eliminate the possibility of tracking mud onto adjacent public streets. Accordingly, we suggest that the contract documents contain provisions to account for such possible additional costs.

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

Finished cut and fill 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 reduce erosion.

Foundation Support

Provided the site is graded as outlined above, post-tensioned slabs should be underlain by at least 12 inches of properly compacted fill. We recommend that the post-tensioned slabs be designed in accordance with the Post-Tensioning Institute's Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, current edition, and the criteria in the latest adopted edition of the CBC. Uniform thickness post-tensioned slabs should be at least 12 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. For slabs positioned in expansive soil areas, we recommend the following parameters be used for design:

Center Lift	Edge Lift
E_m = 8.0 feet	E_m = 5.25 feet
Y_m = 2 inches	Y_m = 1 inch

As previously discussed, we anticipate that slab foundations for some of the residences would likely be underlain by both expansive soils on the upslope side and fills placed over compressible natural soils along the downslope side. Accordingly, we recommend the slab design be checked using the following criteria for a compressible soil condition.

Anticipated differential settlement between the center and downslope edge of slabs = 1½ inch

For design, an allowable bearing value of 1,000 pounds per square foot (psf) and an effective friction factor of 0.30 can be used.

Prior to placing the reinforcing or slab rock, the subgrade soils should be thoroughly moistened and be smooth, firm and uniform. Slab subgrade should not be allowed to dry prior to concrete placement.

From our experience, we have observed that where slopes, retaining walls or trees are located near foundations at expansive soil sites, an increased risk of foundation distress due to 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 soil movements (shrink/swell), we recommend that a 36 inch deep thickened edge be provided at the slab perimeter where:

1) the distance between building foundations and retaining walls or face of slopes is less than 8 feet;

2) the distance from buildings to trees is less than one-half the trees maximum mature height.

The actual need for and location of the thickened edges should be determined during final design.

Moisture vapor will condense on the underside of slabs. Where migration of moisture vapor through slabs is detrimental, a 10-mil moisture vapor retarder conforming to ASTM E1745 Class C 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 actual use of sand should be determined by the architect or design engineer. The use of a less permeable and stronger membrane should be considered if sand is not to be 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. In addition, where sand is used, the thickened slab edge should penetrate through the sand (and any slab rock) and bottom in compacted pad soils to act as a moisture barrier.

Retaining Walls

Retaining walls that are free to rotate slightly and support level (and up to 3:1 slope) backfill should be designed to resist an active equivalent fluid pressure of 40 pounds per cubic foot (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.

Because of the presence of a weak, compressible and highly expansive upper soils, we recommend that foundation support for retaining walls be obtained from drilled piers that penetrate well into firm natural soils or bedrock. Foundation piers should be at least 12 inches in diameter. The foundation piers should extend at least 5 feet into firm underlying material, as determined in the field by the soil engineer. Specific pier depths should be determined during final design when wall location and heights are determined.

Vertical loads on the piers can be carried below the upper 2 feet in skin friction using a value of 650 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. In general, piers should be spaced no closer than three diameters, center to center.

Resistance to lateral loads on piers can be obtained from a passive equivalent fluid pressure of 300 pcf applied over 2 pier diameter. The passive pressure can be assumed to commence at the ground surface, but should be neglected in the upper 12 inches unless confined by pavement or slab.

Where planned cuts exceed about 7 feet below the original ground surface to final grade (top of slab, etc.), spread footings can be used for retaining wall foundations. However, the use of spread footings at retaining wall locations should be evaluated by the soil engineer during final design. The portion of retaining wall foundations extending into firm, natural soil or bedrock can impose a passive equivalent fluid pressure and a friction factor of 300 pcf and 0.30, respectively, to resist sliding. Such spread footings can be designed for dead plus code live load and total design load (including wind or seismic forces) bearing pressures of 2,000 and 3,000 psf, respectively.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch-diameter, perforated, rigid plastic pipe (SDR 35 or equivalent) sloped to drain to outlets by gravity and free-draining, crushed rock or gravel (drainrock). The crushed rock or gravel should extend to within 1-foot of the surface. The drainrock should conform to the quality requirements for Class 2 Permeable Materials in accordance with the latest edition of the 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 covered and separated from the soil bank by a nonwoven, geotextile fabric weighing at least 4 ounces per square yard, such as Mirafi 140N or equivalent. The upper 1 foot should be backfilled with compacted soil to inhibit surface water infiltration unless capped with a concrete slab. The ground surface behind retaining walls should be sloped to drain. Where migration of moisture through retaining walls would be detrimental, the walls should be waterproofed.

Roadway Pavements

For planning purposes, we assume that the streets in the subdivision will be constructed of asphalt concrete and aggregate base materials overlying compacted on-site soil subgrade. The asphalt concrete and aggregate base materials used should conform to the quality requirements of the current edition of the Caltrans Standard Specifications.

The results of the laboratory stabilometer tests from two bulk samples obtained at approximate subgrade level indicate an average R-value of 45. However, because of variability of soil conditions at planned subgrade level, we recommend that an R-value of 20 be used for determination of the pavement sections.

Prior to subgrade preparation, all underground utilities in the paved areas should be installed and properly backfilled. Subgrade soils should be uniformly moisture conditioned, compacted to at least 95 percent relative compaction and provide a firm and nonyielding surface. This may require overexcavation or scarifying and recompaction to achieve uniformity. Where expansive soils are exposed at subgrade elevation, the recommended relative compaction can be reduced to 93 percent, however, the expansive subgrade soils should be moisture conditioned to at least 4 percent over optimum and maintained at the above optimum moisture content until covered with the aggregate base. The aggregate base materials should be placed in layers and compacted to at least 95 percent relative compaction. The aggregate base should also be firm and nonyielding.

Where on-site expansive soils are exposed at subgrade, future wetting and drying along pavement edges can occur. Pavement maintenance, especially repair of edge cracking, should be anticipated. Increased pavement performance and reduced future maintenance could be accomplished by underlying asphalt-paved areas with at least 12 inches of imported fill of low expansion potential or lime-treated on-site soil. Such materials, if used, should extend at least 3 feet beyond pavement edges, where attainable.

Conventional curb and sidewalk and/or adjacent landscaping with an automatic sprinkler system that provides an ample, fairly uniform distribution of water can also provide some benefit in reducing future maintenance. Where sidewalks or concrete driveway aprons are not immediately adjacent to the edge of pavement, a moisture cutoff barrier approximately 36 inches deep could be constructed behind the pavement edge. An example of a moisture cutoff barrier is presented on Plate 11. Prior to the installation of moisture cutoff barriers, if used, on-site highly expansive soils adjacent to and within 3 feet of the pavement edges should be fully preswelled so as to close all shrinkage cracks for their full depth, as previously discussed. The specific need for a moisture cutoff barrier should be evaluated by the appropriate governing agency and soil engineer during the grading when the materials are exposed.

Soil Engineering Drainage

Ponding water will cause softening of the site soils and could be detrimental to foundations. It is important that the areas adjacent to the houses be sloped to drain away from and around the structures. A gradient of at least 1/2-inch per foot extending at least 4 feet from the foundations should be maintained. The roofs should be provided with gutters, and the downspouts should discharge onto paved areas, splash blocks draining at least 30 inches away from foundation, or be connected to nonperforated rigid plastic pipelines that discharge into planned drainage facilities.

To help reduce potential hydrostatic pressure beneath the concrete floor slabs, perforated plastic pipes could be embedded in the grade below the slabs. The underslab subdrain system, if installed, should be configured so as to drain each bay created by interior and/or perimeter foundations. The underslab subdrain system should be connected to a nonperforated outlet pipe that extends through or beneath the perimeter foundation to a suitable discharge point. A typical cross-section of our recommended underslab subdrain is shown on Plate 12. We could provide additional consultation concerning the configuration and location of underslab subdrain systems during final design once foundation plans have been prepared. Roof downspouts and surface drains must be maintained entirely separate from underslab subdrains.

Supplemental Soil Engineering Services

We should review final grading and foundation plans for conformance with the intent of our recommendations. During site grading operations, we should provide intermittent observation and testing to determine the conditions encountered and to 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 also observe foundation excavations and pier drilling operations to verify that suitable bearing materials are encountered. In addition, we recommend that the building pads be observed by the soil engineer to verify that soils are sufficiently moist and in a fully preswelled condition prior to casting of slabs, to verify the conditions observed are as anticipated, and to modify our recommendations, if warranted. Concrete and reinforcing placement 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 placement of concrete.

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. This scope of work is limited to evaluating the physical properties of earth materials considered typical of geotechnical engineering practice and does not include other concerns such as soil chemistry, corrosion potential, mold and soil and/or groundwater contamination.

Subsurface conditions are complex and may differ from those indicated by surface features or encountered at test boring locations. Therefore, variations in subsurface conditions not indicated on the boring 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 charged for on an hourly basis in accordance with our Standard Schedule of Charges. Such supplemental services are performed on an as-requested basis, and we can 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.

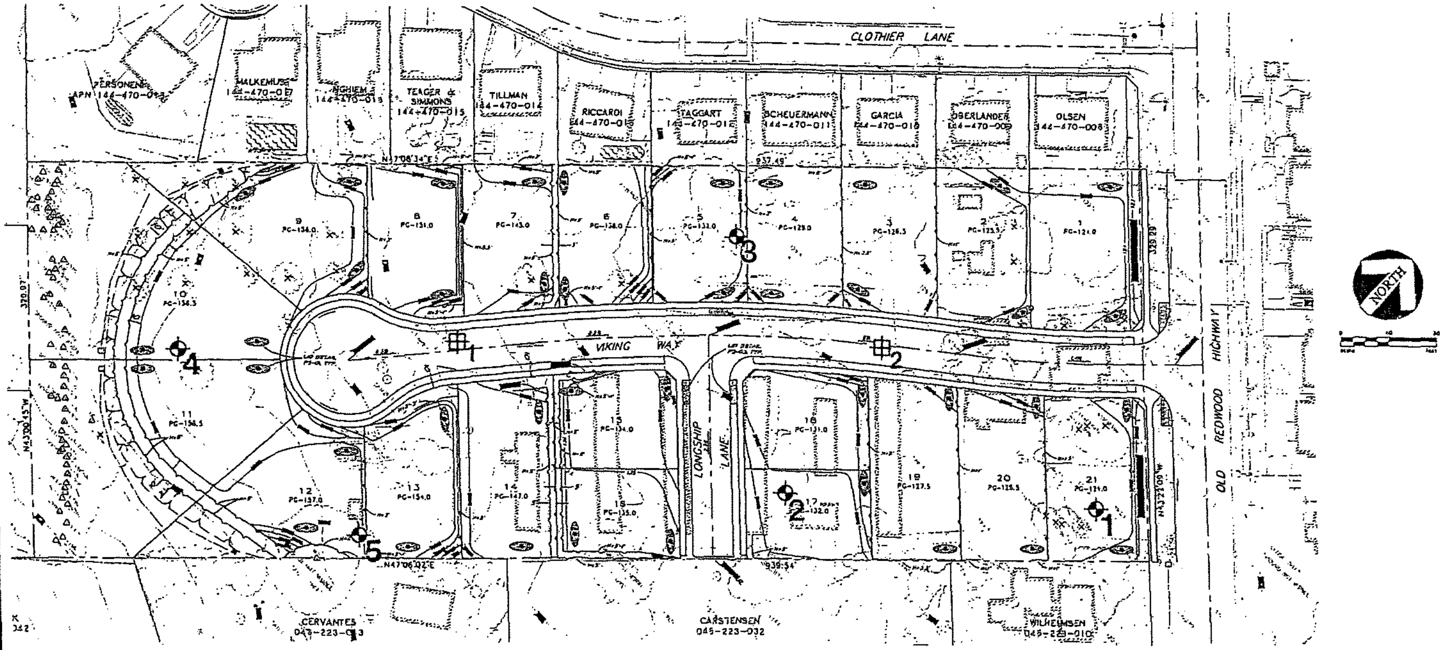
LIST OF PLATES

DISTRIBUTION

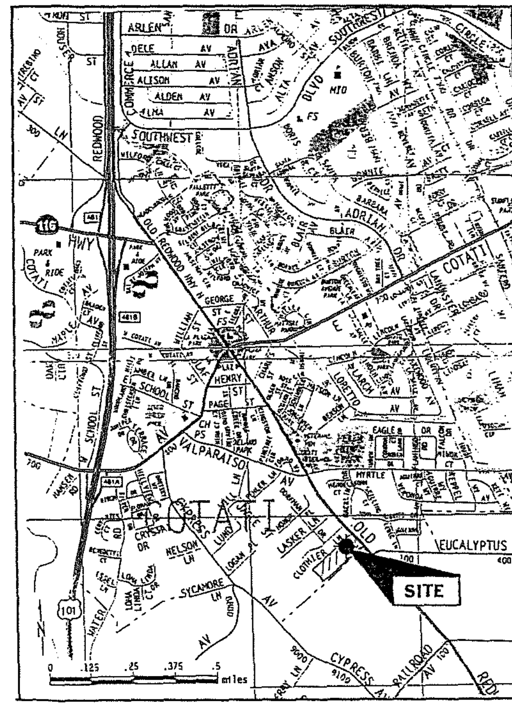
Copies submitted: 5

Pink Viking, LLC
435 E Street
Santa Rosa, CA 95404
Attention: Keith Christopherson

JM/JKR:nay/ra/Job No. 689.1.1



MP DESIGN



SITE VICINITY MAP

Approximate Test Boring Location

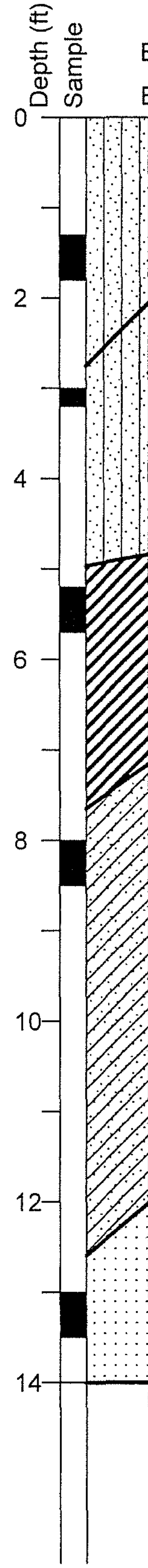
Approximate Location of Bulk Sample for R-value Testing

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 09/13/14 Appr: JM	TEST BORING LOCATION PLAN AND SITE VICINITY MAP PINK VIKING SUBDIVISION 8841 OLD REDWOOD HIGHWAY COTATI, CALIFORNIA	PLATE 1

LOG OF BORING 1

Equipment 6" FLIGHT AUGER

Elevation 124.0 Date 8-7-14



Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Description
				LIGHT GRAY SILTY FINE SAND (SM), dense, dry, with gravel, porous, with roots to 1 foot (topsoil)
Percent Free Swell = 10 LL = NONPLASTIC PL = NONPLASTIC PI = NONPLASTIC	32	1.8	108	
				LIGHT BROWN SILTY SAND (SM), very dense, dry, with gravel
	50	5.3		
				DARK GRAY-BROWN SANDY CLAY (CH), stiff, wet, with fine gravel, plastic
LL = 60 PL = 35 PI = 35 UC = 1930	14	20.2	107	
				YELLOW-BROWN CLAYEY SAND (SC), dense, moist, slightly plastic
UC(P) = 2500 UC(P) = 4500+	38	21.2	98	
				YELLOW-BROWN SANDSTONE OF THE PETALUMA FORMATION, deeply weathered, low hardness, friable
	40	14.2	107	
				(No free water encountered)

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 689.1.1

Date: 9-29-14

Appr: JM

LOG OF BORING 1

PINK VIKING SUBDIVISION
COTATI, CALIFORNIA

PLATE **2**

*Converted to Standard Penetration Blow Counts

LOG OF BORING 2

Equipment 6" FLIGHT AUGER

Elevation 135.5 Date 8-7-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	LIGHT GRAY SILTY FINE SAND (SM), medium dense, dry, porous to 1 foot, with roots (topsoil)
UC(P) = 4500+ Percent Free Swell = 65	28	22.4	94	2	ORANGE-BROWN SANDY CLAY (CL), very stiff, moist to wet
UC(P) = 4500+ Percent Free Swell = 40	25			4	MOTTLED LIGHT GRAY AND ORANGE CLAYSTONE OF THE PETALUMA FORMATION, deeply weathered, low hardness, friable
				6	
	38	35.2	85		becomes light gray
				8	
	50+	15.8		10	MOTTLED YELLOW AND ORANGE SANDSTONE OF THE PETALUMA FORMATION, deeply weathered, low hardness, friable

(No free water encountered)

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 689.1.1

Date: 9-29-14

Appr: Gm

LOG OF BORING 2

PINK VIKING SUBDIVISION
COTATI, CALIFORNIA

PLATE
3

*Converted to Standard Penetration Blow Counts

LOG OF BORING 3

Equipment: 6" FLIGHT AUGER

Elevation: 129.5 Date: 8-7-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Description
				MOTTLED DARK BROWN AND DARK GRAY VERY SANDY SILT (ML), very stiff, dry, porous to 1 foot (topsoil)
Percent Free Swell = 30	16	9.4	108	
				becomes moist
Percent Free Swell = 90	13			DARK BROWN SANDY CLAY (CH), very stiff, moist, plastic
Percent Free Swell = 75	25	17.7	113	with gravel
	50+	32.6		MOTTLED ORANGE AND BROWN SILTSTONE OF THE PETALUMA FORMATION, deeply weathered, low hardness, friable
	50+			

(No free water encountered)

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 689.1.1

Date: 9-29-14

Appr: JM

LOG OF BORING 3

PINK VIKING SUBDIVISION
COTATI, CALIFORNIA

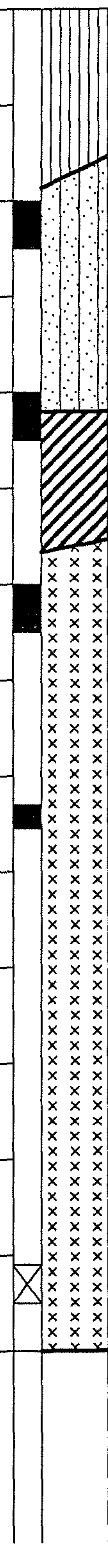
PLATE 4

*Converted to Standard Penetration Blow Counts

LOG OF BORING 4

Equipment: 6" FLIGHT AUGER

Elevation: 164.0 Date: 8-7-14



Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)	Depth (ft)	Description
				0	DARK BROWN SANDY SILT (ML), very stiff, dry, porous (topsoil)
TxUU = 3690 (500)	33	6.1	109	2	MOTTLED BROWN AND GRAY SILTY SAND (SM), dense, dry, porous to 2.5 feet
	44	13.3	117	4	DARK GRAY SANDY CLAY (CH), hard, moist, plastic, with gravel
Percent Free Swell = 50	50+	15.3	104	6	MOTTLED RED AND YELLOW SILTSTONE OF THE PETALUMA FORMATION, deeply weathered, low hardness, friable
Percent Free Swell = 50				8	becomes dark yellow-brown sampled from cuttings
				10	
				12	
	50+				sample discarded
				14	
					(No free water encountered)

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 689.1.1

Date: 9-29-14

Appr: Jm

LOG OF BORING 4

PINK VIKING SUBDIVISION
COTATI, CALIFORNIA

PLATE **5**

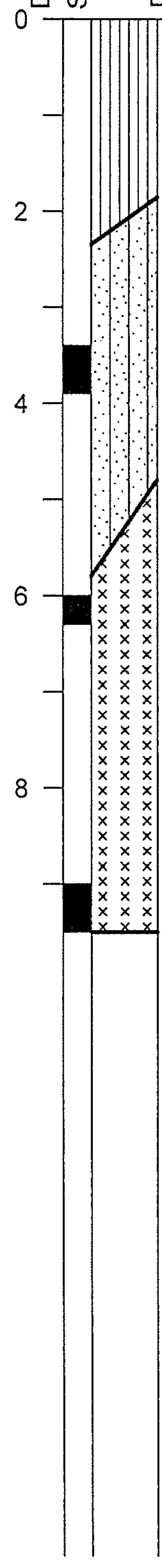
*Converted to Standard Penetration Blow Counts

LOG OF BORING 5

Equipment: 6" FLIGHT AUGER

Elevation: 158.0 Date: 8-7-14

Laboratory Test Results or Remarks	Blows/foot *	Moisture Content (%)	Dry Density(pcf)
TxUU = 3650 (1500)	32	4.6	108
UC(P) = 4500+	50+	10.2	108
	50+	36.7	



LIGHT BROWN SANDY SILT (ML), medium stiff, dry to moist, porous, with roots

LIGHT GRAY VERY SILTY SAND (SM), dense, dry, with some root fibers, gravel

LIGHT YELLOW-BROWN SILTSTONE OF THE PETALUMA FORMATION, low hardness, deeply weathered, friable

becomes mottled gray and orange

(No free water encountered)

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 9-29-14 Appr:	LOG OF BORING 5 PINK VIKING SUBDIVISION COTATI, CALIFORNIA	PLATE 6

*Converted to Standard Penetration Blow Counts

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

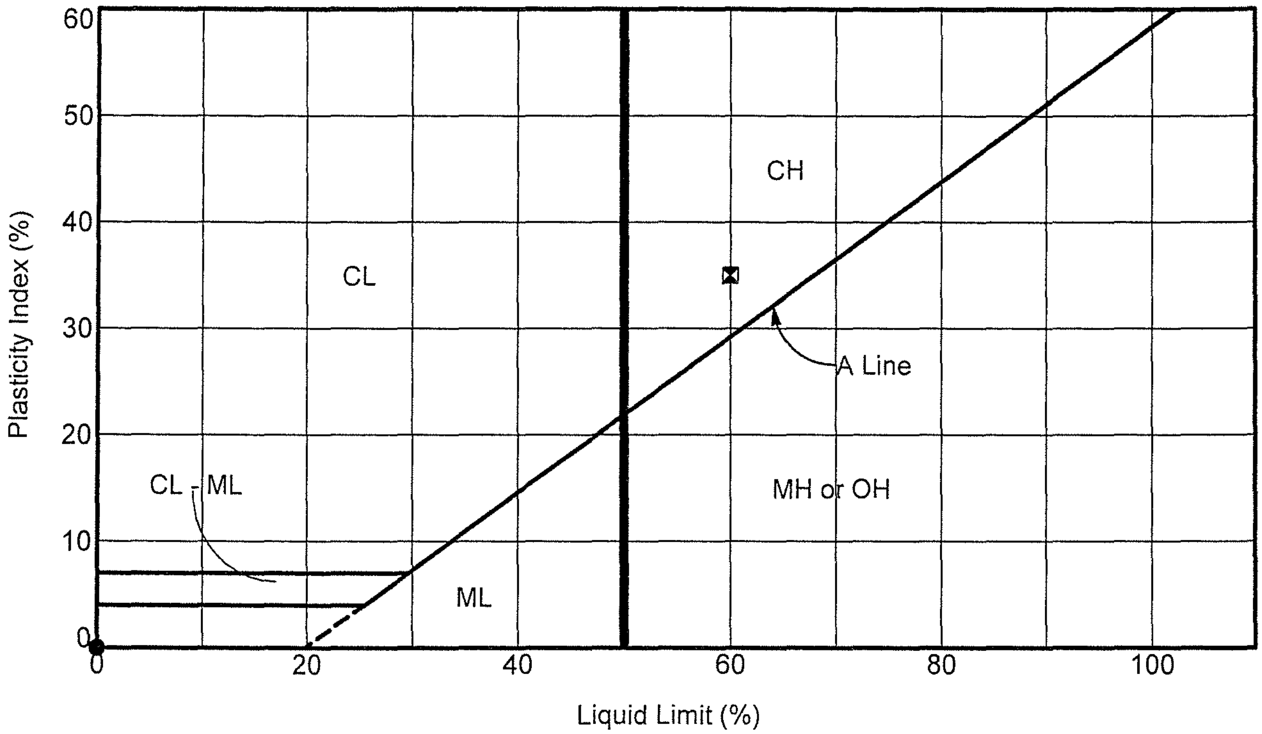
KEY TO TEST DATA

				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: 689.1.1 Date: 9-29-14 Appr: [signature]	SOIL CLASSIFICATION CHART AND KEY TO TEST DATA PINK VIKING SUBDIVISION COTATI, CALIFORNIA	PLATE 7



ASTM D 4318-98

Symbol	Classification and Source	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Free Swell (%)
●	**LIGHT GRAY SILTY FINE SAND (SM)** **Test Boring 1 at 1.3 feet**	**NP**	**NP**	**NP**	**10**
⊠	**DARK GRAY-BROWN SANDY CLAY (CH)** **Test Boring 1 at 5.2 feet**	**60**	**25**	**35**	**90**

REESE & ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS

Job No: 689.1.1

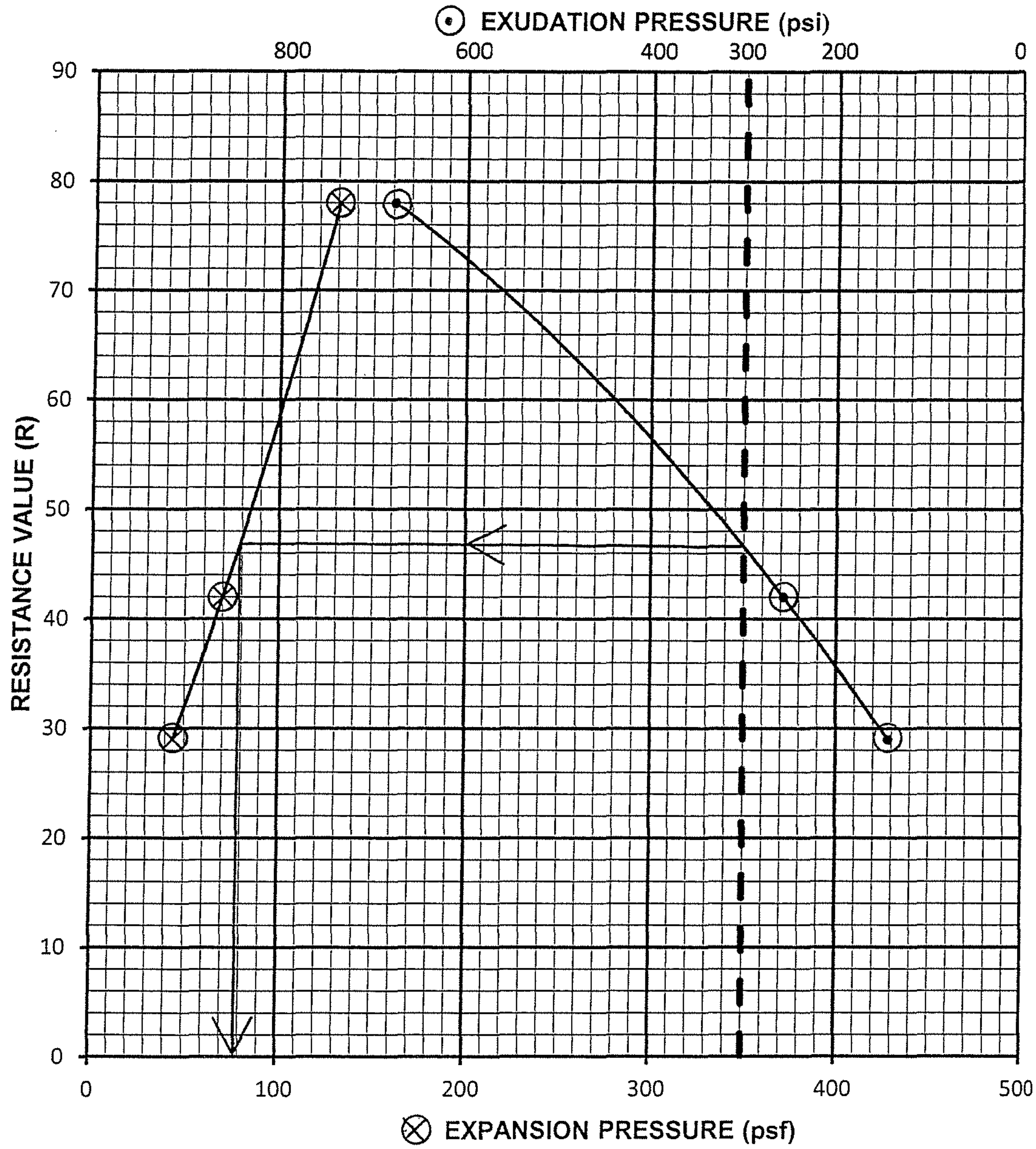
Date: 9-29-14

Appr: Jm

ATTERBERG LIMITS TEST RESULTS

PINK VIKING SUBDIVISION
COTATI, CALIFORNIA

PLATE
8

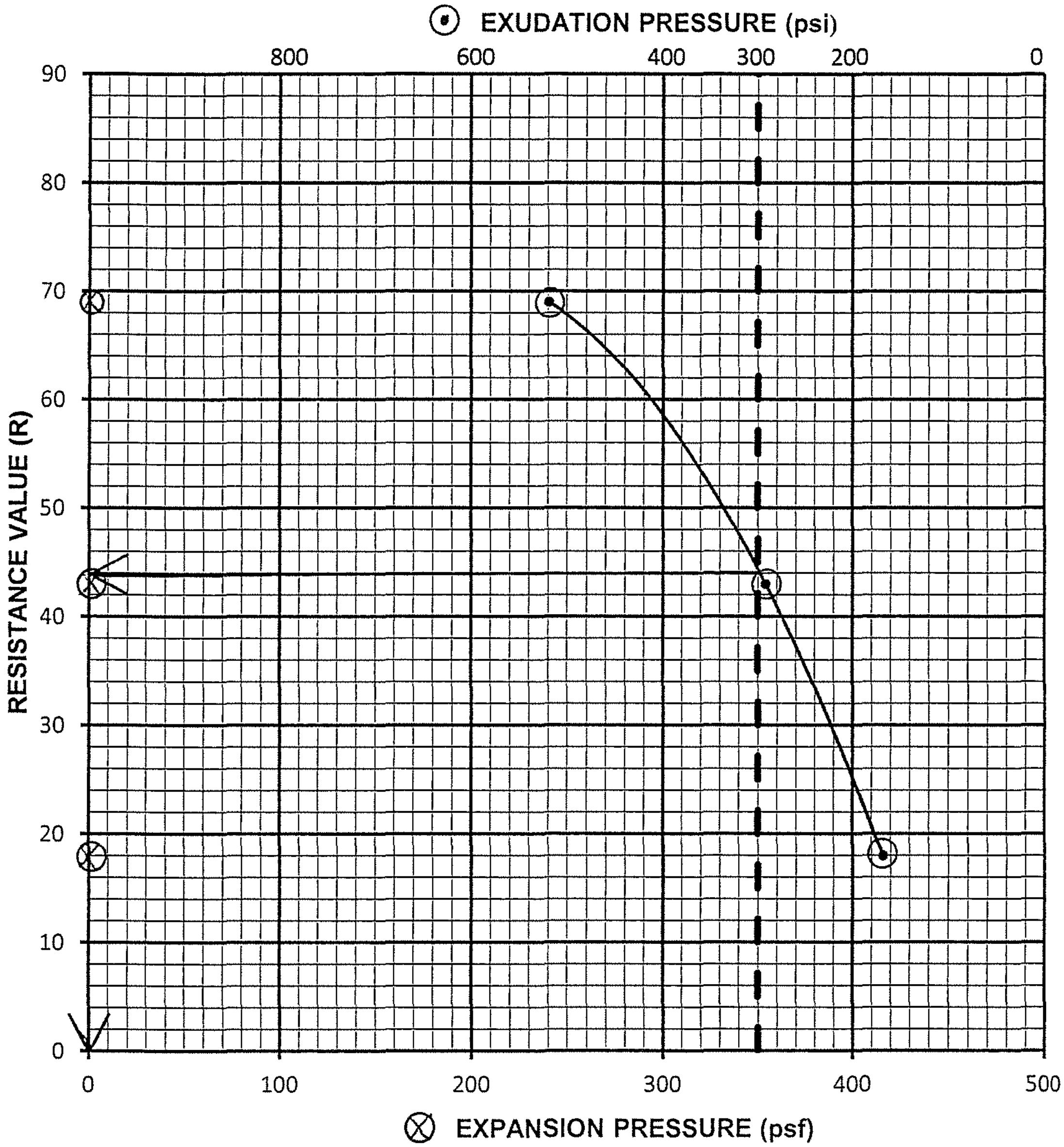


Specimen	A	B	C
Moisture Content (%)	10.0	11.6	13.0
Dry Density (pcf)	124	122	119
Exudation Pressure (psi)	678	257	144
Expansion Pressure (psf)	131	70	44
Resistance Value (R)	78	42	29

Sample Source	Classification	Expansion Pressure (psf)	R-Value
Bulk 1: On-site Subgrade	BROWN CLAYEY SAND (SC) with gravel	79	47

Reference: State of California 301, ASTM D2844

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 09-15-14 Appr: [signature]	R-VALUE TEST RESULTS PINK VIKING SUBDIVISION 8841 OLD REDWOOD HIGHWAY COTATI, CALIFORNIA	PLATE 9

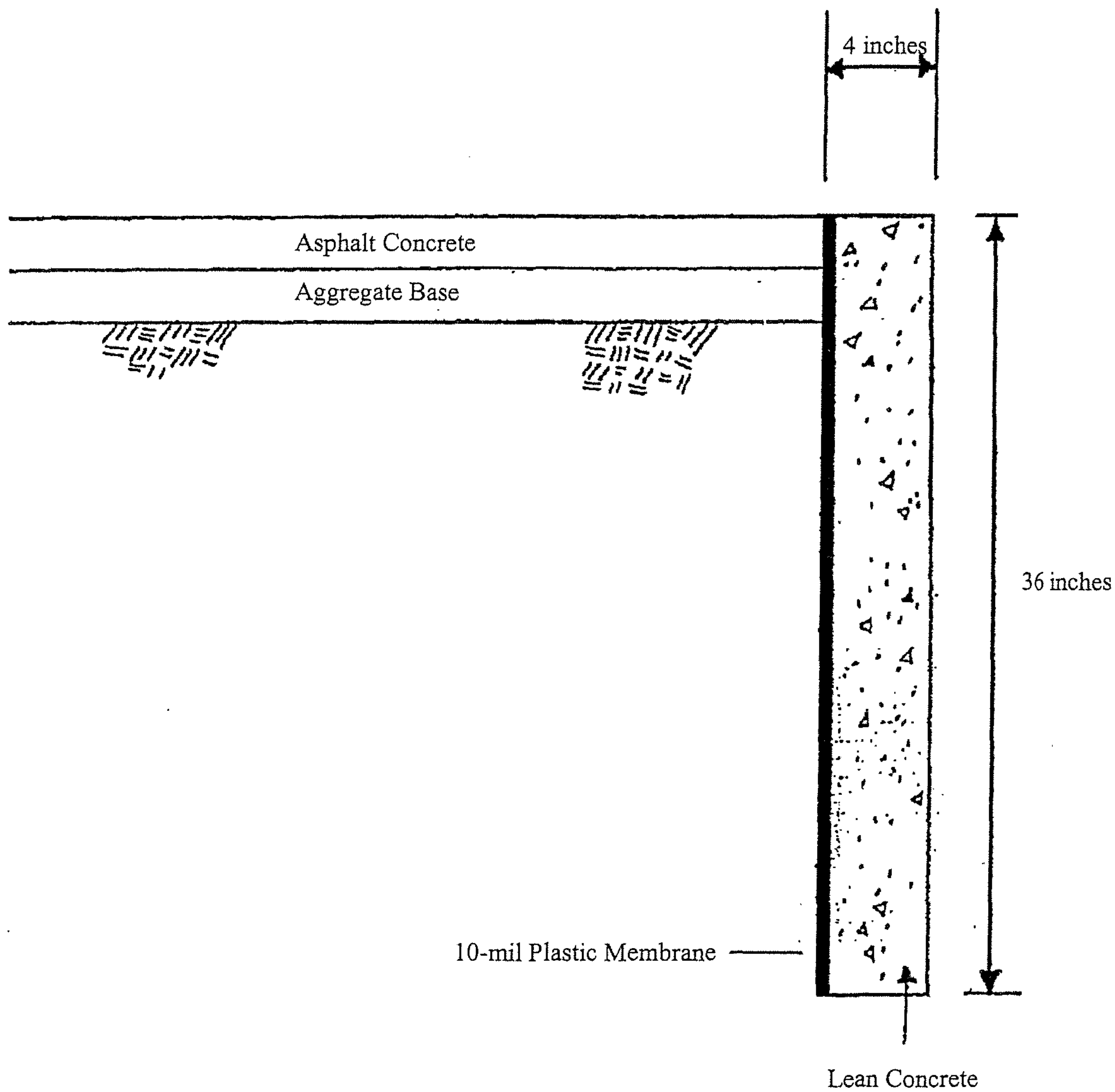


Specimen	A	B	C
Moisture Content (%)	8.7	9.7	11.0
Dry Density (pcf)	128	127	123
Exudation Pressure (psi)	519	292	168
Expansion Pressure (psf)	0	0	0
Resistance Value (R)	69	43	18

Sample Source	Classification	Expansion Pressure (psf)	R-Value
Bulk 2: On-site Subgrade	LIGHT BROWN CLAYEY SAND (SC) with gravel	0	44

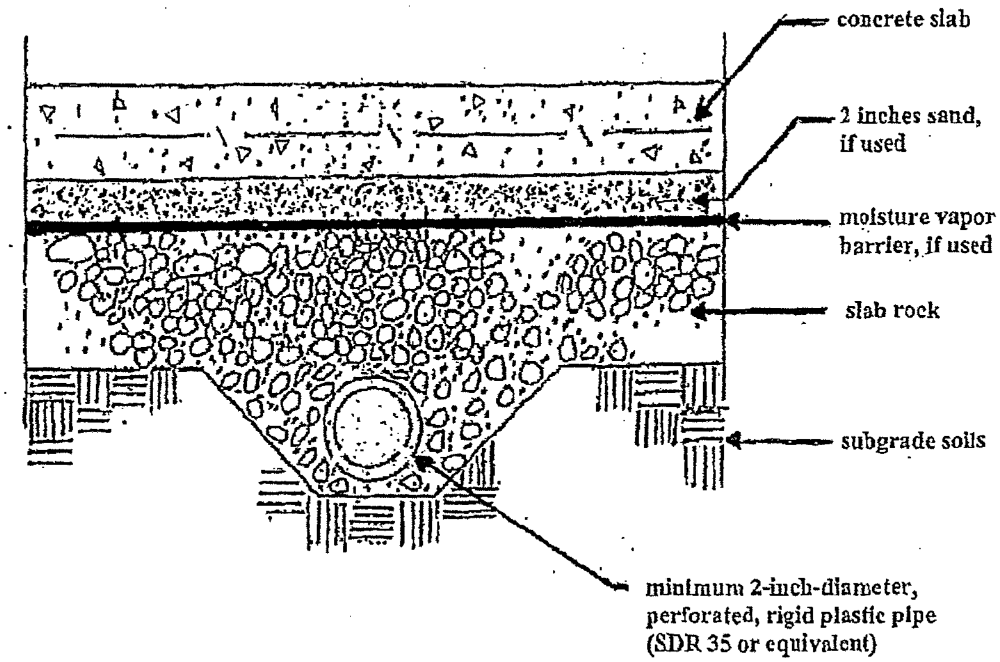
Reference: State of California 301, ASTM D2844

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 09-15-14 Appr:	R-VALUE TEST RESULTS PINK VIKING SUBDIVISION 8841 OLD REDWOOD HIGHWAY COTATI, CALIFORNIA	PLATE 10



Not to Scale

REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 09-27-14 Appr: Jm	TYPICAL CROSS-SECTION MOISTURE CUTOFF BARRIER PINK VIKING SUBDIVSION COTATI, CALIFORNIA	PLATE 11



REESE & ASSOCIATES CONSULTING GEOTECHNICAL ENGINEERS	Job No: 689.1.1 Date: 09-27-14 Appr: JM	TYPICAL CROSS SECTION UNDERSLAB SUBDRAIN PINK VIKING SUBDIVISION COTATI, CALIFORNIA	PLATE 12

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February 4, 2022

Job No. 2541.1.13

Pink Viking, ORH
400 College Avenue
Santa Rosa, CA 95401
Attention: Brian Flahavan
btf@flahavanlaw.com

Report
Soil Engineering Consultation
and Report Update
Pink Viking Subdivision
Cotati, California

As requested, this report presents the results of our soil engineering consultation and investigation report update for the proposed Pink Viking Subdivision at 8841 Old Redwood Highway in Cotati, California. We performed a soil investigation for the project, and the results were presented in a report dated October 14, 2014. Our general recommendations included criteria for site grading to accommodate post-tensioned slab floor and foundation systems. We have reviewed that report, and our engineer was on-site on February 1, 2022 to perform a brief site observation. Based on our review of the report and the current surface features exposed, we judge that the general conclusions and recommendations would still be applicable to the proposed construction with the following updated seismic design criteria per the 2019 California Building Code (CBC).

SEISMIC DESIGN

The geologic maps reviewed did not indicate the presence of active faults at the site, and the property is not located within a presently designated Alquist-Priolo Earthquake Fault Zone. 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 presented below in accordance with procedures outlined in Section 1613 of the 2019 CBC.

2019 CBC Ground Motion Parameters

Site Class	C

Mapped Spectral Response Accelerations:

S_S	1.579g
S_1	0.600g

Design Spectral Response Accelerations:

S_{DS}	1.263g
S_{D1}	0.560g

We trust this provides the information needed at this time. If you have questions or wish to discuss this in more detail, please do not hesitate to contact us.

Yours very truly,

REESE & ASSOCIATES

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Civil Engineer No. 85560

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Copies Submitted: 1