R G H

CONSULTANTS, INC.

GEOTECHNICAL STUDY REPORT

WILLOW CREEK ROAD 2ND BRIDGE CROSSING
WILLOW CREEK ROAD
SONOMA COUNTY, CALIFORNIA

Project Number:
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INTRODUCTION

This report presents the results of our geotechnical study for the Willow Creek Road 2nd Bridge project to be constructed on Willow Creek Road in Sonoma County, California. Willow Creek Road is a paved County of Sonoma roadway. The planned bridge location is approximately 1 mile east (measured along the road) of the intersection of Willow Creek Road and Highway 1. Six corrugated metal culverts are located below the existing roadway. An existing pile supported bridge is located northeast of the proposed improvements. The site location is shown on Plate 1, Appendix A.

We understand it is planned to construct a new crossing at the location of the corrugated metal culverts. The crossing may consist of segmented box culverts, bottomless arched culverts, or a bridge. Some fill may be required to raise the grade before and after the new crossing.

SCOPE

The purpose of our study, as outlined in our Professional Service Agreement dated February 1, 2008, was to generate geotechnical information for the design and construction of the project. Our scope of services included reviewing selected published geologic data pertinent to the site; evaluating subsurface conditions with test borings and laboratory tests; analyzing the field and laboratory data; and presenting this report with the following geotechnical information:

1. A brief description of soil and groundwater conditions observed during our study;

2. A discussion of seismic hazards that may affect the proposed project; and

3. Conclusions and recommendations regarding:

   a. Primary geotechnical engineering concerns and mitigating measures, as applicable;

   b. Site preparation and grading in the roadway including treatment of weak, porous, compressible and/or expansive surface soils;

   c. Foundation types, design criteria, and estimated settlement behavior;

   d. Lateral forces for bridge abutments and wing walls or culvert design, as applicable;

   e. Preliminary pavement thickness based on our experience with similar soils and projects;

   f. Utility trench backfill;

   g. Geotechnical engineering drainage improvements; and

   h. Supplemental geotechnical engineering services.

STUDY

Site Exploration

We reviewed selected geologic references pertinent to the site and the soil borings performed for the existing bridge located northeast of the new improvements (Moore and Taber, 1975). The geologic literature reviewed is listed in Appendix B.

On March 3 and 5, 2008, we performed a geotechnical reconnaissance of the site and explored the subsurface conditions by drilling four test borings to depths ranging from about 5½ to 71½ feet. The borings were drilled with a truck-mounted rotary wash drill rig at the approximate locations shown on the Exploration Plan, Plate 2. The test boring locations were determined approximately by pacing their distance from features shown on the Exploration Plan and should be considered accurate only to the degree implied by the method used. Our field engineer located and logged the borings and obtained samples of the materials encountered for visual examination, classification and laboratory testing.

Relatively undisturbed samples were obtained from the borings at selected intervals by driving a 2.43-inch inside diameter, split spoon sampler, containing 6-inch long brass liners, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches. The blows required to drive each 6-inch increment were recorded and the blows required to drive the last 12 inches, or portion thereof, were converted to equivalent Standard Penetration Test (SPT) blow counts for correlation with empirical data. Disturbed samples were also obtained at selected depths by driving a 1.375-inch inside diameter (2-inch outside diameter) SPT sampler, without liners or rings, using a 140-pound hammer dropping approximately 30 inches. The sampler was driven 12 to 18 inches, the blows to drive each 6-inch increment were recorded, and the blows required to drive the final 12 inches, or portion thereof, are provided on the test boring logs. A disturbed "bulk" sample of the anticipated subgrade soils at the bridge approach was also obtained from the test borings and placed in a plastic bucket.

The logs of the test borings showing the materials encountered, groundwater conditions, converted blow counts and sample depths are presented on Plates 3 through 6. The soils are described in accordance with the Unified Soil Classification System, outlined on Plate 7.

The test boring logs show our interpretation of subsurface soil and groundwater conditions on the dates and at the locations indicated. Subsurface conditions may vary at other locations and times. Our interpretation is based on visual inspection of soil samples, laboratory test results, and interpretation of drilling and sampling resistance. The location of the soil boundaries should be considered approximate. The transition between soil types may be gradual.

Laboratory Testing

The samples obtained from the borings were transported to our office and re-examined to verify soil classifications, evaluate characteristics, and assign tests pertinent to our analysis. Selected samples were laboratory tested to determine their water content, dry density, classification (Atterberg Limits, percent of silt and clay), triaxial shear strength, consolidation, and expansion potential (Expansion Index - EI). The test results are presented and/or referenced on the test boring logs. Results of the classification, triaxial shear strength, and consolidation tests are presented on Plates 8 through 15.

SITE CONDITIONS

General

Sonoma County is located within the California Coast Range geomorphic province. This province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges and valleys. The oldest bedrock units are the Jurassic-Cretaceous Franciscan Complex and Great Valley sequence sediments originally deposited in a marine environment. Subsequently, younger rocks such as the Tertiary-age Sonoma Volcanics group, the Plio-Pleistocene-age Clear Lake Volcanics and sedimentary rocks such as the Guinda, Domengine, Petaluma, Wilson Grove, Cache, Huichica and Glen Ellen formations were deposited throughout the province. Extensive folding and thrust faulting during late Cretaceous through early Tertiary geologic time created complex geologic conditions that underlie the highly varied topography of today. In valleys, the bedrock is covered by thick alluvial soils.

Geology and Soils

The California Geological Survey's (CGS), formerly known as the California Division of Mines and Geology (CDMG), geologic maps (Huffman and Armstrong, 1980) indicate the property is underlain by alluvium (Qal) that is flanked by hillsides underlain by conglomerate of the Great Valley Sequence (KJgvc) to the northeast and Franciscan Complex (KJfs) to the south and west. The alluvium is shown to comprise sand, gravel, silt, and clay. The Franciscan Complex is shown to comprise sheared shale and sandstone that contains generally resistant masses of chert, "high grade" metamorphic rock, variable shattered sandstone and greenstone, metagreenstone and generally less resistant serpentinite. Fault mapping by CGS (Bortugno, 1982) indicates that a fault showing no evidence of Quaternary (within the last 5,000,000 years) displacement extends through the alluvium between the proposed improvements and the existing bridge.

Mapping by the U.S. Soil Conservation Service (Soil Survey Staff, 2008) has classified soil over the portion of this property proposed for improvement as belonging to the Tidal Marsh series. The Tidal Marsh series is shown to comprise variable soil textures. Degree of plasticity and shrink-swell potential are not described. The risk of corrosion is given as high for uncoated steel and high for concrete. Performing corrosivity tests to verify these values was not part of our requested and/or proposed scope of work. Should the need arise, we would be pleased to provide a proposal to evaluate these characteristics.

Landslides

The CGS maps of landslides (Huffman, 1980) indicate large-scale slope instability of the hillside south of the proposed improvements including a large landslide that extends to the top of the ridge. We did observe landslides in that area during our study. In addition, there is a landslide mapped on the slope northeast of the site, easterly of the existing bridge. The proposed site is located in the alluvial soils that make up the valley floor. It is possible that landslide debris could extend below the alluvium and thus below the proposed improvements. Movement of the landslides described herein would not only impact the planned improvements, but the valley floor in general. Therefore, for our analysis of the proposed improvements, we have not included a detailed analysis of the landslides. Reactivation, although unlikely, would uniformly disrupt the bridge approaches, creek alignment and existing features.

Surface

The proposed improvements are located within a small valley where Willow Creek flows towards the Russian River. An existing roadway that is essentially flat in the immediate vicinity of the planned improvements traverses through the valley. The area beyond the roadway is covered with heavy vegetation. Six culverts allow water to pass under the roadway. Natural drainage consists of sheet flow over the ground surface that concentrates in man made surface drainage elements such as culverts and natural drainage elements such as swales and creeks.

Subsurface

Our borings and laboratory tests indicate that the existing roadway below the asphalt concrete and aggregate base section is blanketed by 4 to 8 feet of medium dense to dense clayey gravel that is weak to a depth of approximately 2 feet below existing roadway grade. These surface materials are underlain by layers of clay and silt with interbedded layers of sand to the maximum depth explored (71½ feet). The clay and silt soils are compressible under structure and fill loading to depths ranging from 45 to 48 feet.

These conditions differ from those encountered in the closest boring drilled for the existing bridge to the northeast (Moore and Taber, 1975). That boring encountered loose to medium dense, potentially liquefiable sand and very soft to soft clay over dense sand encountered at a depth of about 37 feet. The differences are likely explained by the existing fault between the two locations (Bortugno, 1982).

Groundwater

Free groundwater was first detected in our borings at depths ranging from 2½ to 5 feet below the ground surface at the time of drilling. Fluctuation in the groundwater level typically occurs because of a variation in rainfall intensity, duration and other factors such as flooding and periodic irrigation.

Flooding

Our review of the Federal Emergency Management Agency (FEMA) Flood Zone Map for Sonoma County, California, Unincorporated Areas (NO. 060375 0640B), dated April 2, 1991, indicates that the site is located within Zone "X," an area determined to be outside the 500-year flood plain. Evaluation of flooding potential is typically the responsibility of the project civil engineer.

DISCUSSION AND CONCLUSIONS

Seismic Hazards

General

We did not observe subsurface conditions within the portion of the property we studied that would suggest the presence of materials that may be susceptible to seismically induced lurching. Therefore, we judge the potential for the occurrence of this phenomenon at the site to be low.

Seismicity

Data presented by the Working Group on California Earthquake Probabilities (2007) estimates the chance of one or more large earthquakes (Magnitude 6.7 or greater) in the San Francisco Bay region within the next 30 years to be approximately 63 percent. Therefore, future seismic shaking should be anticipated at the site. It will be necessary to design and construct the proposed improvements in strict adherence with current standards for earthquake-resistant construction.

Faulting

We did not observe landforms within the area that would indicate the presence of active faults and the site is not within a current Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). An unnamed fault that shows no evidence of Quaternary (last 5,000,000 years) displacement is shown on the fault map by Bortugno (1982). Therefore, we believe the risk of fault rupture at the site is low. However, the site is within an area affected by strong seismic activity. Several northwest-trending Earthquake Fault Zones exist in close proximity to and within several miles of the site (Bortugno, 1982). The shortest distances from the site to the mapped surface expression of these faults are presented below in Table 1.

TABLE 1
ACTIVE FAULT PROXIMITY

Fault	Direction	Distance-Miles
San Andreas	SW	2½
Healdsburg-Rodgers Creek	NE	18½
West Napa	E	39
Maacama	NE	23

Liquefaction

Liquefaction is a rapid loss of shear strength experienced in saturated, predominantly granular soils below the groundwater level during strong earthquake ground shaking due to an increase in pore water pressure. The occurrence of this phenomenon is dependent on many complex factors including the intensity and duration of ground shaking, particle size distribution and density of the soil.

Granular soils were encountered at the site below the groundwater table. Therefore, we performed an analysis of the blow count data from our borings using the methods of Seed and Idriss (1982), Seed and others (1985), Youd and Idriss (2001), and Boulanger and Idriss (2006). These procedures normalize the blow counts to account for overburden pressure, rod length, hammer energy, and fines (percent of silt and clay) content. Once the blow counts are normalized and adjusted to a clean sand blow count, the critical blow count is then determined. The critical blow count is calculated using the same procedures referenced above and requires a peak ground acceleration and design earthquake magnitude.

Peak ground acceleration (PGA) was determined using the methods in the 2007 California Building Code (CBC) and Chapter 11 of the American Society of Civil Engineers (ASCE) Standard 7-05, titled "Minimum Design Loads for Buildings and Other Structures" (2006). Section 11.8.3 of ASCE Standard 7-05 states that the PGA for liquefaction evaluation can be defined as the design spectral response acceleration at short periods with 5 percent damping (S_{DS}) divided by 2.5. The S_{DS} value is determined using the United States Geological Survey's Earthquake Ground Motion Parameter Java Application (2007). Based on the site's latitude and longitude of 38.435 °N and –123.087 °W, respectively, the S_{DS} value is 1.171g for Site Class E. Therefore, the PGA used for our evaluation is 0.468g.

The San Andreas fault is most likely controlling the ground motions at the Willow Creek Road site. According to Petersen (1996), the San Andreas fault is capable of a M_M 7.6 earthquake. Using this information and the scaling factors presented in Youd and

Idriss (2001), the critical blow count at the site ranges from 21 to 29 blows per foot depending on the depth. The normalized and adjusted blow counts at the site do not exceed this value for soils located below the water table, and thus would have potential to liquefy. Using the methods of Boulanger and Idriss (2006), we were able to screen some of the sand layers out because their Plasticity Index was greater than 7 and their fines content greater than 35 percent. However, two layers of sand approximately 3 and 9 feet thick in Boring B-2 are judged to have a moderate to high potential for liquefaction.

There are three potential consequences of liquefaction: bearing capacity failure, lateral spreading, and differential settlement. Bearing capacity failure is large unpredictable differential settlements that occur when foundations bear in or slightly above (typically within two foundation widths for spread footings) liquefiable materials. The potentially liquefiable soils encountered in Boring B-2 were found as shallow as 11 feet below the existing ground surface. Shallow abutment and culvert foundations are not likely to extend to that depth. Deep foundations, if used, would extend below the potentially liquefiable soils. Therefore, we judge that the potential for bearing capacity failure at the site is low.

Lateral spreading can occur when a potentially liquefiable layer extends to a free face, such as a creek or river bank. Willow Creek is the nearest possible free face and is in the immediate vicinity of the proposed improvements. The potentially liquefiable layer is at least 11 feet deep and was not encountered in both of the deep borings we drilled at the site. Given the depth of the questionable layer and the discontinuous nature of the layers encountered in our borings, we judge that the potential for lateral spreading at the site is low.

Differential, non-bearing capacity related, settlement is caused when the soil densifies under seismic loading. Using the blow count data, potential settlement for the liquefied layers was calculated using the methods of Tokimatsu and Seed (1987). These methods yielded earthquake-induced settlement of the suspect soils in Boring B-2 of about 3 inches. Because potentially liquefiable soils were not encountered in Boring B-3, differential settlement along the improvements could be on the order of 3 inches. In order to reduce the impacts of this settlement, the improvements will need to be supported on a mat slab or on a deep foundation likely consisting of driven piles.

Densification

Densification is the settlement of loose, granular soils above the groundwater level due to earthquake shaking. Typically, granular soils that would be susceptible to liquefaction, if saturated, are susceptible to densification. As discussed in the "Liquefaction" section, the soils at the site have a moderate to high potential for liquefaction. However, the potentially liquefiable soils are located below the groundwater table. Therefore because of the relatively high groundwater level, we judge that there is a low potential for densification to impact the proposed culverts or bridge at the site.

Geotechnical Issues

General

Based on our study, we judge the proposed culverts or bridge can be built as planned, provided the recommendations presented in this report are incorporated into its design and construction. The primary geotechnical concerns during design and construction of the project are:

1. The presence of weak surface soils that extend to about two feet below the existing roadway grade;

2. The presence of soils that are compressible under fill and structure loads to depths ranging from 45 to 48 feet;

3. The presence of potentially liquefiable soils in Boring B-2; and

4. The strong ground shaking predicted to impact the site during the life of the project.

Weak Surface Soils

Weak, porous surface soils, such as those found to a depth of about two feet below the existing pavement surface, appear hard and strong when dry but will lose strength rapidly and settle under the load of fills, foundations, and pavements as their moisture content increases and approaches saturation. The moisture content of these soils can increase as the result of rainfall, periodic irrigation or when the natural upward migration of water vapor through the soils is impeded by and condenses under fills, foundations, and pavements. The detrimental effects of such movements can be remediated by strengthening the soils during grading. This can be achieved by excavating the weak soils and replacing them as properly compacted (engineered) fill.

Compressible Soils

Compressible soils, such as the silt and clay found at the Willow Creek site, will settle under the load of new fills and structure loads. These soils were encountered starting at about 8 feet below the ground surface and extend to 45 to 48 feet in the area of the proposed culverts or bridge. Layers of sand within the silt and clay were encountered in Boring B-2.

We calculated consolidation settlement under various loading conditions at the site including new fill and shallow foundations. We assumed fill thicknesses could range from negligible to 5 feet, which coincides with settlement ranging from negligible to 3½ to 4¼ inches depending how much sand is in the profile. Essentially for every foot of new fill, we estimate approximately 2/3- to 7/8-inch of settlement. For shallow foundations, we analyzed strip footings/mats that could represent culvert foundations and/or widths. Our analysis yielded settlement ranging from 1½ to 3½ inches depending on foundation width and bearing pressure. Due to the presence of the sand layers in the area of Boring B-2, differential settlement could be on the order of 1-inch along the roadway alignment. Because of the variability of the soils in our borings and those in previous borings drilled for the adjacent bridge, differential settlement in the direction of creek flow could be on the order of ½-inch.

We understand that one of the culvert systems being considered for the crossing is segmented boxes. These are connected together in the direction of flow and can also be placed side by side. Due to the variability of the soils, differential settlement between adjacent side by side boxes could range from ½ to 1 inch at the transition between the boxes. In order to reduce the impacts of settlement for the segmented box system, the boxes need to be structurally tied together along the direction of water flow and side to side so that the entire system will act as a unit. Alternatively, the segmented boxes need to be founded on a mat slab.

Another alternative being considered is the bottomless culvert that has foundations on either side of the flow of the creek and an arch support connecting them. This type of culvert system is backfilled in accordance with the manufacturer's recommendations to establish finished grade for the roadway. The settlement experienced at the foundations is somewhat dampened at the roadway surface by the backfill materials. If the above-described settlement is tolerable, the bottomless culverts can be supported on spread footings. Alternatively, the culverts can be supported on a driven pile foundation.

A third alternative being considered is a bridge that would have an abutment on both ends, and depending on the length, may have a center bent. Provided the bridge can withstand the above-described settlement, it can be supported on spread footing supported abutments. Alternatively, the bridge can be supported on driven pile foundations.

Potentially Liquefiable Soils

As discussed in the "Liquefaction" section the site has a moderate to high potential for liquefaction. Potentially liquefiable soils were only encountered in two layers in Boring B-2. Estimated differential settlement along the roadway (between the locations of Borings B-2 and B-3) could be on the order of 3 inches. Because of the presence of a fault in the area and because the extent and thickness of the sand layers encountered are undefined, it is possible that similar differential settlement could occur in the direction of the flow of the creek.

The foundation alternatives described above for the compressible soil condition are applicable for the liquefaction condition. Please note that earthquake-induced settlement will likely make the roadway impassable for foundations constructed on spread footings and for the segmented box culverts that are not structurally tied together or founded on a mat slab. Structurally tied boxes and mat slabs will provide better performance during an earthquake. Liquefaction-induced differential settlement of pile supported will be less than ½-inch.

Foundation Support - Depending on the required performance, especially post-earthquake, for the culverts or bridge, foundation support for these structures can be obtained from spread footings or the segmented box culvert bottoms bottomed on engineered fill or firm, natural materials. Alternatively the segmented boxes can be structurally tied together or founded on a mat slab, and the bottomless culverts or bridge can be supported on driven piles that gain support in friction below the compressible soils.

Pavement Support - After remedial grading, satisfactory support for paved approaches to the culverts or bridge can be obtained on the engineered fill.

On-Site Soil Quality

All fill materials must be select, as subsequently described in "Recommendations." We anticipate that, with the exception of organic matter and of rocks or lumps larger than 6 inches in diameter, the excavated material in the upper 5 feet will be suitable for re-use as general and select fill. Depending on the time of year of construction, the on-site soils may be at a moisture content that makes them difficult to compact. These soils may need to be allowed to dry or mixed with imported soil with lower moisture content (drier material).

Select Fill

The select fill can consist of approved on-site soils or import materials with a low expansion potential. The geotechnical engineer must approve the use of on-site soils as select fill during grading.

Settlement

For the segmented box culverts and structures founded on spread footings or mat slabs, we estimate that differential settlement related to the compressible soils (consolidation) will be about 1 inch along the roadway alignment and ½ inch in the direction of creek flow. Consolidation differential settlement of pile supported foundations is estimated to be about ½ inch in all directions. Earthquake-induced differential settlements are estimated to be about 3 inches for non-pile supported structures and less than ½-inch for pile supported structures.

Fill soils placed to construct the approaches will settle as described previously for compressible and potentially liquefiable soils. The differential settlement between the fill and the culverts or bridge is a function of the fill thickness and the foundation type for the structure. We can provide estimates of this settlement once finished grades and structure foundations have been determined.

Surface Drainage

The site will be impacted by surface runoff. Surface runoff typically sheet flows over the ground surface but can be concentrated by the planned site grading, landscaping, and drainage. It will be necessary to divert surface runoff around slopes and improvements, provide positive drainage away from structures, and install energy dissipaters at discharge points of concentrated runoff.

Excavation Dewatering

Groundwater was encountered within the planned excavation depth. Therefore, in order to accomplish excavations at the site, it may be necessary to dewater excavations. The dewatering system can consist of a perforated plastic pipe (in a grid array) embedded in free draining rock. The system should discharge to a sump area that is pumped continuously during construction. The general contractor is responsible for the design, operation and maintenance of the temporary dewatering system.

RECOMMENDATIONS

Seismic Design

Seismic design parameters presented below are based on Section 1613 titled "Earthquake Loads" of the 2007 California Building Code (CBC). Based on CBC Table 1613.5.2, we have determined a Site Class E should be used for the subject site. Using a site latitude and longitude of 38.435°N and -123.087°W, respectively, and the United States Geological Survey's Earthquake Ground Motion Parameter Java Application (USGS, 2007) we recommend that the following seismic design criteria be used for structures at the site.

Maximum Considered Earthquake Spectral Response Acceleration:

S_S (0.2 second period) = 1.951g
S_1 (1 second period) = 1.029g

Maximum Considered Earthquake Spectral Response Acceleration for Site Class E:

S_{MS} (0.2 second period) = 1.756g
S_{M1} (1 second period) = 2.469g

Design Spectral Response Acceleration (5% damped) for Site Class E:

S_{DS} (0.2 second period) = 1.171g
S_{D1} (1 second period) = 1.646g

Grading

Site Preparation

Areas to be developed should be cleared of vegetation and debris, including that left by the removal of obsolete structures. Trees and shrubs that will not be part of the proposed development should be removed and their primary root systems grubbed. Cleared and grubbed material should be removed from the site and disposed of in accordance with County Health Department guidelines. We did not observe septic tanks, leach lines or underground fuel tanks during our study. Any such appurtenances found during grading should be capped and sealed and/or excavated and removed from the site, respectively, in accordance with established guidelines and requirements of the County Health Department. Voids created during clearing should be backfilled with engineered fill as recommended herein.

Stripping

Areas to be graded should be stripped of the upper few inches of soil containing organic matter. Soil containing more than two percent by weight of organic matter should be considered organic. Actual stripping depth should be determined by a representative of the geotechnical engineer in the field at the time of stripping. The strippings should be removed from the site, or if suitable, stockpiled for re-use as topsoil in landscaping.

Excavations

Following initial site preparation, excavation should be performed as planned or recommended herein. Excavations extending below the proposed finished grade should be backfilled with suitable materials compacted to the requirements given below.

Within fill areas, the weak surface soils should be excavated to about two feet below the existing ground surface. The excavation of weak soils should also extend at least 12 inches below pavement subgrade (where planned excavations do not completely remove the weak soils). The excavation of weak surface materials should extend at least 3 feet beyond the edge of pavements. The excavated materials should be stockpiled for later use as compacted fill, or removed from the site, as applicable.

At all times, temporary construction excavations should conform to the regulations of the State of California, Department of Industrial Relations, Division of Industrial Safety or other stricter governing regulations. The stability of temporary cut slopes, such as those constructed during the installation of underground utilities, should be the responsibility of the contractor. Depending on the time of year when grading is performed, and the surface conditions exposed, temporary cut slopes may need to be excavated to 1½:1 or flatter. The tops of the temporary cut slopes should be rounded back to 2:1 in weak soil zones.

Fill Quality

All fill materials should be free of perishable matter and rocks or lumps over 6 inches in diameter, meet the requirements herein for select fill, and must be approved by the geotechnical engineer prior to use. We judge the on-site soils are generally suitable for use as general and select fill. The suitability of the on-site soils for use as select fill should be verified during grading. Depending on the time of year of construction, the on-site soils may be at a moisture content that is high enough to make compaction difficult. These soils should either be allowed to dry to moisture contents within 4 percent of optimum or mixed with import soils with lower moisture content.

Select Fill

Select fill should be free of organic matter, have a low expansion potential, and conform in general to the following requirements:

SIEVE SIZE	PERCENT PASSING (By Dry Weight)
6 inch	100
4 inch	90 - 100
No. 200	10 - 60

Liquid Limit - 40 Percent Maximum
Plasticity Index - 15 Percent Maximum
R-value – 20 Minimum

In general, imported fill, if needed, should be select. Material not conforming to these requirements may be suitable for use as import fill; however, it shall be the contractor's responsibility to demonstrate that the proposed material will perform in an equivalent manner. The geotechnical engineer should approve imported materials prior to use as compacted fill. The grading contractor is responsible for submitting, at least 72 hours (3 days) in advance of its intended use, samples of the proposed import materials for laboratory testing and approval by the soils engineer.

Fill Placement

The surface exposed by stripping and removal of weak surface soils should be scarified to a depth of at least 6 inches, uniformly moisture-conditioned to near optimum and compacted to at least 90 percent of the maximum dry density of the materials as determined by ASTM Test Method D-1557. Approved fill material should then be spread in thin lifts, uniformly moisture-conditioned to near optimum and properly compacted. All structural fills, including those placed to establish site surface drainage, should be compacted to at least 90 percent relative compaction. Only approved select materials should be used for fill.

Permanent Fill Slopes

In general, fill slopes should be designed and constructed at slope gradients of 2:1 (horizontal to vertical) or flatter, unless otherwise approved by the geotechnical engineer in specified areas. Where steeper slopes are required, retaining walls should be used. Fill slopes steeper than 2:1 will require the use of geogrid to increase stability. Providing recommendations for grid type and spacing was not part of our requested and/or proposed scope of work. Should the need to use geogrid arise, additional laboratory testing and stability analyses will be required. Fill slopes should be constructed by overfilling and cutting the slope to final grade. "Track walking" of a slope to achieve slope compaction is not an acceptable procedure for slope construction. The geotechnical engineer is not responsible for measuring the angles of these slopes. Denuded slopes should be planted with fast-growing, deep-rooted groundcover to reduce sloughing or erosion. The cut and fill slope inclinations recommended herein address only the stability of the slopes. It should not be inferred that they address the feasibility of landscaping and weed control. Where these are concerns, the slopes should be flattened accordingly.

Wet Weather Grading

Generally, grading is performed more economically during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring due to excessive moisture in on-site soils. Special and relatively expensive construction procedures, including dewatering of excavations and importing granular soils, should be anticipated if grading must be completed during the winter and early spring or if localized areas of soft saturated soils are found during grading in the summer and fall.

Open excavations also tend to be more unstable during wet weather as groundwater seeps towards the exposed cut slope. Severe sloughing and occasional slope failures should be anticipated. The occurrence of these events will require extensive clean up and the installation of slope protection measures, thus delaying projects. The general contractor is responsible for the performance, maintenance and repair of temporary cut slopes.

Foundation Support

Depending on the amount of acceptable settlement and the required performance of Willow Creek Road, the culverts or bridge can be supported on spread footings, a mat slab, or driven concrete or steel friction piles. Specific recommendations for each alternative are given in the following sections of the report.

Spread Footings

Spread footings should be at least 24 inches wide and should bottom on firm, natural soils or engineered fill, as applicable, at least 24 inches below lowest adjacent grade. Additional embedment or width may be needed to satisfy code and/or structural requirements.

The bottoms of all footing excavations should be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing steel and concrete. This will remove the soils disturbed during footing excavations, or restore their adequate bearing capacity, and reduce post-construction settlements. Footing excavations should not be allowed to dry before placing concrete. If shrinkage cracks appear in soils exposed in the footing excavations, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the foundation excavations should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

Bearing Pressures - Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 1200, 1800 and 2400 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

Lateral Pressures - The portion of spread footing foundations extending into firm, natural soil or select engineered fill may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.35, respectively, to resist sliding. Passive pressure should be neglected within the upper 6 inches, unless the soils are confined by concrete slabs or pavements.

Mat Slabs

Mat slabs should bottom on firm, natural soil or engineered fill. We understand that mat slabs will generally be a uniform thickness with thickened areas along the edges and at the column locations. The bottoms of excavations for thickened portions should be treated like footings and be thoroughly cleaned out or wetted and compacted using hand-operated tamping equipment prior to placing reinforcing steel and concrete. This will remove the soils disturbed during excavations. The slab excavation should not be allowed to dry before placing concrete. If shrinkage cracks appear in soils exposed in the excavation, the soil should be thoroughly moistened to close all cracks prior to concrete placement. The moisture condition of the excavation should be checked by the geotechnical engineer no more than 24 hours prior to placing concrete.

The slabs should be designed to accommodate the differential settlement described in the "Settlement" section of this report. Due to the presence of compressible and potentially liquefiable soils, we recommend using allowable bearing pressures of 1200, 1800, and 2400 pounds per square foot (psf) for dead load, dead plus live load, and total loads (including wind and seismic forces), respectively. Based on the variability of the mat slab subgrade soils and correlations in Bowles (1990), we recommend a modulus of subgrade reaction (k) of 100 pounds per cubic inch (pci) be used for design. The mat slab may impose a passive equivalent fluid pressure and a friction factor of 400 pcf and 0.40, respectively, to resist sliding.

Driven Piles

In order to reduce the impacts of total and differential settlement (both consolidation and earthquake-induced), the proposed culverts or bridge can be supported on a driven pile foundation system. There are a wide variety of pile types that could be used for the project. We typically see 12-inch square pre-cast concrete piles or 16-inch diameter steel pipe piling used for this type of project. Recommendations for both pile types are presented herein. If the project design team elects to use a different type or dimension of pile, we should be contacted for additional evaluation and recommendations.

<u>Vertical Capacity</u> - The ultimate vertical capacity of the pile depends on the skin friction developed in the underlying sand and clay soils minus the effect of soil downdrag. As the clay and silt soils, that extend from 8 to approximately 45 feet deep, compress under fill and/or structural loads they, and the soils above them, impart a negative skin friction, referred to as downdrag, on the pile. For the Willow Creek site, the soils imparting a downdrag force include all the soils above a depth of approximately 45 feet. For design purposes, these soils will impart an approximate downdrag force onto the piles of 15 kips. The piles should extend into competent soils below a depth of 45 feet. Twelve-inch diameter concrete piles and 16-inch diameter steel pipe piles extending to a depth of 70 feet below existing grade will have ultimate capacities of 47 and 50 kips, respectively. These values are an estimate of actual forces that are likely to develop and are intended for use in a working stress analysis. The values do not contain a factor of safety or load factor. The actual pile lengths and tip elevations should be established from results of an indicator pile driving program, discussed in a subsequent section. If piles other than those described herein are to be used, we should be consulted to provide revised criteria.

Under static loading conditions, we estimate total settlement of a single pile designed in accordance with these recommendations will be less than ½-inch. This value pertains to soil compression only and does not include elastic compression of the pile.

<u>Lateral Capacity</u> - We understand that lateral loads for determining load deflections (p-y) curves are not currently available. Once the project structural engineer has developed these loads, we should be consulted to provide the required p-y curves.

<u>Pile Installation</u> - The piles should be installed with a diesel hammer having a rated energy of at least 40,198 foot-pounds. This energy corresponds to a Delmag D16-32. The contractor should select a hammer and driving system that is capable of driving piles to the

desired capacity without overstressing the piles in either tension or compression. Prior to the start of pile installation at the site, the contractor should submit the following information regarding the hammer and driving system to the geotechnical engineer:

- hammer type and rated energy
- helmet weight, including striker plate
- hammer cushion material, cross-section area, and thickness
- pile cushion material and thickness, if used

This information will be used to provide driving criteria based on wave equation analysis using the proposed pile and hammer combination. The contractor should be advised that modifications to the proposed equipment, including the use of a different hammer, may be required if the analysis indicates that the proposed equipment is not sufficient to obtain the desired ultimate pile capacity, or is likely to damage the pile during driving.

Our subsurface exploration at the site did not encounter subsurface conditions that would be expected to obstruct pile driving. If obstructions are encountered, we recommend the pile locations be pre-drilled.

Abutment and Wing Walls

Abutment and wing walls constructed at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls.

Walls free to rotate (yielding greater than 0.1 percent of the wall height at the top of the backfill) should be designed for active lateral earth pressures. If walls are restrained by rigid elements to prevent rotation, they should be designed for "at rest" lateral earth pressures.

Walls should be designed to resist the following earth equivalent fluid pressures (triangular distribution):

Active Pressure (level backfill) ....40 pcf
At Rest Pressure ....70 pcf

These pressures do not consider additional loads resulting from adjacent foundations or other loads. If these additional surcharge loadings are anticipated, we can assist in evaluating their effects. Where wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to two feet of additional backfill.

Walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on, or adjacent to, the walls. Backfill against walls should be compacted to at least 90 and not more than 95 percent relative compaction. Over-compaction or the use of large compaction equipment should be avoided because increased compactive effort can result in lateral pressures higher than those recommended above.

Foundation Support

Abutment or wing walls should be supported on spread footings, mat slabs or driven piles, designed in accordance with the recommendations presented in this report. Wall foundations should be designed by the project civil or structural engineer to resist the lateral forces set forth in this section.

Wall Drainage and Backfill

Abutment and wing walls should be backdrained as shown on Plate 16, Appendix A. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in Class 2 permeable material. The pipe should be PVC Schedule 40 or ABS with SDR 35 or better, and the pipe should be sloped to drain to outlets by gravity. The top of the pipe should be at least 8 inches below lowest adjacent grade. The Class 2 permeable material should extend to within 1½ feet of the surface. The upper 1½ feet should be backfilled with compacted soil to exclude surface water. Expansive soils should not be used for wall backfill. Where expansive soils are present in the excavation made to install the abutment and wing walls, the excavation should be sloped back 1:1 from the back of the footing or pile cap. The ground surface behind walls should be sloped to drain. Where migration of moisture through the abutment and wing walls would be detrimental, retaining walls should be waterproofed.

Utility Trenches

The shoring and safety of trench excavations is solely the responsibility of the contractor. Attention is drawn to the State of California Safety Orders dealing with "Excavations and Trenches."

Unless otherwise specified by the County of Sonoma, on-site, inorganic soil may be used as general utility trench backfill. Where utility trenches support pavements, slabs and foundations, trench backfill should consist of aggregate baserock. The baserock should comply with the minimum requirements in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base. Trench backfill should be moisture-conditioned as necessary, and placed in horizontal layers not exceeding 8 inches in thickness, before compaction. Each layer should be compacted to at least 90 percent relative compaction as determined by ASTM Test Method D-1557. The top 6 inches of trench backfill below vehicle pavement subgrades should be moisture-conditioned as necessary and compacted to at least 95 percent relative compaction. Jetting or ponding of trench backfill to aid in achieving the recommended degree of compaction should not be attempted.

Pavements

Based on our study, we believe the near-surface soils will have a moderate supporting capacity, after proper compaction, when used as a pavement subgrade. However, we understand that the approaches to the crossing may be raised, which will require fill. Therefore, provided grading is performed as recommended herein, the uppermost 12-inches of pavement subgrade soils will be either on-site or imported select fill with a minimum R-value of 20. Based on this R-value we recommend the pavement sections listed in Table 2 be used.

TABLE 2
PAVEMENT SECTIONS

TI	THICKNESS (feet)		
	ASPHALT CONCRETE	CLASS 2 AGGREGATE BASE	MINIMUM ENGINEERED FILL THICKNESS*
9.0	0.45	1.30	1.0
8.0	0.40	1.15	1.0
7.0	0.30	1.05	1.0
6.0	0.25	0.85	1.0
5.0	0.20	0.70	1.0

* R-value $\geq$ 20

Pavement thicknesses were computed using Method 301 F of the Caltrans Highway Design Manual and are based on a pavement life of 20 years. These recommendations are intended to provide support represented by the indicated Traffic Indices (TI). They are not intended to provide pavement sections for heavy concentrated construction storage or wheel loads such as forklifts, parked truck-trailers and concrete trucks.

In areas where heavy construction storage and wheel loads are anticipated, the pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Alternatively, paving can be deferred until heavy construction storage and wheel loads are no longer present.

Prior to placement of aggregate base, the upper 6 inches of the pavement subgrade soils should be scarified, uniformly moisture-conditioned to near optimum, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. Aggregate base materials should be spread in thin layers, uniformly moisture-conditioned, and compacted to at least 95 percent relative compaction to form a firm, non-yielding surface. The materials and methods used should conform to the requirements of the County of Sonoma and the current edition of the Caltrans Standard Specifications, except that compaction requirements should be based on ASTM Test Method D-1557. Aggregate used for the base course should comply with the minimum requirements specified in Caltrans Standard Specifications, Section 26 for Class 2 Aggregate Base.

Wet Weather Paving

In general, the pavements should be constructed during the dry season to avoid the saturation of the subgrade and base materials, which often occurs during the wet winter months. If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas may have to be overexcavated to remove soft soils. The excavations will probably require backfilling with imported crushed (ballast) rock. The geotechnical engineer should be consulted for recommendations at the time of construction.

Geotechnical Drainage

Surface water should be diverted away from slopes, foundations and edges of pavements. Surface drainage gradients away from foundations should conform to the

2007 CBC and/or the local jurisdiction Water seepage or the spread of extensive root systems into the soil subgrade of footings, slabs or pavements could cause differential movements and consequent distress in these structural elements. Landscaping should be planned with consideration for these potential problems.

Maintenance

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently, and cleaned and maintained as necessary or at least annually. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge.

Supplemental Services

RGH Consultants, Inc. (RGH) recommends that we be retained to review the project plans and specifications to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly site excavations, compaction of fills and backfills, foundation and subdrain installations, and perform field and laboratory testing. As part of these services, we recommend that prior to construction a meeting be held at the site that includes, but is not limited to, the owner or owner's representative, the general contractor, the grading contractor, the foundation contractor, the underground contractor, any specialty contractors, the project civil engineer, other members of the project design team and RGH. This meeting should serve as a time to discuss and answer questions regarding the recommendations presented herein and to establish the coordination procedure between the contractors and RGH.

If, during construction, we observe subsurface conditions different from those encountered during the explorations, we should be allowed to amend our recommendations accordingly. If different conditions are observed by others, or appear to be present beneath excavations, RGH should be advised at once so that these conditions may be evaluated and our recommendations reviewed and updated, if warranted. The validity of recommendations made in this report is contingent upon our being notified and retained to review the changed conditions.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at, or adjacent to, the site, the recommendations made in this report may no longer be valid or appropriate. In such case, we recommend that we be retained to review this report and verify the applicability of the conclusions and recommendations or modify the same considering the time lapsed or changed conditions. The validity of recommendations made in this report is contingent upon such review.

These supplemental services are performed on an as-requested basis and are in addition to this geotechnical study. We cannot accept responsibility for items that we are not notified to observe or for changed conditions we are not allowed to review.

LIMITATIONS

This report has been prepared by RGH for the exclusive use of the Stewards of the Coast and Redwoods and their consultants as an aid in the design and construction of the proposed improved crossing of Willow Creek described in this report.

The validity of the recommendations contained in this report depends upon an adequate testing and monitoring program during the construction phase. Unless the construction monitoring and testing program is provided by our firm, we will not be held responsible for compliance with design recommendations presented in this report and other addendum submitted as part of this report.

Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided to us regarding the proposed construction, the results of our field exploration, laboratory testing program, and professional judgment. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of construction.

The test borings represent subsurface conditions at the locations and on the dates indicated. It is not warranted that they are representative of such conditions elsewhere or at other times. Site conditions and cultural features described in the text of this report are those existing at the time of our field exploration on March 3 and 5, 2008, and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or a study of the presence or absence of toxic mold and/or hazardous, toxic or corrosive materials in the soil, surface water, groundwater or air (on, below or around this site), nor did it include an evaluation or study for the presence or absence of wetlands. These studies should be conducted under separate cover, scope and fee and should be provided by a qualified expert in those fields.