GEOTECHNICAL INVESTIGATION REPORT
FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA
KLEINFELDER PROJECT NO.: 20162052.001A

DECEMBER 11, 2015

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1 INTRODUCTION

1.1 GENERAL

This report presents the results of Kleinfelder's geotechnical investigation for the proposed 10-inch-diameter recycled water pipeline crossing below Nathanson Creek in Sonoma, California. The preferred construction method for the proposed pipeline is horizontal directional drilling (HDD). The proposed crossing is located on the north side of East Watmaugh Road, approximately 400 feet west of Fifth Street East. The approximate location of the proposed crossing is shown on Plate 1, Site Location Map.

The purpose of this investigation was to evaluate the subsurface conditions near the project alignment in order to characterize the subsurface materials likely to be encountered during HDD activities and to provide conclusions and recommendations for the design of the HDD crossing.

This report includes our recommendations related to the geotechnical aspects of project planning, design, and construction of the proposed trenchless installations. Conclusions and recommendations presented in this report are based on the subsurface conditions encountered in two exploratory soil borings (KB-1 and KB-2) drilled for this investigation and our review of published geologic data referenced in this report. Recommendations presented herein should not be extrapolated to other areas or used for other projects without our prior review. The approximate locations of our exploratory borings are shown on Plate 2, Site Plan.

1.2 PROJECT DESCRIPTION

Our understanding of the project is based on multiple site visits, discussions with SCWA representatives, and the review of preliminary drawings provided by Sonoma Valley County Sanitation District (SVCSD). The proposed Nathanson Creek crossing is planned to be about 280 feet in length and a minimum of 10 feet below the creek flow line, or approximately 23 feet below the roadway. The crossing will be constructed using HDD methods. The proposed HDD

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entry and exit points are located approximately 290 feet and 570 feet west of Fifth Street East on East Watmaugh Road. The alignment trends in an east-west direction.

1.3 SCOPE OF SERVICES

As authorized by SCWA, and as outlined in our proposal dated September 8, 2015, our scope of services included the following items:

- A description of the proposed project including a site vicinity map and a site plan showing the project alignment, the locations of subsurface explorations, and the proposed HDD entry and exit points.
- A description of the site geologic setting and potentially adverse geologic hazards that could impact the project such as soil liquefaction, ground shaking and ground rupture due to earthquake activity.
- A site geology map along the proposed HDD crossing alignment depicting the anticipated geologic conditions.
- A summary of the subsurface investigative procedures, methods and a description of the surficial and subsurface site conditions encountered during our field investigation.
- Results of geotechnical laboratory testing.
- A brief discussion of the corrosion potential of the subsurface soils encountered during our field exploration based on laboratory corrosivity tests performed by others. This scope of work does not include corrosion engineering.
- Analysis of the potential for hydraulic fracturing and inadvertent fluid releases from the HDD bore.
- Recommendations related to the geotechnical aspects of HDD including:
    - Anticipated drilling conditions
    - Soil characteristics with regard to drill tooling selection and separation plant considerations
    - Drilling fluid considerations
    - Solids and fluid volume guidelines
    - Borehole and fluidic drag coefficients for pipe pullback load estimating

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- Recommendations for control of inadvertent fluid releases and related contingency planning
- Equipment support

- Recommendations for Contractor selection and pre-bid services.
- Appendices including logs of borings and laboratory test results.

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2 FIELD INVESTIGATION AND LABORATORY TESTING

2.1 SITE DESCRIPTION

The proposed HDD bore will cross beneath Nathanson Creek, just north of East Watmaugh Road in Sonoma, California. It is currently unknown which side of Nathanson Creek the bore will begin and end. However, the total distance between the proposed entry and exit points is approximately 280 feet.

The surrounding area of the site is relatively level. The elevation of the west end of the HDD bore is approximately 2-3 feet higher than the east end. A cross section of the topography along the proposed crossing is presented on Plate 3, Proposed HDD Cross Section.

2.2 FIELD EXPLORATION

Prior to drilling, the boring locations were marked in the field and Underground Service Alert of Northern California (USA-North) was contacted to inform local utilities of planned drilling operations. After the site was marked by local utilities, the exact boring locations were selected. A single drilling permit was obtained from the Sonoma County Permit and Resource Management Department (PRMD) for the boring locations.

The subsurface conditions at the site were explored on September 9, 2015 by drilling two (2) borings (KB-1 and KB-2) to depths of approximately 61½ and 60½ feet respectively below the existing ground surface. The borings were drilled using a CME-55 track-mounted drill rig equipped with mud rotary drilling capabilities. Borings KB-1 and KB-2 were drilled on the east and west sides of Nathanson Creek, respectively. The locations of borings performed for this investigation are shown on Plate 2, Site Plan.

The borings were located in the field by visual sighting and/or pacing from existing site features. Therefore, the locations of the borings shown on Plate 2 should be considered approximate and may vary slightly from those indicated.

A Kleinfelder geologist maintained logs of the borings, and visually classified pertinent engineering properties of soils encountered according to applicable portions of the Unified Soil Classification System (American Society for Testing and Materials International [ASTM] D2488

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visual-manual procedure). Both disturbed and relatively undisturbed samples of the subsurface materials were obtained in order to visually classify the soils. Sample classifications, blow counts recorded during sampling, and other related information were recorded on the boring logs and are presented in Appendix A. The Unified Soil Classification System and a key to the symbols used on the boring logs are both described on Plate A-1, and a Soil Description Key is presented on Plate A-2. Boring logs are presented on Plates A-3 and A-4.

2.3 SAMPLING PROCEDURES

Relatively undisturbed samples were obtained from the borings at selected depths by driving a 2.5-inch inside diameter (I.D.), split-barrel, California sampler containing stainless steel liners into undisturbed soil with a 140-pound automatic hammer free-falling a distance of 30 inches. The California sampler is in general conformance with ASTM D3550. Soil sampled using this method likely experienced some minor disturbance due to hammer impact, retrieval, and handling.

Disturbed samples were also obtained at selected depths by driving a 1.4-inch I.D. Standard Penetration Test (SPT) sampler into undisturbed soil with a 140-pound automatic hammer free-falling a distance of 30 inches. The SPT sampler was used in gravelly soils. The SPT sampler is in general conformance with ASTM D1586.

Blow counts were recorded at 6-inch intervals, or fraction thereof, for each driven sample attempt and are reported on the boring logs. Blow counts shown on the boring logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. Sampler size correction factors were applied to estimate the material apparent density noted on the boring logs. The consistency terminology used in soil descriptions for cohesive soils is based on field observations (see Plate A-2). Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance, and returned to our laboratory for further testing.

After the borings were completed, they were backfilled with cement grout per Sonoma County Permit and Resource Management Department requirements.

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2.4 GEOTECHNICAL LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to evaluate physical and engineering properties. The laboratory testing included the following tests:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Material Finer Than No. 200 Sieve (ASTM D1140)
- Atterberg Limits (ASTM D4318)
- Particle Size Analysis (ASTM D422)
- Unconsolidated Undrained Triaxial Compression, TXUU (ASTM D2850)

Unit weight, moisture content, percent passing the No. 200 sieve, Atterberg Limits, and unconsolidated undrained triaxial shear (TXUU) test results are summarized on the boring logs presented in Appendix A. The Summary of Laboratory Tests is presented on Plate B-1 in Appendix B. The results of other laboratory tests are included in Appendix B.

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3 SITE CONDITIONS

3.1 REGIONAL GEOLOGY

The site is located in the central Sonoma Valley, within the Coast Range Geomorphic Province of Northern California. This province is generally characterized by northwest-trending mountain ranges and intervening valleys, which are a reflection of the dominant northwest structural trend of the bedrock in the region. The basement rock in the northern portion of this province consists of the Great Valley Sequence, a Jurassic (200 to 145 million years old) volcanic ophiolite sequence with associated Jurassic to Cretaceous (200 to 65 million years old) sedimentary rocks, and the Franciscan Complex, a subduction complex of diverse groups of igneous, sedimentary, and metamorphic rocks of late Jurassic to early Tertiary age (160 to 34 million years old). The Great Valley Sequence was tectonically juxtaposed with The Franciscan Complex most likely during subduction accretion of the Franciscan, and these ancient fault boundaries are truncated by a modern right-lateral fault system that includes the San Andreas, Hayward-Rodgers Creek, and Maacama faults. The San Andreas Fault defines the westernmost boundary of the local bedrock, approximately 24.0 miles southwest of the site. In the site vicinity, the Great Valley Sequence and Franciscan Complex are unconformably overlain by Tertiary age (65 to 1.8 million years old) continental and marine sedimentary and volcanic rocks. These Tertiary age rocks are locally overlain by younger Quaternary age (1.8 million years old to present) alluvial and colluvial deposits.

3.2 PROJECT GEOLOGY

The geology of the site and vicinity has been mapped by Huffman and Armstrong (1980) and Wagner et al. (2004). Huffman and Armstrong (1980) indicate the bore alignment is underlain by Quaternary alluvial fan deposits consisting dominantly of deeply weathered, poorly sorted coarse sand and gravel. Wagner et al. (2004) shows the east and west segments of the bore to be underlain by latest Pleistocene to Holocene age (less than 30,000 years old) alluvial fan deposits comprised of gravel, sand, silt, and clay. Wagner et al. (2004) indicates the channel of Nathanson Creek is underlain by modern (less than 150 years old) stream channel deposits comprised of loose alluvial gravel, sand and silt. The portion of the geologic map by Wagner et al. (2004) that incorporates the site is presented on Plate 4, Site Geology.

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The site has also been mapped by Witter et al. (2006). This reference indicates the east and west bore segments are underlain by latest Pleistocene to Holocene age alluvial fan deposits, and the Nathanson Creek channel by modern stream channel deposits. According to Witter et al. (2006) the alluvial fan deposits are considered moderately susceptible to liquefaction, and the channel deposits very highly susceptible to liquefaction.

3.3 SEISMICITY AND FAULTING

The entire Fifth Street East Recycled Water Pipeline project is not located within an Earthquake Fault Zone, as defined by the California Geological Survey (CGS, 2010) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. The nearest zoned, active fault is the Hayward-Rodgers Creek fault, which is capable of producing a maximum earthquake magnitude of M7.3. Moderate to major earthquakes generated on the Hayward-Rodgers Creek fault, San Andreas fault and other nearby faults can be expected to cause strong ground shaking at the site.

The proximities and seismic parameters of significant faults in the vicinity of the site are listed in Table 1. For faults with multiple segmentation scenarios, we have only listed parameters for the scenario rupturing the most segments (i.e., the most severe scenario). The locations of the faults and associated parameters presented on Table 1 are based on Petersen et al. (2008). The maximum earthquake magnitudes presented in this table are based on the moment magnitude scale developed by Kanamori (1977). Felzer (2008) details calculations of California seismicity rates including correction for magnitude rounding and error, Gutenberg-Richter b value and seismicity rates.

TABLE 1
SIGNIFICANT FAULTS

Fault Name	Closest Distance to Site* (mi)	Magnitude of Characteristic Earthquake**	Slip Rate (millimeters/year)
Hayward-Rodgers Creek-HS+HN+RC	3.6	7.3	9
West Napa (unzoned)	6.4	6.7	1
San Andreas-SAS+SAP+SAN+SAO	24.0	8.1	17-24
Maacama-Garberville	26.0	7.3	9

* Closest distance to the potential rupture.

** Moment magnitude: An estimate of an earthquake's magnitude based on the seismic moment (measure of an earthquake's size utilizing rock rigidity, amount of slip, and area of rupture).

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According to Petersen et al. (2008), characterizations of the Hayward-Rodgers Creek, San Andreas, and Maacama-Garberville faults are based on the following fault rupture segments and fault rupture scenarios.

- The Hayward-Rodgers Creek fault has been characterized by three segments and six rupture scenarios plus a floating earthquake. The three segments are the Rodgers Creek fault (RC), the Hayward North (HN), and the Hayward South (HS).
- The San Andreas fault has been characterized by four segments and nine rupture scenarios, plus a floating earthquake. The four segments are Santa Cruz Mountains (SAS), North Coast (SAN), Peninsula (SAP), and Offshore (SAO).

3.4 SUBSURFACE CONDITIONS

The following descriptions provide a general summary of the subsurface conditions encountered during the field exploration program. For more detailed descriptions of the actual conditions encountered at specific boring locations, refer to the boring logs presented in Appendix A.

3.4.1 Boring KB-1 (East Side of Nathanson Creek)

Boring KB-1 was drilled on the shoulder located along the north edge of East Watmaugh road, west of Nathanson Creek. Approximately 1½ feet of fill associated with roadway construction, was encountered near the ground surface. The fill consisted of lean clay with sand and gravel. The fill was underlain by alluvium comprised of hard silt with sand and medium dense silty sand to a depth of about 14 feet below the ground surface. Alluvial deposits between depths of about 14 feet and 41 feet below the ground surface consisted of medium dense to very dense interbedded sand layers with varied silt clay and gravel content. Hard, lean to fat clay was encountered between depths of about 41 and 49 feet below the ground surface. The clay deposits were underlain by a relatively massive, very dense gravel unit containing subrounded gravel up to about 1.4 inches in diameter. The gravel deposit was encountered to the maximum depth explored in KB-1 (about 61½ feet below the ground surface).

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3.4.2 Boring KB-2 (West Side of Nathanson Creek)

Boring KB-2 was drilled on the shoulder located along the north edge of East Watmaugh Road, east of Nathanson Creek. Approximately 1½ feet of fill associated with roadway construction was encountered at the ground surface. The fill consisted of lean clay with sand and gravel. The fill was underlain by alluvium comprised of very dense sand with clay to a depth of about 5½ feet below the ground surface. A very dense alluvial gravel unit was encountered between depths of about 5½ feet and 14½ feet below the ground surface, and contained subrounded gravel up to about ¾ inches in diameter. A hard, lean to fat clay layer was encountered from about 14½ feet to about 24 feet below the ground surface. The alluvial deposits between depths of about 24 and 41 feet below the ground surface consisted of medium dense to very dense sand with varied silt and gravel content. The sand deposits were underlain by a relatively massive, very dense gravel unit containing subrounded gravel up to 2 inches in diameter. Firm to hard, lean to fat clay interlayers were encountered between depths of about 41 and 44½ feet below the ground surface, and about 54½ to 58 feet below the ground surface. The gravel deposit was encountered to the maximum depth explored in KB-2 (about 60½ feet below the ground surface).

3.5 GRAVEL PROPERTIES

In Boring KB-2, the gravel layer encountered between 5½ and 14½ feet contained dominantly highly weathered, weak clasts up to ¾ inch in diameter, supported in a sand matrix. A second gravel unit was encountered in Borings KB-1 and KB-2 below a depth of about 44½ to 49 feet. The unit is clast-supported with a minor clayey sand matrix. Gravel within this deposit was composed of basalt and rhyolite, which was slightly weathered and strong. The deposit contained gravel greater than 2.5 inches in diameter (California sampler inside diameter), indicating clasts of cobbles and possibly boulders (over 12 inches) could be present. If the HDD drill path encounters basalt or rhyolite gravel, cobbles or boulders, difficult drilling conditions can be expected. Coarse gravel units containing cobbles and boulders should be avoided along the bore path.

3.6 GROUNDWATER CONDITIONS

Groundwater level was measured during our field investigation by means of using solid flight auger drilling techniques until groundwater was encountered, then switching to rotary wash

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methods. Groundwater was encountered at a depth of approximately 15 feet below the ground surface in Boring KB-1.

Groundwater and soil moisture conditions within the area will vary heavily depending on rainfall, Nathanson Creek stream stages, irrigation practices, well pumping, and other factors that may not have been apparent during drilling of the borings. Recent groundwater data[1] provided by the California Department of Water Resources of a nearby well indicate groundwater depths that seasonally fluctuate between about 5 to 30 feet below the ground surface.

3.7 SOIL CORROSIVITY TESTING

A sample of the near-surface soil from each boring was subjected to chemical analysis for the purpose of corrosion assessment. The sample was tested for pH, resistivity, soluble chlorides and soluble sulfates, by Sunland Analytical of Rancho Cordova, California. This sample was tested in general accordance with Caltrans Test Methods 643 Mod., 422, and 417 for pH, resistivity, soluble chlorides, and soluble sulfates, respectively. The test results are tabulated in Table 2, and are presented in Appendix C

TABLE 2
SUMMARY OF CORROSION TEST RESULTS

Boring No.	Sample Depth (feet)	Resistivity (Ohm-Cm)	pH	Water Soluble Chlorides (ppm)	Water Soluble Sulfates (ppm)
KB-1	5.5-6.5	600	7.38	299.3	51.8
KB-2	10.5-11.0	3,220	7.79	27.6	6.6

Based on the Caltrans Corrosion Guidelines (2012 version 2.0), a site is considered corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is 500 ppm or greater, sulfate concentration is 2000 ppm or greater, or the pH is 5.5 or less. A minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion.

- Based on the above results and the Caltrans Corrosion Guidelines, the tested sample from Boring KB-1 is considered corrosive to steel based on resistivity.

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According to ACI 318 Section 4.3, Table 4.3.1, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type). A water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete.

- Based on these guidelines and the above test results, the sample tested is considered non-corrosive to concrete.

Kleinfelder had laboratory testing performed to provide data regarding corrosivity of on-site soils. The above corrosivity results are preliminary and are an indicator of potential soil corrosivity for the samples tested. Other soils found on the site may be more, less, or of a similar corrosive nature. Our scope of services does not include corrosion engineering, and therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services if requested.

[1] *http://www.water.ca.gov/waterdatalibrary/groundwater/hydrographs/brr_hydro.cfm?CFGRIDKEY=7477*

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4 CONCLUSIONS AND DESIGN CONSIDERATIONS

4.1 GENERAL CONCLUSIONS

Based on the results of our field investigation, laboratory testing, and review of surface topography, it is our professional opinion that a proposed HDD crossing is technically feasible. However, after reviewing the project drawings (90% submittal, dated 11/19/14) Kleinfelder recommends altering the design approach to increase the likelihood of a successful HDD installation. Kleinfelder can assist in the design of the HDD bore path, if requested. Alternatively, a performance specification could be drafted for the HDD installation. The performance specification would rely upon the HDD contractor to determine the optimum geometry and profile of the HDD alignment. Kleinfelder can assist with the development of this specification or review the contractor's design submittal, if requested. In general, compound curves (horizontal and vertical at the same time) are difficult to achieve and can result in steering misalignment issues. Additionally, due to the potential for cobble and gravel along the bore path, a mud motor bit may be necessary to complete the HDD installation. If a mud motor bit is necessary the minimum bend radius will need to be about 1,000 to 1,200 feet. These issues will affect the alignment as well as the length of the bore and pipe laydown area. In addition to the above issues, there are a number of conditions that could affect pipeline design and/or cause difficulties or delays for HDD construction. These include:

- Difficult drilling and reaming in gravel with cobbles and boulders
- The presence of reactive/swelling clay soils that are difficult to remove from drilling fluids and may cause restrictions in the HDD bore hole
- Difficult jetting in dense soil or gravel. A mud motor may be required
- Borehole instability in gravel layers
- Difficult steering due to horizontal layering and variation in soil density/consistency
- Ground settlement due to soil liquefaction from an earthquake

Conclusions and recommendations regarding HDD design and construction are further discussed below. It is recommended that the proposed HDD crossing be designed and constructed in general accordance with the third edition of the "Horizontal Directional Drilling Good Practices Guidelines" by the HDD Consortium (Bennett, 2008).

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4.2 LIQUEFACTION

Liquefaction describes a condition in which saturated soil loses shear strength and deforms as a result of increased pore water pressure induced by strong ground shaking during an earthquake. Dissipation of the excess pore water pressures will produce volume changes within the liquefied soil layer, which causes settlement. Factors known to influence liquefaction include soil type, structure, grain size, relative density, confining pressure, depth to groundwater and the intensity and duration of ground shaking. Soils most susceptible to liquefaction are saturated, loose sandy soils, and low plasticity clays and silts. If liquefaction occurs, pipelines above the liquefiable layers may undergo settlement. Within liquefiable soils, a pipeline can become buoyant, or loose support and settle if it is not buoyant. The degree of buoyancy or settlement will be affected by the fines content of the soil. More fines generally result in less susceptibility to buoyancy and settlement due to the residual soil strength that may be present.

Liquefaction susceptibility of the soils encountered within Borings KB-1 and KB-2 were evaluated using methodologies proposed by Youd et al. (2001), Seed et al (2003), Idriss & Boulanger (2008), and Cetin et al. (2009). Within Boring KB-2, the granular soil encountered was adequately dense and the fine-grained soil encountered was plastic and stiff enough to consider the liquefaction potential low within the soils encountered.

Within Boring KB-1 a few isolated soil deposits were encountered below the design groundwater depth of 10 feet and above 20 feet that exhibited potentially liquefiable characteristics. Depended on the methodology, the estimated ground surface settlement estimated using the Boring KB-1 profile is approximately ½ inch to 1¼ inches.

Based on observed conditions at the site, potentially liquefiable layers that may be present at this site appear to be discontinuous and sporadic. The settlement estimates are relatively small. However, the pipeline crossing should be designed to tolerate differential settlement of about 1 inch in 100 feet.

4.3 ANTICIPATED DRILLING CONDITIONS

The density and consistency of soils encountered in our exploratory borings were highly variable. In general, the near-surface soil conditions encountered in our borings consisted of medium dense to very dense sands with varying amounts of gravels and fines, underlined by

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hard lean clays, and fat clays interbedded with medium dense to very dense clayey, well-graded and, poorly-graded gravels extending to the maximum depths explored in Borings KB-1 and KB-2.

Depending upon drill rig and mud pump capacity the harder clayey soils and dense sands and gravels may be difficult to excavate by jetting and a mud motor may be required. In addition, the fine clay soils may be reactive and prone to swelling in the bore hole.

The contractor should carefully evaluate the ground conditions identified in this report before selecting drilling equipment and tooling.

4.4 STEERING

As stated above, the density and consistency of soils encountered in the exploratory borings were variable. The soil units within the proposed bore path appear to consist of interlayered firm to very hard clays, and medium dense to very dense, well-graded and poorly-graded sands and gravels.

The proposed bore path is anticipated to penetrate medium to very dense sands and gravels, and hard clays. These variations in density/consistency may cause some difficulty steering along the bore path at these transitions. Where a mud motor is used, the minimum radius of the bends in the HDD bore path should be about 1,000 to 1,200 feet or greater, since the mud motor drill cannot steer on a tighter radius. This would change the proposed bore path profile that was provided to us and shown on Plate 3.

4.5 HDD BOREHOLE INSTABILITY

Layers of well-graded gravel with sand and silt or clay, and clayey gravel were encountered at depths of about 14 to 41 feet in Boring KB-1 and from the ground surface to depths of about 14½ feet, 29 to 41½ feet and in Boring KB-2 extending to the maximum depths explored. The well-graded gravels between depths of about 18 and 27½ feet in Boring KB-1 and 29½ to 38½ feet and 58 to 61 feet in Boring KB-2 may be prone to instability in the HDD borehole. The clayey gravels should be more stable but gravel or cobble clasts dislodged from the borehole walls could be an issue during drilling and removal of cuttings. Proper drilling fluid makeup or

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use of conductor casings can reduce the potential for borehole caving and stuck pipe during pullback.

4.6 INADVERTENT RETURNS OF DRILLING FLUID

4.6.1 General

Hydraulic fracturing occurs when borehole pressure causes plastic deformation of the soil surrounding the borehole, initiating and propagating fractures in the soil mass. The resistance to plastic deformation and fracturing is a function of soil strength, overburden pressure, and pore water pressure. Hydraulic fracturing can result in drilling fluid inadvertently returning to the ground surface or running horizontally away from the borehole. Allowable borehole pressure was evaluated using the Delft Geotechnics equation as published in “Recommended Guidelines for Installation of Pipelines Beneath Levees Using Horizontal Directional Drilling, Appendix B, CPAR-GL-98-1,” published by the Pipeline Research Council International (PRCI) and the U.S. Army Corps of Engineers, dated April 1998. The estimated allowable borehole pressure was compared to predicted borehole pressure in our analyses.

For hydraulic fracturing analysis, we assumed a pilot-hole diameter of 6 inches, a drill rod diameter of 3.25 inches, and a mud pump output of up to 220 gallons per minute. Target up-hole fluid velocities in the analyses range from about 200 to 225 feet per minute. The drilling fluid density was estimated to be about 10 to 12 pounds per gallon. Since the proposed bore path provided to us is not buildable with HDD, we have reviewed similar bore analyses we have performed previously to make preliminary recommendations for the bore depth and contingency planning near the exit point. Changes in the bore path, drilling fluid properties and drilling equipment affect the analysis results.

Borehole instability issues and/or the contactor not maintaining a clean borehole can result in poor drilling returns and partial or complete plugging of the borehole. This will result in higher fluid pressures within the bore and can lead to hydraulic fracturing and inadvertent fluid returns to the ground surface. Depending on the contractor’s means and methods, a minimum cover depth of 25 feet below the creek bottom should be maintained to minimize the risk of hydraulic fracturing where a relatively clean borehole is maintained with good drilling returns. A deeper bore path, 25 feet or greater below the bottom of the creek, should be considered if the contractor determines that borehole plugging and/or poor drilling returns are likely.

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At depths less than about 10 feet near the exit point, hydraulic fracturing is likely and is expected to occur as the drill bit approaches the ground surface. Provisions should be in place to mitigate the effects of hydraulic fracturing and inadvertent fluid returns near the exit point. A contingency plan to mitigate these effects of hydraulic fracturing near the bore path exit point can include countermeasures such as, plastic sheeting, earthen berms, straw bales or waddles, etc., which are commonly used to provide containment for inadvertent drilling fluid releases to the surface.

4.6.2 Loss of Drilling Fluid Returns

Loss of drilling fluid returns typically occurs when the drill bit encounters fractures in large interstitial pore spaces in coarse materials (i.e., coarse sands and gravels). Loss of returns is recognized by a decrease of drilling fluid returns, or a drop in drilling fluid pressure.

If fractures or interstitial pore spaces are small or discontinuous, they may fill with solids contained in the drilling fluid returns as drilling progresses beyond them. Once the fractures or pore spaces are filled, fluid will return up the bore hole again and fluid pressure will increase until another fracture is encountered. If fractures are continuous to the surface, drilling fluid may inadvertently return to the surface.

Based on the soil conditions encountered in the borings, drilling fluid losses are not anticipated. No fluid losses were observed during the geotechnical explorations.

4.7 DRILLING FLUID PROGRAM

4.7.1 General

The drilling contractor should develop a Drilling Fluid Program (DFP) as part of the HDD Bore Plan. A properly designed drilling fluid program can substantially reduce losses due to hydraulic fracturing out, stuck product pipe, or loss of tooling. The drilling fluid program should take into account anticipated soil conditions, fluid selection, drill bit and reamer selection, and volume calculations.

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4.7.2 Borehole Slurry Density

The density of the slurry in the borehole directly affects the buoyancy force and therefore the normal force between the pipe and the wall of the borehole. The density of drilling returns is a function of ground conditions, penetration rate, mud flow rate, drilling fluid composition, and efficiency of the mud cleaning system. In general, drilling return density assuming good HDD practices is expected to be about 10 to 12 pounds per gallon for the soil units encountered at the site.

4.7.3 Soil Conditions

For the purpose of drilling fluid design, earth materials are generally divided into two categories: Inert, including sands; and reactive, including clays. Information regarding subsurface conditions likely to be encountered at the site is provided in the Subsurface Conditions section of this report as well as in the boring logs contained in Appendix A.

4.7.4 Drilling Fluid Selection

The base drilling fluid should be designed for site specific soil conditions. The base fluid may consist of either a bentonite or polymer and water, with additives to achieve specific fluid properties.

In reactive clay soils the use of PHPA polymers to inhibit swelling and wetting agents to reduce stickiness may prove beneficial. Salt (chloride) and calcium are detrimental to base fluid performance and should not be present in make-up water.

The drilling contractor should submit a base fluid design with a list of additives and loss of circulation materials that may be used on the project and MSDS sheets for approval at least two weeks prior to mobilization. Assistance with drilling fluid selection can be obtained from reputable drilling fluid suppliers.

4.7.5 Drill Bit and Reamer Selection

Drill bits and reamers should be selected based on anticipated subsurface conditions and past experience. The drilling contractor should be prepared with a variety of bits and reamers that have worked well in similar soil conditions. The use of mud motors should be considered in hard

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clay or dense gravel with Standard Penetration Test blow counts exceeding about 50 blows per foot and California sampler blow counts exceeding about 75 blows per foot.

4.7.6 Solids Separation Plant

Clays are generally the most difficult to remove from drilling fluids. Depending on their extent, the presence of these soils along the proposed bore paths may require use of desilters/centrifuge equipment in order to remove the fine soils from the drilling fluids.

4.7.7 Soil and Fluid Volume

The volume of soil or rock to be removed can be estimated as follows:

$$\frac{(\text{Hole Diameter in Inches})^2}{25} = \text{Volume in Gallons per Foot}$$

Sufficient fluid should be pumped during drilling and reaming operations to maintain flow. Drilling rates and drilling fluid flow rates may be adjusted in the field to match varying site conditions. However, an estimate of drilling fluid demand is useful when sizing drilling equipment, mud pumps, and solids removal systems, and can be particularly helpful in determining realistic drilling rates. Drilling fluid demand can be estimated based on the borehole volume and the ratios presented in Table 3 below.

TABLE 3
DRILLING FLUID DEMAND

Soil or Rock Type	Ratio of Fluid Volume to Soil Volume
Sand, Gravel	1:1
Sand and Gravel mixed with Clay	2:1
Clay or Reactive Shale	3-5:1

Drilling rates can be estimated based on the drilling fluid demand and the pump output at the design base fluid viscosity.

4.8 FLUIDIC DRAG COEFFICIENT

A fluidic drag coefficient of 0.050 psi (345 Pa) was recommended in the original Pipeline Research Council International (PRCI) design guidelines and is still routinely used by pipeline

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designers. Recently it has been suggested that the coefficient could be decreased to 0.025 psi (172 Pa) for a stable borehole with good solids removal (Puckett 2003). The higher value (0.050 psi) is recommended for routine calculations. The lower value (0.025 psi) may be appropriate for long bores in stable formations where significant cost savings could be realized by using a lower grade of steel or thinner pipe wall.

The potential for borehole instability exists in the well-graded gravel layers mentioned previously in this report. Although not apparently encountered in Borings KB-1 and KB-2, it is likely that there are poorly-graded sand and poorly-graded gravel layers that will be encountered along the proposed bore path. If caving sand or gravel occurs during installation, there is the likelihood of increased pullback loads. Good drilling and fluid management practices can reduce risk of increased pullback loads. If the pipe becomes stuck, the maximum capacity of the drill rig may be exerted on the pipe. Accordingly, the pipe should be designed to withstand the maximum anticipated rig capacity, or a maximum pullback load should be specified.

4.9 BOREHOLE FRICTION FACTOR AND ABRASION

A large portion of the pullback load is generated from friction between the pipe and the wall of the borehole. The pipe rubs against the borehole as it goes around bends and is pushed against the top of the borehole by buoyancy and capstan forces. The friction factor is an expression of the ratio of the normal force between the pipe and the borehole wall, and the axial force needed to drag the pipe along the wall. The PRCI Guidelines recommend friction factors of 0.2 to 0.3 for steel pipe. ASTM Standard F1962-99 recommends a friction factor of 0.3. Sand and gravel was encountered within the proposed bore path, therefore an abrasion-resistant coating is recommended for steel pipes. Recommended friction factors for abrasion-resistant polymer concrete coating were not found in the above literature. The coating material is similar in texture to smooth, formed concrete. NAVFAC DM 7.2[2], Chapter 3, Table 1 reports friction factors for formed concrete against various soils types as presented in Table 4 below. The friction factors reported below do not account for the presence of a drilling fluid filter cake.

[2] *Design Manual 7.2, Naval Facilities Engineering Command, May 1982*

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TABLE 4
ULTIMATE FRICTION FACTORS

Interface Material	Friction Factor (tanδ)	Friction angle δ (deg.)
Clean gravel, sandy gravel, coarse sand	0.55 to 0.60	29 to 31
Clean fine-to-medium sand, silty medium-to-coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine-to-medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, non-plastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or pre-consolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay, and silty clay	0.30 to 0.35	17 to 19

4.10 DRILL PAD SUPPORT

Surface soils in the vicinity of our exploratory borings consist of hard clay and silt and dense sand and are likely to provide adequate support for HDD drilling equipment. However, when these soils become wet they may be unstable if saturated. Soil stabilization may be required to provide a stable platform for the HDD drill rig and surrounding area depending on the time of year the installation occurs. A potential stabilization option can be use of a gravel surface course underlain by a geotextile where heavy truck and equipment traffic is planned. The contractor should perform a pre-bid site visit to evaluate support conditions for their equipment.

4.11 UTILITIES AND WELL CLEARANCE

The location of existing utilities and water wells is beyond the scope of this report. Nearby underground utilities and wells may exist and must be located and protected to prevent being impacted by underground construction. The HDD bore profile should be designed to allow sufficient clearance from underground utilities and wells. In general, we recommend wells be located at least 100 feet from the HDD bore path for this type of HDD installation.

4.12 CONTRACTOR SELECTION

The success of the project will be substantially dependent on the experience and performance of the specialty contractor retained to perform the work. We recommend the use of a specialty contractor with a minimum of 3 years construction experience in the field of horizontal directional drilling in similar drilling conditions on projects of similar scope (i.e., diameter, length, and depth). The contractor should be familiar with the use of drilling mud and additives,

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washover and conductor casings and should provide examples of projects they have successfully completed installing similar utilities in similar conditions.

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5 ADDITIONAL SERVICES

5.1 PROJECT BID DOCUMENTS

It has been our experience that contractors bidding on the project often contact us to discuss the geotechnical aspects of the project. Informal contacts between Kleinfelder and an individual contractor could result in misleading or incomplete information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project owner or his designated representative. After consultation with Kleinfelder, the project owner (or his representative) should provide clarifications or additional information to all contractors bidding the job.

5.2 EXCECUTION PLAN AND PERMIT ASSISTANCE

In order to facilitate best management practices and obtaining the required permits for HDD, a project execution plan should be developed prior to construction. The plan should include layout of equipment, MSDS sheets for all proposed drilling fluids and additives, development of a drilling fluid containment and contingency plan in case of inadvertent fluid returns, and discussion of any other site specific constraints relative to the project.

5.3 CONSTRUCTION OBSERVATION AND TESTING

It is recommended that all HDD construction be monitored by a representative from Kleinfelder, including the pilot hole boring and pipe pullback and installation. The purpose of these services is to observe the soil and drill mud conditions encountered during construction, evaluate the applicability of the recommendations presented in this report to the soil conditions encountered, and recommend appropriate changes to the owner in design or construction procedures if conditions differ from those described herein.

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6 LIMITATIONS

This report presents information for planning, permitting, design, and construction of a horizontal directional drill (HDD) crossing for the Fifth Street East Recycled Water Pipeline in Sonoma, California. It is expected that information contained in this report will be used to help other design professionals select the ultimate location, length, and depth of the proposed boring. However, this report should not be used to define site conditions for contractual purposes and Kleinfelder will accept no liability for changed conditions claims based on this report.

Recommendations contained in this report are based on materials encountered in Borings KB-1 and KB-2, evaluation of existing geotechnical data, geologic interpretation based on published articles and geotechnical data, and our present knowledge of the proposed construction.

It is possible that soil conditions could vary between or beyond the points explored, especially if the originally planned HDD entry and exit points need to be moved during design. If the scope of the proposed construction, including the proposed alignment location, changes from that described in this report, we should be notified immediately in order that a review may be made and any supplemental evaluation and/or recommendations can be provided.

We have prepared this report in accordance with the generally accepted geotechnical engineering practice as it exists in the site area at the time of our study. No warranty is expressed or implied.

This report may be used only by the Sonoma Valley County Sanitation District (SVCSD) and only for the purposes stated, and within 2 years of its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

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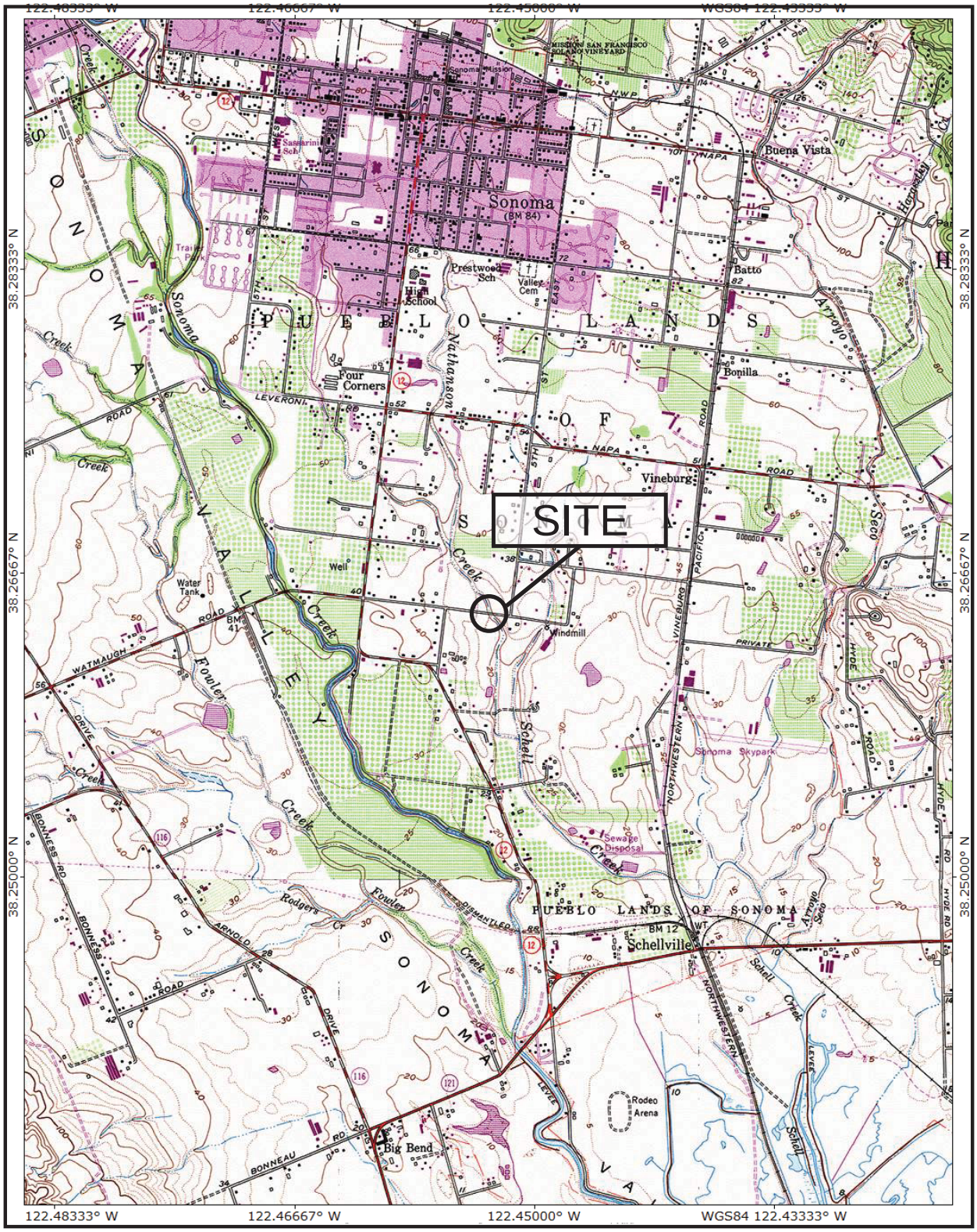
Plates



Reference: National Geographic Holdings, 2000

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3000 1500 0 3000

APPROXIMATE SCALE (feet)

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Santa Rosa:

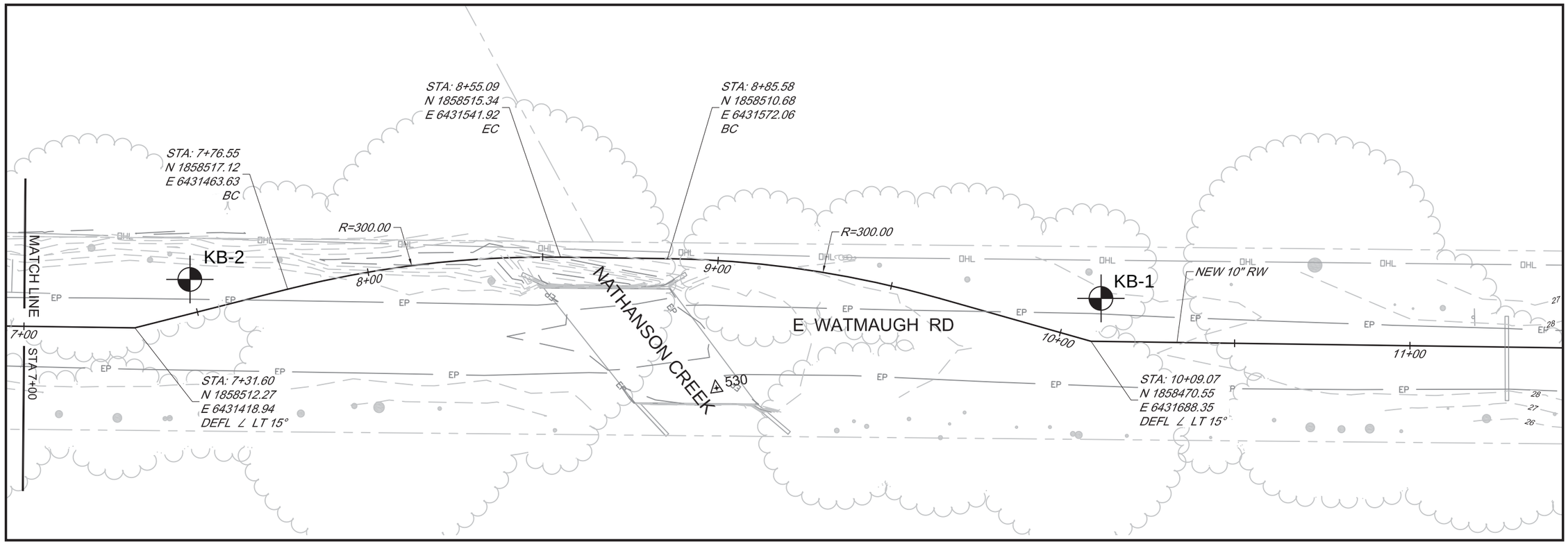
PROJECT NO.	20162052
DRAWN	SEP 2015
DRAWN BY	JCR
CHECKED BY	WVM
FILE NAME	Site Location.ai

SITE LOCATION MAP

FIFTH STREET EAST
RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CA

PLATE

1



SYMBOLS

KB-2	Approximate Boring Location (Kleinfelder, 2015)

N

30 15 0 30

APPROXIMATE SCALE (feet)

Reference: SVCSD, Dated: 11/19/14

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DRAWN	SEP 2015
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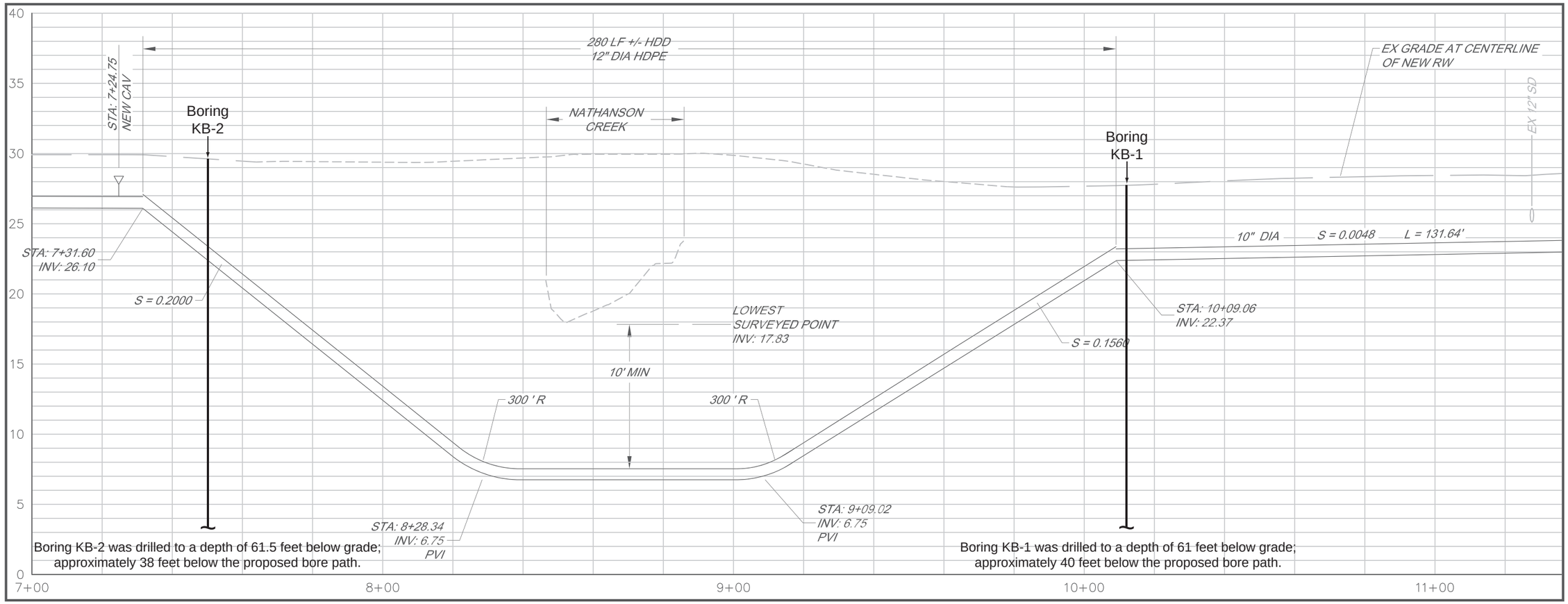
CHECKED BY	MJP
FILE NAME	Site Plan.ai

SITE PLAN

FIFTH STREET EAST
RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CA

PLATE 2



Reference: SVCSD, Dated: 11/19/14

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30 15 0 30

APPROXIMATE HORIZONTAL SCALE: 1”=30’
APPROXIMATE VERTICAL SCALE: 1”=7.5’

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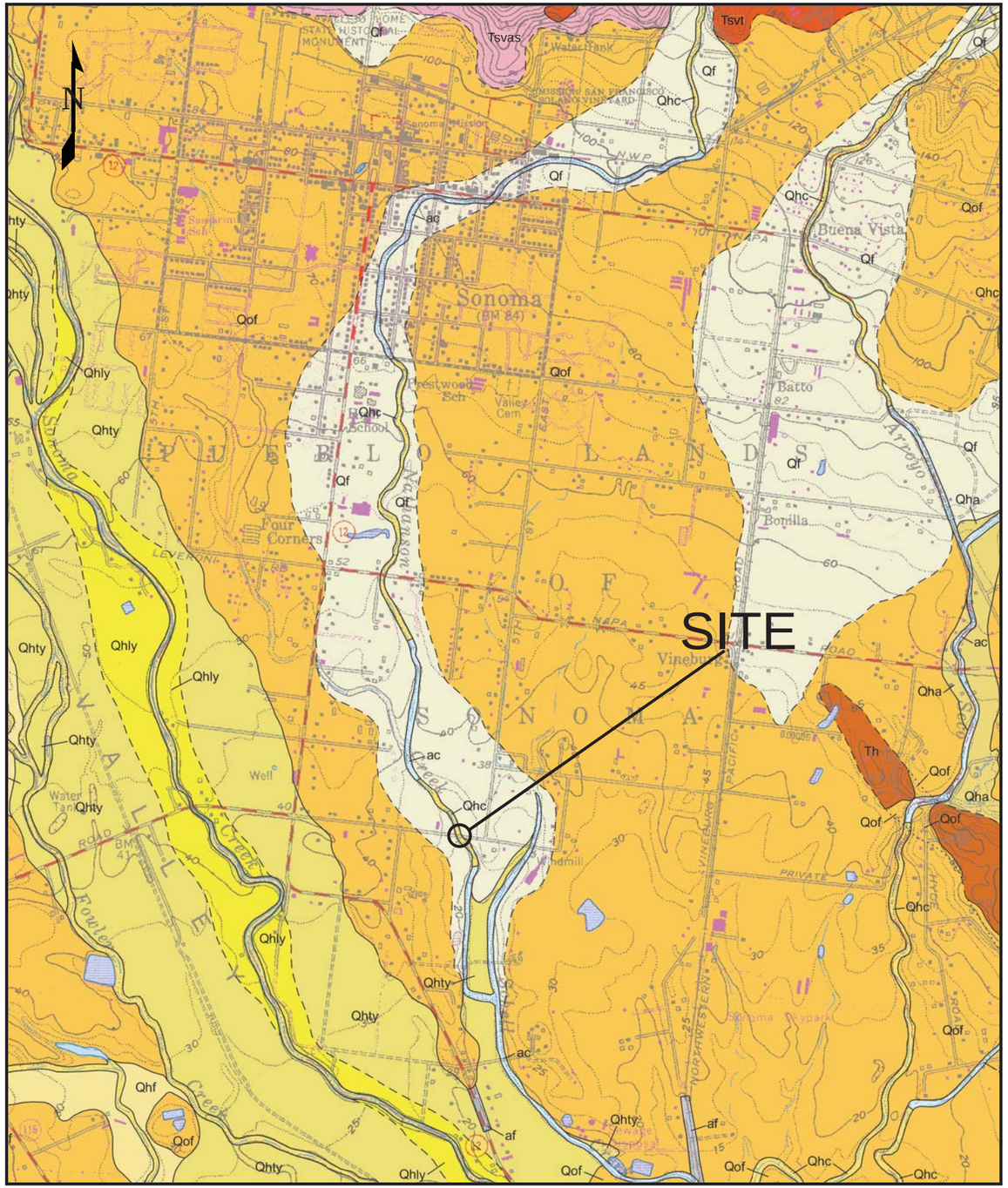
FILE NAME	Cross Section.ai

PROPOSED HDD CROSS SECTION

FIFTH STREET EAST
RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CA

PLATE
3



Unit Explanation

af	Artificial fill (Holocene, historic) - May be engineered and/or non-engineered.
Qhc	Modern stream channel deposits (Holocene <150 years) - Deposits in active, natural stream channels; consists of loose alluvial sand, gravel, and silt.
Qhty	Stream terrace deposits (latest Holocene <1,000 years) - Stream terraces deposited as point bar and overbank deposits along Somona Creek; composed of moderately sorted clayey sand and sandy clay with gravel.
Qhly	Levee deposits (Holocene <1,000 years).
Qf	Alluvial fan deposits (latest Pleistocene <~30,000 years to Holocene) - Sand, gravel, silt and clay mapped on gently sloping, fan-shaped, relatively undissected alluvial surfaces.
Qof	Alluvial deposits (early to late Pleistocene) - Sandy gravel, silt, and clay.
Th	Huichica Formation - (early Pleistocene and Pliocene) - Gravel, sand, silt, reworked tuff, clay and conglomerate. Clasts are mostly Sonoma Volcanics but coarse sand is rich in Franciscan pebbles and cobbles.
Tsvas	Andesite of Schocken Hill - Gray, aphyric andesite lava flows; interbedded with tuff. A fission track age of 7.9 +/- 0.8 Ma was reported by Fox and others (1985).
Tsvt	Light colored tuff, lithic rich in places. Locally includes tuffaceous, diatomaceous lacustrine sediments. Includes the tuff of Mt. Pisgah (Tsvtp) and the tuff of Lovall Valley (Tsvtl) along the south fork of Agua Caliente Creek.

Symbol Explanation

Contact between map units - Solid where accurately located, dashed where approximately located, short dash where inferred, dotted where concealed.

Reference: Wagner et al. (2004)

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2,000 1,000 0 2,000
APPROXIMATE SCALE (feet)

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SITE GEOLOGY

FIFTH STREET EAST
RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CA

PLATE
4

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Appendix A

SAMPLE/SAMPLER TYPE GRAPHICS

CALIFORNIA SAMPLER
(3 in. (76.2 mm.) outer diameter)

STANDARD PENETRATION SPLIT SPOON SAMPLER
(2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)

GROUND WATER GRAPHICS

- WATER LEVEL (level where first observed)
- WATER LEVEL (level after exploration completion)
- WATER LEVEL (additional levels after exploration)
- OBSERVED SEEPAGE

NOTES

- The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.
- Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown.
- No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.
- Logs represent general soil or rock conditions observed at the point of exploration on the date indicated.
- In general, Unified Soil Classification System designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.
- Fine grained soils that plot within the hatched area on the Plasticity Chart, and coarse grained soils with between 5% and 12% passing the No. 200 sieve require dual USCS symbols, ie., GW-GM, GP-GM, GW-GC, GP-GC, GC-GM, SW-SM, SP-SM, SW-SC, SP-SC, SC-SM.
- If sampler is not able to be driven at least 6 inches then 50/X indicates number of blows required to drive the identified sampler X inches with a 140 pound hammer falling 30 inches.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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DATE:
REVISED: -

GRAPHICS KEY

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-1

PLOTTED: 09/25/2015 09:15 AM BY: AMay
gINT FILE: PROJECTWISE: Nathanson Creek Crossing.gpj
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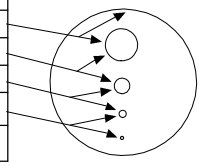
GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #10	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller

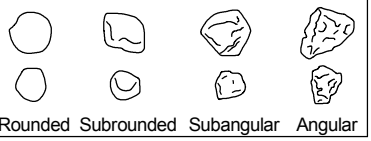


Munsell Color

NAME	ABBR
Red	R
Yellow Red	YR
Yellow	Y
Green Yellow	GY
Green	G
Blue Green	BG
Blue	B
Purple Blue	PB
Purple	P
Red Purple	RP
Black	N

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges



Particles Present

Amount	Percentage
trace	<5
few	5-10
little	15-25
some	30-45
and	50
mostly	50-100

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump or thread crumbles when drier than the plastic limit
High (H)	> 50	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump or thread can be formed without crumbling when drier than the plastic limit

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

NOTE: AFTER TERZAGHI AND PECK, 1948

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (q_u)(psf)	CRITERIA
Very Soft	< 1000	Thumb will penetrate soil more than 1 in. (25 mm.)
Soft	1000 - 2000	Thumb will penetrate soil about 1 in. (25 mm.)
Firm	2000 - 4000	Thumb will indent soil about 1/4-in. (6 mm.)
Hard	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail
Very Hard	> 8000	Thumbnail will not indent soil

STRUCTURE

DESCRIPTION	CRITERIA
Stratified	Alternating layers of varying material or color with layers at least 1/4-in. thick, note thickness
Laminated	Alternating layers of varying material or color with the layer less than 1/4-in. thick, note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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SOIL DESCRIPTION KEY

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-2

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Date Begin - End:	9/09/2015	**Drilling Company:**	Taber Drilling		**BORING LOG KB-1**
Logged By:	C. Ewing	**Drill Crew:**	Andy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary		
Weather:	Hot	**Exploration Diameter:**	4.5 in. O.D.		

FIELD EXPLORATION — Surface Condition: Road Shoulder

Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	Surface: **Lean CLAY with Sand and Gravel** dry (Road Shoulder)											
	SILT with Sand (ML): low plasticity, light brownish gray, dry to moist, hard, medium to fine grained sand, subrounded gravel to 0.75"											
5			BC=11 23 29	89%		24.0	101.3	100	78			
	Silty SAND (SM): non-plastic to low plasticity, dark brown to dark reddish brown, moist, medium dense, coarse to fine grained sand, subrounded to rounded gravel to 0.50", (decreasing fines content with depth)											
10			BC=5 9 18	83%	SM			99	14			
	Poorly-graded SAND with Silt and Gravel (SP-SM): non-plastic, brown, wet, dense, coarse to fine grained sand, subangular to subrounded gravel to 1.0"											
15			BC=7 15 21	94%	SP-SM			59	8.7			
	brown to dark brown, subangular to subrounded gravel to 0.75"		BC=10 14 23	78%								Switch to mud rotary at 18'
20	No Recovery, subangular to subrounded gravel in cuttings to 0.75"		BC=13 14 10	NR								
	Clayey SAND with Gravel (SC): non-plastic to low plasticity, grayish brown with occasional reddish brown, wet, very dense, medium to fine grained sand, occasional gravel fragments											
25			BC=15 29 50/6"	NR					26			
	Well-graded SAND with Silt and Gravel (SW-SM): grayish brown, wet, very dense, coarse to fine grained sand, subrounded to rounded gravel to 0.75"											
30			BC=21 39 50/6"	67%	SW-SM			55	5.7			

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CHECKED BY:	JCR
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BORING LOG KB-1

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-3

PLOTTED: 09/25/2015 10:46 AM BY: Mpucci
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gINT TEMPLATE: PROJECTWISE: KLF_STANDARD_GINT_LIBRARY_2015.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End:	9/09/2015	**Drilling Company:**	Taber Drilling		**BORING LOG KB-1**
Logged By:	C. Ewing	**Drill Crew:**	Andy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary		
Weather:	Hot	**Exploration Diameter:**	4.5 in. O.D.		

FIELD EXPLORATION — Surface Condition: Road Shoulder

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Well-graded SAND with Silt and Gravel (SW-SM): (continued) grayish brown, wet, very dense, coarse to fine grained sand, subrounded to rounded gravel to 0.75"		BC=19 50/5"	55%								
40				BC=9 11 18	67%								
		Lean/Fat CLAY (CL-CH): medium to high plasticity, olive yellow, moist, hard, coarse to fine grained sand, subrounded gravel to 0.5"											
45		**Lean CLAY (CL)**: mottled olive yellowish and brownish gray, moist, hard, coarse to fine grained sand		BC=10 8 12	2.75%								
		Poorly graded GRAVEL with Clay and Sand (GP-GC): medium plasticity, grayish brown, moist to wet, very dense, coarse to fine grained sand, subrounded gravel to 1.0"											
50				BC=22 32 50/6"	61%								
55		becomes wet, subrounded gravel to 1.4", increase in fines content		BC=24 42 50/3"	53%								
60				BC=31 22 23	78%								
65													

The boring was terminated at approximately 61.5 ft. below ground surface. The exploration was backfilled with Neat Cement Grout on September 09, 2015.

GROUNDWATER LEVEL INFORMATION:
▽ Groundwater was observed at approximately 15 ft. below ground surface during drilling.
GENERAL NOTES:

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BORING LOG KB-1

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-3

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gINT TEMPLATE: PROJECTWISE: KLF_STANDARD_GINT_LIBRARY_2015.GLB [KLF_BORING/TEST PIT SOIL LOG]

Date Begin - End:	9/09/2015	**Drilling Company:**	Taber Drilling		**BORING LOG KB-2**
Logged By:	C. Ewing	**Drill Crew:**	Andy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary		
Weather:	Hot	**Exploration Diameter:**	4.5 in. O.D.		

FIELD EXPLORATION — Surface Condition: Road Shoulder

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Surface: **Lean CLAY with Sand and Gravel** dry (Road Shoulder)											
		Poorly graded SAND with Clay (SP-SC): low plasticity, light grayish brown, dry, very dense, medium to fine grained sand											
5				BC=27 33 44	83%								
		Poorly graded GRAVEL with Sand (GP): brown to grayish brown, dry, very dense, coarse to fine grained sand, subangular to subrounded gravel to 0.75"											
		Poorly graded GRAVEL with Clay and Sand (GP-GC): mottled dark brown and light grayish brown, dry to moist, dry to moist, dense											
10				BC=16 24 28	89%								Switch to mud rotary at 10'
15		**Lean/Fat CLAY (CL-CH)**: medium to high plasticity, brown to grayish brown, moist, hard, fine grained sand		BC=8 16 23 PP=4.5+	83%		28.7	87.4					
20		becomes dark brown, trace coarse to fine sand, rootlets		BC=7 10 14 PP=4.5+	100%		26.4	97.5					TXUU = 3.4 ksf
		Clayey SAND (SC): non-plastic to low plasticity, grayish brown, moist, dense, coarse to fine grained sand, (increasing sand content/coarseness with depth)											
25				BC=6 17 26	78%		23.5	104.2		25			
		Poorly graded SAND with Gravel (SP): brownish gray, wet, very dense, coarse to fine grained sand, subangular to subrounded gravel to 0.5"											
30				BC=30 32 50/6"	83%	SP			85	3.6			

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BORING LOG KB-2

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-4

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Date Begin - End:	9/09/2015	**Drilling Company:**	Taber Drilling		**BORING LOG KB-2**
Logged By:	C. Ewing	**Drill Crew:**	Andy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	CME-55	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary		
Weather:	Hot	**Exploration Diameter:**	4.5 in. O.D.		

FIELD EXPLORATION — Surface Condition: Road Shoulder

Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Well-graded SAND with Silt and Gravel (SW-SM): grayish brown to olive yellow, wet, very dense, coarse to fine grained sand, subangular to subrounded gravel to 0.75", (increasing gravel content/coarseness with depth)		BC=31 41 48	83%	SW-SM			62	5.3			
40		subangular to subrounded gravel to 2.0"		BC=26 19 10 PP=1.5	28%								
		Fat CLAY (CH): high plasticity, gray to bluish gray, moist, firm											
45		**Poorly graded GRAVEL with Clay and Sand (GP-GC)**: low plasticity, olive yellow to grayish brown, moist, very dense, coarse to fine grained sand, subrounded to rounded gravel to 2.0"		BC=39 47 32	67%								
50				BC=21 50/5"	27%								
55		**Lean CLAY (CL)**: low plasticity, brown to olive yellow, moist, hard, coarse to fine grained sand		BC=9 15 19	17%								
60		**Poorly graded GRAVEL with Clay and Sand (GP-GC)**: non-plastic, light brown to olive yellow, moist, very dense, coarse to fine grained sand, subrounded to rounded gravel to 0.5"		BC=27 50/3"	56%								
65													

The boring was terminated at approximately 61 ft. below ground surface. The exploration was backfilled with Neat Cement Grout on September 09, 2015.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not encountered during drilling or after completion.
GENERAL NOTES:

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BORING LOG KB-2

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
A-4

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Appendix B

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
KB-1	5.5 - 6.5	SILT WITH SAND (ML)	24.0	101.3		100	78				
KB-1	11.0	SILTY SAND (SM)				99	14				
KB-1	16.0 - 18.0	POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)			89	59	8.7				
KB-1	25.5	CLAYEY SAND WITH GRAVEL (SC)					26				
KB-1	30.5 - 31.5	WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)			90	55	5.7				
KB-2	15.5	LEAN/FAT CLAY (CL-CH)	28.7	87.4							
KB-2	21.0	LEAN/FAT CLAY (CL-CH)	26.4	97.5							
KB-2	26.0	CLAYEY SAND (SC)	23.5	104.2			25				
KB-2	30.5 - 31.5	POORLY GRADED SAND WITH GRAVEL (SP)			100	85	3.6				
KB-2	35.5 - 36.5	WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)			94	62	5.3				

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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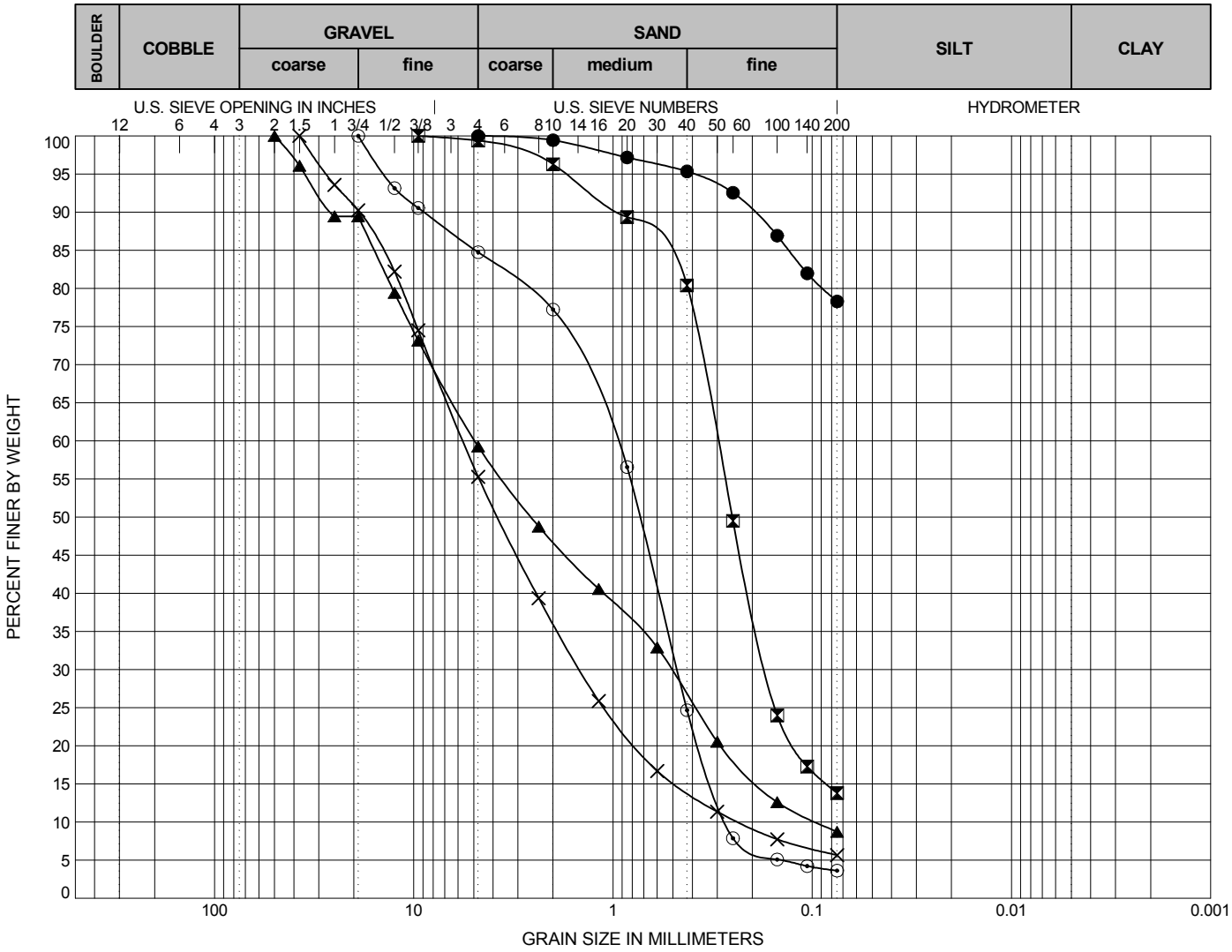
LABORATORY TEST RESULT SUMMARY

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

TABLE
B-1



	Exploration ID	Depth (ft.)	Sample Description	LL	PL	PI
●	KB-1	5.5 - 6.5	SILT WITH SAND (ML)	NM	NM	NM
⧗	KB-1	11	SILTY SAND (SM)	NM	NM	NM
▲	KB-1	16 - 18	POORLY GRADED SAND WITH SILT AND GRAVEL (SP-SM)	NM	NM	NM
×	KB-1	30.5 - 31.5	WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)	NM	NM	NM
⊙	KB-2	30.5 - 31.5	POORLY GRADED SAND WITH GRAVEL (SP)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	KB-1	5.5 - 6.5	4.75	NM	NM	NM	NM	NM		100	78	NM	NM
⧗	KB-1	11	9.5	0.299	0.169	NM	NM	NM		99	14	NM	NM
▲	KB-1	16 - 18	50	4.928	0.51	0.094	0.56	52.42	89	59	8.7	NM	NM
×	KB-1	30.5 - 31.5	37.5	5.631	1.459	0.231	1.63	24.35	90	55	5.7	NM	NM
⊙	KB-2	30.5 - 31.5	19	0.98	0.477	0.267	0.87	3.66	100	85	3.6	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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SIEVE ANALYSIS

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
B-2

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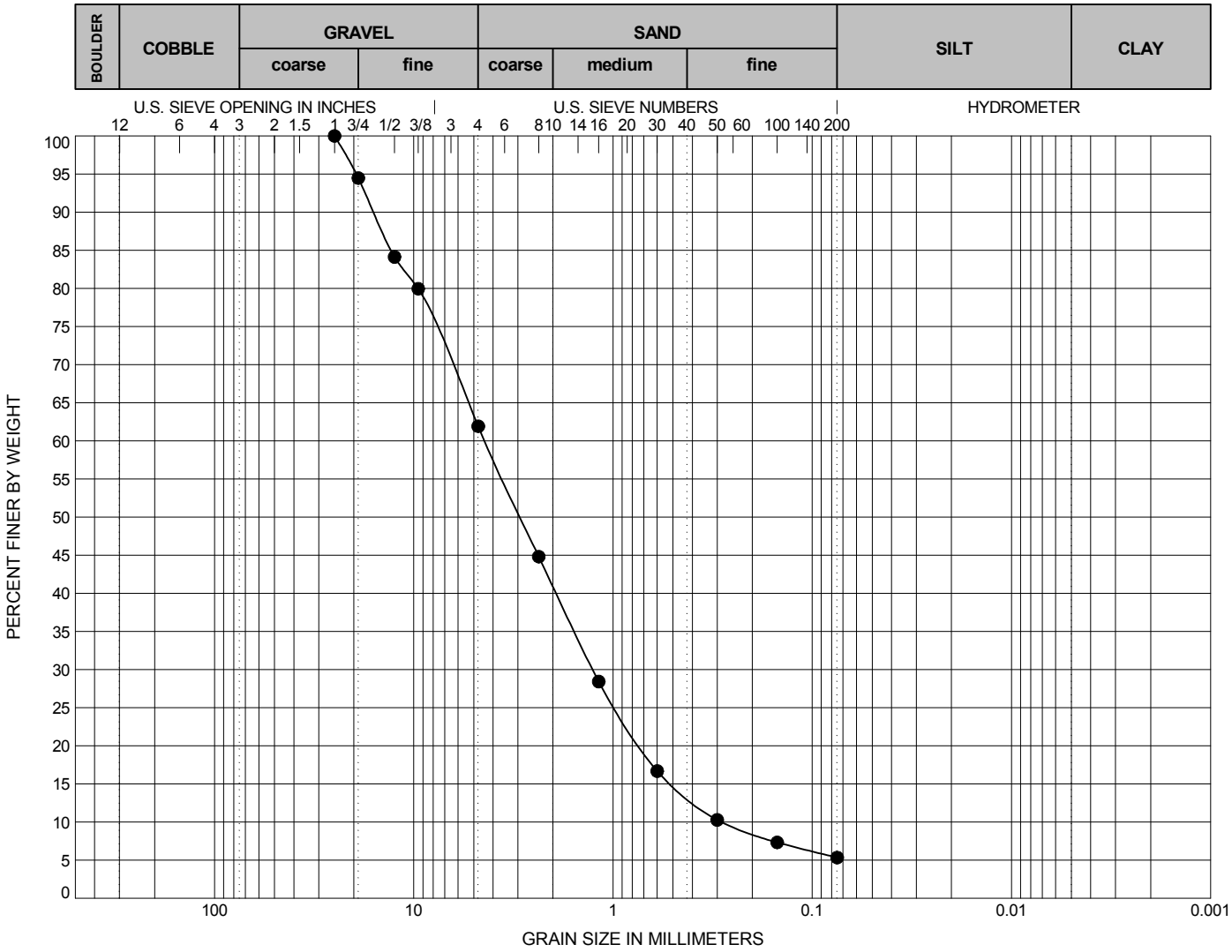

	Exploration ID	Depth (ft.)	Sample Description	LL	PL	PI
●	KB-2	35.5 - 36.5	WELL GRADED SAND WITH SILT AND GRAVEL (SW-SM)	NM	NM	NM

	Exploration ID	Depth (ft.)	D_{100}	D_{60}	D_{30}	D_{10}	Cc	Cu	Passing 3/4"	Passing #4	Passing #200	%Silt	%Clay
●	KB-2	35.5 - 36.5	25	4.392	1.261	0.281	1.29	15.62	94	62	5.3	NM	NM

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60}\, D_{10}$
D_{60} = Grain diameter at 60% passing
D_{30} = Grain diameter at 30% passing
D_{10} = Grain diameter at 10% passing

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SIEVE ANALYSIS

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE
B-2

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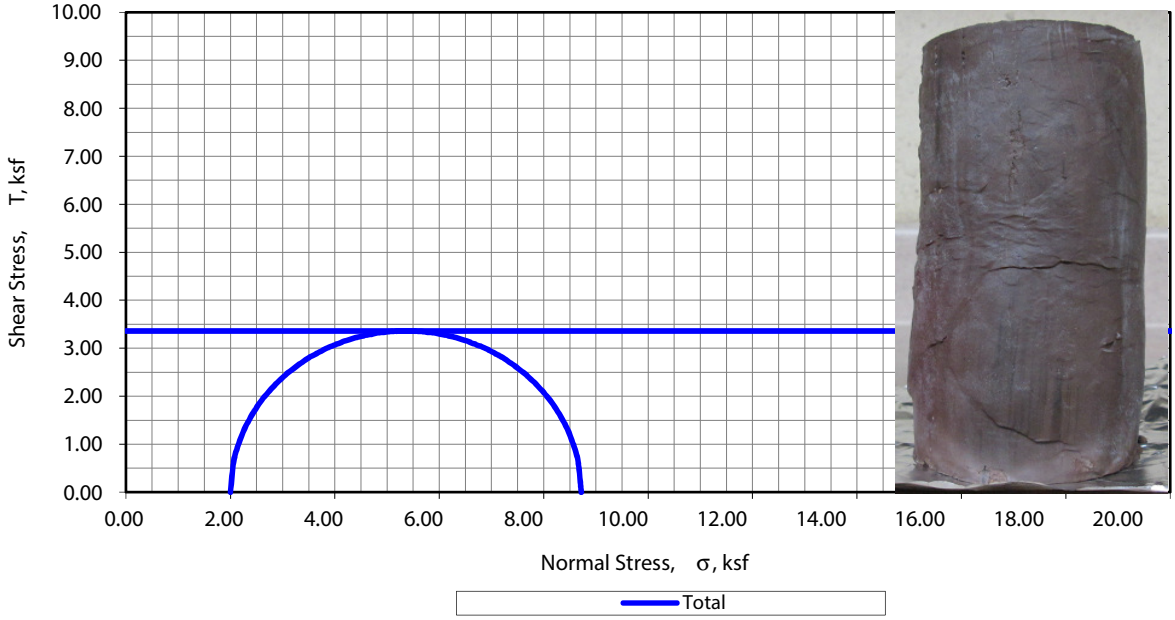
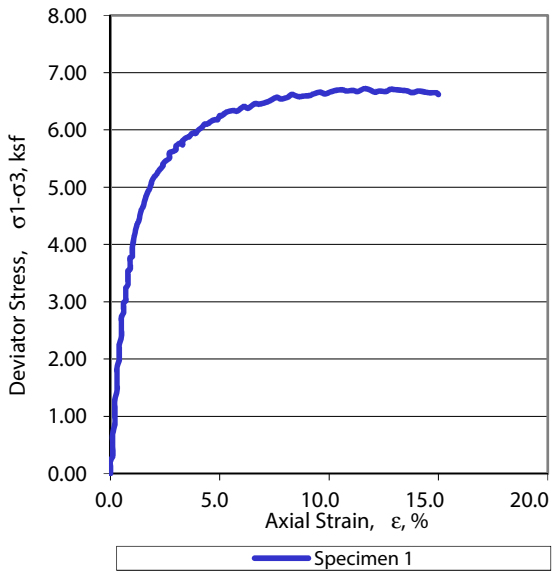
Total		
c =	3.36	ksf



Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.40
	Height, in	H_O	5.81
	Water Content, %	ω_O	26.4
	Dry Density, lbs/ft ^{3}	γd_o	97.5
	Saturation, %	S_O	101
	Void Ratio	e_O	0.696
Minor Principal Stress, ksf		σ_3	2.00
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.72
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	11.60
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.62
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	11.60

Description of Specimen: Lean/Fat Clay (CL-CH)

Amount of Material Finer than the No. 200, %: mn

LL: nm	PL: nm	PI: nm	G_S: 2.65 Assumed	Specimen Type: Undisturbed	Test Method: ASTM D2850

Membrane correction applied

Boring:	KB-2
Sample:	4C
Depth, ft:	21.0
Test Date:	9/15/15

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.

PROJECT NO.:	20162052
DRAWN BY:	CSE
CHECKED BY:	JCR
DATE:	9/17/2015
REVISED:	-

UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST (TXUU)

FIFTH STREET EAST RECYCLED WATER PIPELINE
PROPOSED NATHANSON CREEK CROSSING
SONOMA, CALIFORNIA

PLATE

B-3

KLEINFELDER

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Appendix C

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 10/02/15
Date Submitted 09/29/15

To: Martin Pucci
Kleinfelder
2240 Northpoint Pkwy
Santa Rosa, CA, 95407

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 20162052-SONOMA Site ID: KB-1 1B-1C
Thank you for your business.

* For future reference to this analysis please use SUN # 70544 - 147177

EVALUATION FOR SOIL CORROSION

Soil pH	7.38			
Minimum Resistivity	0.60	ohm-cm (x1000)		
Chloride	299.3	ppm	0.0299	%
Sulfate-S	51.8	ppm	0.0052	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11419 Sunrise Gold Cir.#10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/30/15
Date Submitted 09/25/15

To: Martin Pucci
Kleinfelder
2240 Northpoint Pkwy
Santa Rosa, CA, 95407

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following:
Location : 20162052-NATHANSON Site ID: KB-2-2B
Thank you for your business.

* For future reference to this analysis please use SUN # 70523 - 147101

EVALUATION FOR SOIL CORROSION

Soil pH	7.79			
Minimum Resistivity	3.22	ohm-cm (x1000)		
Chloride	27.6	ppm	0.0028	%
Sulfate-S	6.6	ppm	0.0007	%

METHODS:
pH and Min.Resistivity CA DOT Test #643 Mod.(Sm.Cell)
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

KLEINFELDER

Bright People. Right Solutions.

Appendix D

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly

problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

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