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May 15, 2009
Project No. 3044.072.03

AECOM
5821 Thille Street, Suite 201
Ventura, California 93003

Attention: Mr. Douglas Hahn

Subject: Summary of Anticipated Geotechnical Conditions, Proposed Turnout Site Alternatives, Calleguas Municipal Water District, Oak Park Reclaimed Water System, Oak Park Area, Ventura County, California

Dear Mr. Hahn:

Fugro is pleased to submit this letter summarizing the anticipated geotechnical conditions at two proposed turnout locations for the Oak Park Reclaimed Water System. The proposed sites are located in the area along Kanan Road, between Lindero Canyon Road and Falling Star Avenue. This letter was prepared in accordance with our correspondence dated August 19, 2008, and provides a summary of the geotechnical data obtained in previous explorations performed by Staal Gardner & Dunne, Inc. (SGD [now Fugro], 1991), within the limits of the proposed turnout locations.

PROJECT UNDERSTANDING

The Calleguas Municipal Water District (CMWD) is considering installing a turnout along the Oak Park Reclaimed Water System in the Oak Park area of Ventura County. There are currently two alternative site locations for the turnout as shown on Plate 1 - Location Map. Site 1 is located near the northeast corner of Kanan Road and Lindero Canyon Road. Site 2 is located near the southwest corner of Kanan Road and Falling Star Avenue. We understand that the base of the turnout will be approximately 10 feet below existing grade.

SGD conducted a geotechnical design study for the reclaimed water distribution system in October 1991 (SGD, 1991) that included subsurface explorations advanced near the proposed turnout sites. For the purpose of this project, we have reviewed our 1991 report and evaluated the relevant data in the areas of the proposed turnout sites along Kanan Road.

WORK PERFORMED

The work performed for this project consisted of the review of data from our previous geotechnical report (SGD, 1991) and preparation of this letter summarizing the geotechnical conditions encountered in the drill holes advanced near the proposed turnout site alternatives.

SUBSURFACE CONDITIONS

SITE 1 - KANAN ROAD AND LINDERO CANYON ROAD

For the 1991 study, SGD advanced drill hole DH-7 on Kanan Road approximately 185 feet southwest of the proposed Site 1 (Plate 1). The subsurface materials encountered in DH-7 consisted of about 5 inches of asphalt pavement overlying artificial fill (af)/alluvium (Qal) soils to the depth explored (16.5 feet below ground surface [bgs]). The soil materials consisted primarily of medium dense sand and clayey sand, and very stiff sandy clay. DH-7 is located within the confluence of a natural drainage feature known as Lindero Canyon (USGS, 1981). Based on our data review, it appears that this area may have been filled by natural and artificial means to existing grade. A turnout at this location is anticipated to be founded in medium dense or very stiff soil materials. The log of the drill hole for DH-7 is provided in Appendix A - Subsurface Exploration (SGD, 1991).

SITE 2 - KANAN ROAD AND FALLING STAR AVENUE

For the 1991 study, SGD advanced drill hole DH-6 on Kanan Road approximately 175 feet northwest of the proposed Site 2 (Plate 1). The subsurface materials encountered in DH-6 consisted of about 6 inches of asphalt pavement overlying artificial fill (af) and artificial fill (af)/alluvium (Qal) overlying weathered Topanga Formation (Tt) bedrock. The artificial fill/alluvial materials consisted of clayey sand to a depth of about 5 feet bgs. The underlying weathered bedrock material consisted primarily of medium dense to dense clayey sand and sand materials. A turnout at this location is anticipated to be founded in bedrock materials of the Topanga Formation that have the consistency/density of medium dense to dense clayey sand or sandy soil materials. The drill-hole log for DH-6 is also provided in Appendix A.

GROUNDWATER

Groundwater was not encountered in either drill hole advanced in our 1991 subsurface exploration program near the proposed turnout sites (SGD, 1991). However, groundwater may be encountered at other locations within the proposed site vicinity, especially in areas adjacent to irrigated parks or landscaped areas. We note that variations in depth to perched groundwater and groundwater seepage can occur as a result of change in land use, irrigation, rainfall, or runoff.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

The conclusions and recommendations presented in this letter are based on review of the field exploration and laboratory testing programs performed during our 1991 study, engineering evaluation, and our current understanding of the project. Our geotechnical recommendations for site preparation and grading, fill placement, and for the design of the turnout are presented below. Site-specific subsurface exploration to confirm the data review was not part of the scope of services for this project.

EARTHWORK AND GRADING

Fill placement and grading operations should be performed in accordance with CMWD specifications, Standard Specifications for Public Works Construction (Greenbook, latest edition), and the grading recommendations presented in this letter.

Excavation Considerations

Excavation Potential. The drill holes advanced during our 1991 study were excavated with a hollow-stem-auger drill rig. In general, the onsite materials were excavated with light to moderate effort. We expect that the onsite materials can be excavated using relatively heavy-duty earth-moving excavation equipment in good working order.

Excavations and Shoring. Based on our review of the soils observed in dill holes DH-6 and DH-7 from our 1991 study, we recommend that excavation plans and specifications adhere to guidelines for a Type C soil per Occupational Safety and Health Administration (OSHA, 1926). Per OSHA (1926), unsupported excavations for Type C soils should be sloped no steeper than 1.5h:1v, and even flatter slopes may be warranted depending on exposed soil conditions. Stockpiled material or equipment should not be placed closer than 5 feet from any slope crest. Dewatering and erosion protection, such as controlled runoff drainage, should be provided as necessary.

If sloped excavations are not feasible at the site, excavations more than 4 feet deep should be excavated, braced, shored, or shielded in accordance with federal and state standards, project specifications, and safe construction practices. Federal and State OSHA regulations should be adhered to with regard to shoring and bracing of the excavations. The contractor should be responsible for design and implementation of shoring systems and safe working conditions. The contractor should continuously monitor the temporary slopes and support and remove or stabilize any loose or unstable soil masses.

Excavation Bottom Preparation

We anticipate that fine-grained to coarse grained soil materials will be exposed across the structure area at the design excavation level for Site 1. We anticipate that bedrock materials with the consistency/density of medium dense to dense soil will be exposed across the structure area at the design excavation level for Site 2. Prior to backfilling, the excavation bottom should be observed by Fugro. If unsuitable soils or materials are encountered at that level, additional excavation may be required.

Following our observations, the subgrade should be cross scarified to a depth of at least six inches, moisture conditioned to a water content between optimum and 2 percent above optimum, and compacted to at least 90 percent relative compaction. We also recommend that Fugro observe all foundation excavations before placing reinforcement and concrete. Foundation excavations should be clean of loose and disturbed materials.

Fill Placement and Compaction

Placement. Based on our review of the soils encountered by drill holes DH-6 and DH-7, we anticipate that the soils removed during excavation of the turnout may be reused as compacted fill during backfilling operations.

Fill materials should be spread evenly, with loose lifts no thicker than 8 inches, and should be thoroughly blade-mixed during spreading to provide relative uniformity of material within each layer. Soft or yielding materials should be removed and be replaced with properly compacted fill material prior to placing the next layer.

Compaction Requirements. Fill materials should be moisture-conditioned to between optimum moisture content and 2 percent above optimum moisture content prior to and during compaction. Water should be added to the fill when the moisture content of the fill material is below that sufficient to achieve the recommended compaction. While water is being added, the soil should be bladed and mixed to provide uniform moisture content throughout the material. When the moisture content of the fill material is excessive, the fill should be aerated by blading or other methods.

Fine-grained materials are sensitive to changes in moisture content, and can be relatively difficult to compact. Hence, if fill materials are placed at a moisture content more than a few percent above optimum, there is a potential for the fill to pump or yield when subjected to construction traffic or compactive effort. Control of the moisture content and compaction using thin lifts will be critical in achieving the required compaction.

All fill materials should be compacted to a minimum of 90 percent (relative compaction) of the maximum dry density determined from ASTM D1557, latest edition. Fill placed within 1 foot of pavement sections should be compacted to a relative compaction of at least 95 percent of the maximum dry density.

Backfill within 5 feet of below grade walls (i.e. vault walls, measured horizontally) should be compacted with lightweight, hand-operated compaction equipment to minimize the potential for large compaction stresses. If large or heavy equipment is used, compaction-induced stresses can result in increased lateral earth pressures on the vault walls.

Compaction testing should be performed during fill placement. Measurements of in situ or field moisture content and relative compaction should be evaluated using either ASTM D2922 (nuclear gauge) and/or ASTM D1556 (sand cone method).

VAULT FOUNDATION DESIGN

Based on information provided by AECOM, the vault portion of the proposed turnout will extend to a depth of about 10 feet bgs, and will likely be founded on a mat-type foundation system. During our foundation design evaluations, we assumed a friction angle of about 30 degrees and cohesion of about 150 pounds per square foot (psf) for the earth materials at Sites 1 and 2.

Allowable Bearing Pressure

As indicated above, we anticipate the vault will be supported on a concrete mat-type foundation system that will be underlain by either fill/alluvial soils at Site 1, or underlain by bedrock with the consistency/density of soil at Site 2. The proposed mat foundation can be designed using a net maximum allowable bearing pressure of 2,500 psf at either of the two site alternatives. The maximum net allowable bearing pressure can be increased by one-third when considering short-term wind or seismic loads.

Modulus of Subgrade Reaction

A modulus of subgrade reaction (Kv_1) of about 250 pounds per cubic inch may be assumed for design of mat-type foundations using a beam on elastic foundation analogy (a Winkler model). The modulus of subgrade reaction value (Kv_1) represents a presumptive value based on soil classification data and is for a 1-foot-square plate. The Kv_1 value may need to be adjusted by the designer for mat size, assuming a cohesive subgrade.

Resistance to Lateral Loads

Sliding Resistance. Resistance to lateral loading can be provided by sliding friction acting along the base of spread footings combined with passive pressure acting on the sides of the foundations. Ultimate sliding resistance generated through a soil/concrete interface can be estimated using a coefficient of friction of 0.40.

Passive Resistance. Ultimate passive resistance of foundation elements can be estimated using an equivalent fluid weight of 300 pcf for foundations bearing against competent material where the backfill surface is horizontal.

Factors of Safety. Passive resistance and friction can be combined when evaluating sliding resistance. For static conditions, a safety factor of 1.5 is recommended for sliding and overturning. The factor of safety for sliding and overturning may be reduced to 1.1 for dynamic conditions.

Lateral Earth Pressures

Earth Pressure State. We anticipate the walls of the proposed vault will be designed as restrained walls subject to at-rest earth pressures. In general, we have assumed that select fill material will be used for backfill behind the walls and within a 1h:1v projection up from the base of the wall; however, we have also provided estimated earth pressure values if general fill materials are used behind the walls. Equivalent fluid weight (EFW) values provided below for select backfill conditions can be used to estimate the design lateral earth pressure. EFWs for active conditions may be used for active conditions in conjunction with dynamic force increments described below.

Table 1. Equivalent Fluid Weights for Wall Design

Backfill Slope	At-Rest[1]	Active[1]
Relatively Level, Select Granular Backfill	60	50
Relatively Level, On-Site Fine-Grained[2] Backfill	90	80
Notes: [1]For drained conditions, [2]Fine-grained soil materials with low expansion potential		

Recommended EFWs for static conditions do not include hydrostatic pressures and assume that drainage measures will be incorporated into the vault design to preclude the development of hydrostatic pressures behind proposed below-grade walls.

Surcharge Loads. Uniform area surcharge pressures for the vault walls may be assumed equal to one-half of the applied surcharge pressure at the ground surface. Surcharge load from automobiles and pickup trucks (not large trucks or construction equipment) may be assumed equivalent to 2 feet of soil surcharge (area surcharge at the ground surface of 250 psf). Lateral pressures for other surcharge loading conditions can be provided, if required.

Dynamic Pressures. For unrestrained walls, the increase in lateral earth pressure due to earthquake loading can be estimated using the Mononobe-Okabe theory, as described in Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure due to earthquake loading to be estimated using the Mononobe-Okabe theory. Because the theory is based on the assumption that sufficient movement occurs so that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct; however, there is a supporting reference (Nadim and Whitman, 1992) that suggests the theory can be used for such walls.

In the Mononobe-Okabe approach, the total dynamic pressure can be divided into static and dynamic components. Dynamic force increments for level backfill conditions are presented below. To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using recommendations for active (not at-rest conditions) lateral earth pressures presented previously. That recommendation is based on the concept that during shaking, earth pressures recommended for permanent conditions will be reduced to those more closely approximating active conditions. The resultant dynamic lateral force increment should be applied at a distance of 0.6H above the base of the wall, while the static lateral force should be applied at a distance of 1/3 the wall height above the base of the wall.

For level backfill conditions, the estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls may be taken as $45 \times Kh \times H^2$ pounds per linear foot of wall. An estimated dynamic lateral force increment of $14\ H^2$ (pounds per foot of wall) can be used for design assuming an acceleration value in the computation equal to 2/3 of the 475-year period peak ground acceleration value of 0.41g (CGS, 2008) (Kh=0.3). In the above formulation, H is the height of wall below the ground surface in feet.

Foundation Settlement

Static settlement of the vault is anticipated to be less to be less than 1/2 to 1 inch and the differential settlement are anticipated to be less than 1/2 inch in 30 feet provided the subgrade materials underlying the vault are firm and unyielding.

When evaluating overall settlement of the vault foundation, both estimated static and seismically induced settlements should be considered. In addition, to account for the potential for movement between the proposed structure and connecting pipelines, we recommend that flexible joints be provided at those locations.

PAVEMENT SECTIONS

As previously discussed, drill holes DH-6 and DH-7 were performed within the asphalt pavement along Kanan Road (SGD, 1991). The thicknesses of the pavement sections were about 6 inches and 5 inches for DH-6 and DH-7, respectively. The existing pavement thicknesses in those areas may vary from the measurements in 1991 due to pavement maintenance practices (grinding and/or overlay) in the years since our 1991 study. Therefore, the pavement sections today in those areas may be greater than those observed in our 1991 study. The paving and base materials, structural section, and compaction requirements should conform to requirements of the latest editions of the Caltrans Highway Design Manual, Greenbook, CMWD specifications, or the permitting agency if the vault will be located in the public right-of-way. As a minimum, we suggest that the existing asphalt pavement section be replaced with an equal thickness of asphalt concrete, plus 1 inch.

CLOSURE

We appreciate the opportunity to provide this summary of the geotechnical conditions encountered in 1991 near the proposed turnout site alternative locations for the CMWD Oak Park Reclaimed Water System. If you have any questions or wish to have Fugro review additional data or perform additional analyses, please contact our office.

Sincerely,

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Attachments: References
Plate 1 - Location Map
Appendix A - Subsurface Exploration (SGD, 1991)

Copies Submitted: (2) Addressee and 1-Pdf

REFERENCES

California Division of Mines and Geology (CDMG) (2000), Seismic Hazard Evaluation of the Thousand Oaks 7.5-Minute Quadrangle, Ventura and Los Angeles Counties, California, Open File Report 2000-008.

California Geological Survey (CGS website, 2008), http://www.consrv.ca.gov/CGS/rghm/psha/index.htm.

Greenbook (2006), *Standard Specifications for Public Works Construction,* BNI Building News, BNI Publications, Inc.

Nadim, F. and Whitman, R.V. (1992), *Seismic Analysis and Design of Retaining Walls*, ASME, OMAE, Volume II, Safety and Reliability.

Occupational Safety and Health Administration (1926), *Subpart B - Excavations,* Table V:2-1 "Allowable Slopes".

Seed, H.B., and Whitman, R.V. (1970), *Design of Earth Retaining Structures for Dynamic Loads*, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Ithaca, New York, pp. 103-147.

Staal Gardner & Dunne Inc (SGD [now Fugro], 1991), "Geotechnical Study, Oak Park/North Ranch Reclaimed Water Distribution System Tank, Pump Station, and Pipelines, Ventura County, California," prepared for Boyle Engineering Corporation, dated October 1991.

United States Geological Survey (1981), Topographic Map the Thousand Oaks 7.5-Minute Quadrangle, Los Angeles and Ventura Counties, California, Map No. 34118-B7-TF-024.

PLATE

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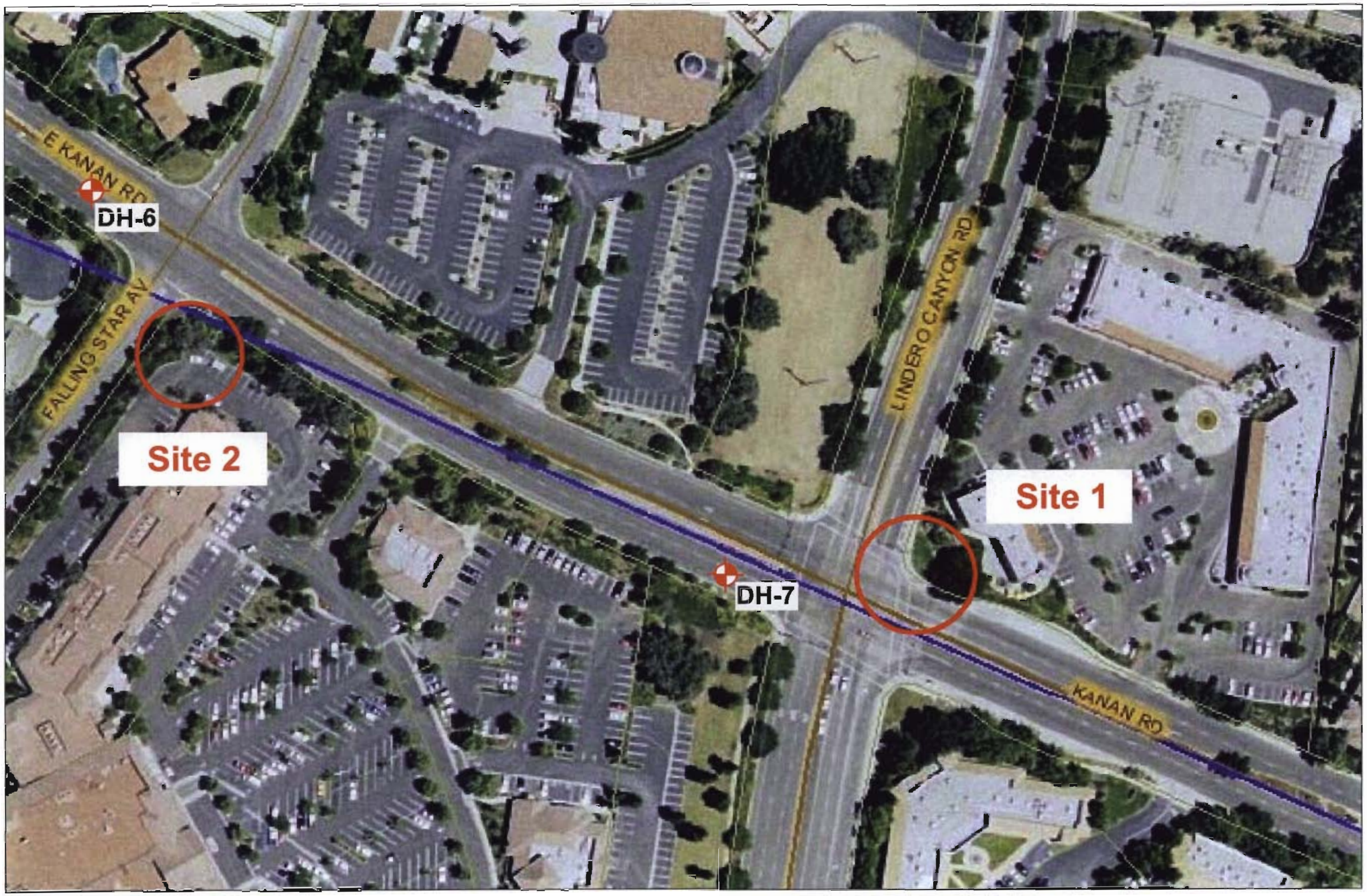

BASE MAP SOURCE: Aerial photo provided by AECOM (August, 2008).

LEGEND

DH-7 Approximate drill hole location (SGD [now Fugro], 1991)

LOCATION MAP
Proposed Turnout Site Alternatives
Calleguas Municipal Water District
Oak Park Reclaimed Water System
Oak Park, California

NORTH
0 150
FEET

APPENDIX A
SUBSURFACE EXPLORATION (SGD, 1991)

LOG OF DRILL HOLE

JOB NO. : V91102
PROJECT : Oak Park/North Ranch RWS
LOCATION: Kanan Road
DRILLING METHOD: Georex T500, 6" HSA

LOGGED BY : L. Prentice
DRILLED BY: Valley Well Drilling
TIME START: 1305 8/28/91
TIME STOP : 1425 8/28/91

DRILL HOLE NO.: DH-6

DATUM: MSL
REFERENCE EL.: 1177 feet (approx.)

ELEVATION (FEET)	DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS
						ARTIFICIAL FILL (af) ASPHALTIC CONCRETE, 6 inches			
1175						**ARTIFICIAL FILL (af)/ALLUVIUM (Qal)** CLAYEY SAND (SC), moderate yellowish brown, damp, fine grained			
	5		2	42		**TOPANGA FORMATION (Tt)** CLAYEY SAND (SC), pale yellowish brown to grayish orange, damp, medium dense, interlaminated with SILT (ML) or silty sand, very pale orange, damp, medium dense	104	15	
1170									
	10		3	47		SAND (SP), grayish orange, dry to damp, dense, fine grained, thinly laminated with mottled iron staining on bedding planes, and minor caliche	100	21	
						Bottom of drill hole at 11.5 feet. No ground water encountered. Drill hole backfilled with native soil, tamped, and patched with Wespro to 1-1/2 inches above existing surface.			

This log applies only at the location of this drill hole and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplification of actual conditions encountered.

GT-1

LEGEND TO LOGS ON PLATE A-2

SGD

LOG OF DRILL HOLE

JOB NO. : V91102
PROJECT : Oak Park/North Ranch RWS
LOCATION: Kanan Road
DRILLING METHOD: Georex T500, 6" HSA

LOGGED BY : L. Prentice
DRILLED BY: Valley Well Drilling
TIME START: 1450 8/28/91
TIME STOP : 1530 8/28/91

DRILL HOLE NO.: DH-7
DATUM: MSL
REFERENCE EL.: 1169 feet (approx.)

ELEVATION (FEET)	DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (PERCENT)	ADDITIONAL TESTS
						ARTIFICIAL FILL (af) ASPHALTIC CONCRETE, 5 inches			
			1			**ARTIFICIAL FILL (af)/ALLUVIUM (Qal)** SAND (SP), dark yellowish brown, damp, fine grained, with clay and common coarse gravel to 1-inch diameter			CHEM
1165	5		2	22		CLAYEY SAND (SC), dark reddish brown, dry to damp, medium dense, fine- to medium-grained with minor coarse sand and fine gravel to 1/4-inch diameter	112	16	
1160						9 feet: black			
	10		3	25		SANDY CLAY (CL), moderate brown, dry, very stiff, fine grained with common caliche veins	106	13	
1155	15		4	31		SAND (SP), moderate yellowish brown, dry, fine grained	115	11	
						Bottom of drill hole at 16.5 feet. No ground water encountered. Drill hole backfilled with cuttings, tamped, and patched with Wespro.			

This log applies only at the location of this drill hole and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented are a simplificationof actual conditions encountered. GT-1

LEGEND TO LOGS ON PLATE A-2