Geotechnical Investigation Report
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

Prepared for

County of El Dorado
Department of Transportation
Tahoe Erosion Control Unit
924B Emerald Bay Road
South Lake Tahoe, California 96150

MACTEC Project No. 4308080010

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Geotechnical Investigation Report
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California
MACTEC Project No. 4308080010

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PLATES

APPENDICES

1.0 INTRODUCTION

This report, by MACTEC Engineering and Consulting, Inc. (MACTEC), presents the results of our geotechnical investigation for the proposed new CON/SPAN[1] roadway bridge, which will span Angora Creek along Lake Tahoe Boulevard, near the Angora Road intersection, in South Lake Tahoe, California. We understand that the bridge is part of the Angora Creek Fisheries/SEZ Enhancement Project.

1.1 Project Description

The project site is located as shown on Plate 1-1, Vicinity Map, and Plate 1-2, Site Plan. Ms. Amy Dillon of Eldorado County Department of Transportation (EDOT) provided us with information regarding the scope of the proposed project. Based on this information, we understand that the proposed new bridge will replace two existing pipe culverts and deteriorating concrete headwalls at the creek crossing. Based on preliminary plans, the bridge will span approximately 20 feet and be approximately 7 feet high, with angled wing walls. The bridge foundation level is planned to be approximately at Elevation 6330 feet (MSL datum). Earthwork required for the new construction is expected to be limited to excavations to remove the existing culverts, preparation of the creek bottom for the new bridge, and backfilling behind and above the CON/SPAN precast concrete panels. The roadway section then will be replaced.

1.2 Scope of Services

Our services were performed in accordance with our proposal, dated October 7, 2008. The scope of our services included exploring subsurface conditions by drilling two test borings, performing laboratory tests, researching available geologic data, performing geotechnical engineering analyses, and developing recommendations for final project planning and design. The obtained information was used to develop conclusions and recommendations regarding the following:

- Subsurface soil and groundwater conditions;
- Site geology and assessment of potential geohazards;
- Appropriate seismic criteria for structural design;
- Site preparation and earthwork, including fill and backfill compaction criteria;

[1] CON/SPAN® is a patented modular precast system for construction of bridges, culverts, and underground structures. More information can be found at CON/SPAN® website(www.con-span.com).

- Subgrade preparation for footings and pavement areas;
- Geotechnical design criteria for use in foundation design, including bearing capacities, resistance to lateral loads, and estimated settlements;
- Lateral earth pressures (static and seismic) for retaining wall design;
- Pavement thicknesses, including aggregate base and asphalt concrete materials;

Our services did not include an assessment of potentially toxic and hazardous material that may be present on or beneath the site.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Prior to starting our field investigation, an EDOT engineer marked the proposed the boring locations (see Plate 1-2) and called for utility Underground Service Alert clearances. On January 5, 2009, we cored the asphalt pavement and drilled borings at two locations near the proposed bridge. The borings were drilled to approximately 20 feet deep with 6-inch diameter hollow stem auger. Our staff observed the drilling and logged the soils encountered. The soils were classified in accordance with the soil classification criteria outlined on Plates A-1 and A-2 in Appendix A. Soil samples were obtained at appropriate intervals in the borings using a Sprague and Henwood (S&H) split barrel sampler (3.0-inch outside diameter, 2.43-inch inside diameter) lined with 6-inch-long brass tubes. Additionally, we obtained two bulk samples of the near surface soils beneath the pavement layer. The S&H sampler was driven by a 140-pound hammer falling 30 inches using the automatic trip method. The number of blows required to drive the samplers the final 12 inches of an 18-inch drive were recorded. The observed blow counts were then converted to approximate SPT N-values[2]. The converted N.values, which should be considered approximate, are shown on the boring logs. At the completion of drilling, the soil borings were backfilled with soils cuttings and capped with asphalt. Level D personal protective equipment was used during drilling operations.

2.2 Laboratory Testing

We re-examined the soil samples from the borings in our office to check field classifications and to select samples for laboratory testing. Laboratory tests performed by MACTEC included R-Value, moisture content and dry density, sieve analysis, sieve No. 200 passing fraction, and organic content tests. Laboratory test results are shown on the boring logs in accordance with the key to test data on Plate A-1 in Appendix A, and on test reports in Appendix B.

[2] The SPT N-value is defined as the number of blows of a 140-pound hammer, falling freely through the height of 30 inches, required to drive a standard split-barrel sampler (2-inch outside diameter and 1-3/8-inch inside diameter) for the last 12 inches of an 18-inch drive. For SPT procedures, see ASTM D1586-84.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Conditions

The project site is located in South Lake Tahoe, California on Lake Tahoe Boulevard, where it crosses Angora Creek (Latitude 38.8812°N and Longitude 120.041°W). Near the proposed bridge, the roadway is constructed on an embankment. The area surrounding the roadway is generally covered with trees, grass and other surface vegetations. The site elevation, on the roadway near the proposed bridge, is about 6342 feet (MSL Datum), and elevations increase to the west. The site is at the foothill of the Angora Ridge (approximate elevation of 7000 to 7200 feet), less than a mile to the northwest of the site.

Currently, the roadway crosses Angora Creek, which is contained in two pipe culverts beneath the roadway. A sheet pile wall is visible south of the existing roadway that appears to be protecting the creek channel. Boulders of different sizes cover the creek channel.

3.2 Subsurface Conditions

Below an 8-inch-thick layer of asphalt concrete, our borings encountered about 9 to 10 feet of light brown silty sand fill soils, which are part of the roadway embankment. The top 5 feet of these soils was dense to very dense and appear to have been placed as compacted fill. Deeper soils in this layer were only medium dense. Below the fill, we encountered native gray silty sands to depths of approximately 20 feet. These sands were medium dense in Boring B-1 and to a depth of 19 feet in Boring B-2, at which they became dense and light brown.

We encountered groundwater at Boring B-1. The depth of ground water varied between 14 to 11 feet below ground surface (Elevation 6,328 and 6,331) during drilling and backfilling, respectively. Groundwater was not encountered in Boring B-2; however, the soils were wet below a depth of 10 feet. Based on the boring data, we judge that the groundwater at the time of our investigation was near the elevation of the bottom of the creek.

4.0 GEOLOGY AND GEOLOGIC HAZARDS

4.1 Geology and Seismicity

4.1.1 Geologic Setting

Angora Creek belongs to the Upper Truckee River Watershed and is located within the Tahoe South Subbasin of the Tahoe Valley Groundwater Basin. The groundwater basin resides within the larger structural feature known as the Lake Tahoe Basin. Bedrock beneath the basin is primarily granitic and is found at depths ranging from tens to many hundreds of feet below ground surface (bgs). Sediments within the basin are glacial, fluvial, and lacustrine, and are referred to collectively as basin-fill deposits. In the vicinity of the project, Angora Creek flows though sediments known as Pre-Tahoe Till which were deposited as lateral moraines and are composed primarily of sands, gravels, and large boulders. Younger fluvial deposits are present along the creek channel. Soils in the basin are typically of granitic or volcanic parent material and poorly developed.

4.1.2 Faults and Seismicity

The project is not located within an Alquist-Priolo Earthquake Fault zone, though several faults have been identified within the Lake Tahoe Basin. Major faults include those of the North Tahoe-Incline village Fault Zone at the north end of the lake, and the West Tahoe-Dollar Point Fault Zone that runs along the western shore nearer to the project. The region is tectonically active, and major earthquakes are estimated to occur roughly every 3,000 years.

4.2 Geologic Hazards

4.2.1 Earthquake Ground Shaking and Seismic Design Criteria

The most significant geologic hazards at the site is strong ground shaking during a major earthquake. The following table presents peak bedrock acceleration (PBA) on outcropping rock, Peak Ground Acceleration (PGA), maximum credible earthquake (MCE), and the soil profile type for the site (stiff soil, Type D), as determined by the California Department of Transportation (Caltrans) 1996 Seismic Hazard Map.

Location	PBA	MCE	PGA	Soil Type
Angora Creek Bridge	0.3g	7.25	0.36g	D

The Caltrans Acceleration Response Spectrum (ARS) for project site is presented in Plate 4-1. This spectrum is calculated based on Caltrans Seismic Design Criteria (SDC, Version 1.4, June 2006).

4.2.2 Seismically-Induced Densification Settlement

Seismically-induced densification settlement usually occurs in soft or low density uniform-sized fine-grained sands or silts above the groundwater level. The sands encountered in our borings below the probable foundation depth are saturated and not susceptible to seismically induced densification settlement.

4.2.3 Liquefaction and Resulting Settlement

Soil liquefaction is a phenomenon in which saturated (submerged), cohesionless soils experience a temporary loss of strength due to the buildup of excess pore water pressure during cyclic loadings, such as those induced by earthquakes. Soils most susceptible to liquefaction are loose, clean, saturated, uniformly graded, fine-grained sands that lie within approximately 50 feet of the ground surface. Saturated silty and clayey sands or well graded sands are less likely to liquefy during strong ground shaking.

Native silty sands below the foundation of the proposed bridge are medium dense and could be susceptible to liquefaction during a strong earthquake. In Boring B-2, at a depth of 19 feet, below these liquefiable sands, we encountered dense sands, which are not susceptible to liquefaction.

Liquefaction calculations were performed for an estimated MCE peak ground acceleration of 0.36g (see section 4.2.1). These calculations indicate that liquefaction settlements of about an inch could occur during a strong earthquake, if the soils are medium dense to a depth of 20 feet (below the existing roadway). If the medium dense soils extend to greater depths, the settlement could be larger, and likely to be in order of six inches.

4.2.3 Seiches

Seiche waves are seismically produced oscillating waves that can occur in enclosed basins such as Lake Tahoe. Computer modeling suggests a major earthquake could produce seiches (waves) within the lake of up to 30 feet (Elevation 6,255 feet), but the Site elevation (6342 feet) is above this level.

4.2.4 Soil Expansion Potential

Based on the results of our borehole logging and the laboratory testing performed for this investigation, the expansion potential for surficial soils (sands) at the site is nil.

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5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

We conclude that the planned new bridge is feasible from a geotechnical standpoint provided our conclusions and recommendations are incorporated into the design and construction of the project.

The primary geotechnical constraints for the proposed construction include: 1) the potential for strong ground shaking in the site vicinity as a result of a moderate to large earthquake; and 2) the potential for small settlements caused by seismically-induced liquefaction of native medium dense sands below the ground water level.

Because of the segmental nature of the CON/SPAN bridge, we judge that liquefaction settlements of an inch (and perhaps larger if medium dense sands are deeper than 20 feet) can be accommodated by the bridge without significant effects. Thus, we conclude that the bridge can be supported on spread footings founded in native medium dense sands.

In general, construction procedures and material should conform the procedures and specifications as described in the "State of California, Department of Transportation, Standard Specifications", dated May 2006 (Standard Specifications).

5.2 Earthwork

5.2.1 Site Preparation

Excavation for the new bridge should clear all surface and subsurface obstructions. This will require removal of the existing asphalt pavement, excavation of the existing roadway embankment, and the existing pipe culverts. Stripped asphalt and any material that does not meet fill requirements (see Section 5.2.3) should not be re-used as backfill. Upon completion of surface cleanup, site stripping, and excavation, the exposed foundation subgrade should be scarified to a depth of 6 inches, moisture conditioned to above optimum moisture content and compacted to a minimum of 95 percent relative compaction[3].

[3] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same soil determined by the ASTM-1557 laboratory procedure. Optimum moisture is the water content that corresponds to the maximum dry density.

5.2.2 Excavation Considerations

Excavations for new foundations could result in disturbance of adjacent soils. All disturbed soils should be excavated and replaced with properly compacted fill.

Groundwater will likely be encountered during the construction. Any groundwater encountered during in excavations shall be removed and excavations kept dry until they are backfilled at least 2 feet above the static ground water table. The contractor should be responsible for selection, design, permitting, and construction of the dewatering system. The contractor should be required to submit a dewatering plan to the County for review prior to start of construction.

All applicable safety requirements and regulations for excavations, including Occupational Safety and Health Administration (OSHA) regulations, should be met. The contractor should be responsible for maintaining the safety of all excavation slopes, including the use of shoring if necessary.

5.2.3 Material for Backfill

Backfill material in the CON/SPAN structure should consist of relatively non-expansive sands and gravels with less than 35% passing the No. 200 sieve, a Liquid Limit of less than 40 and a Plasticity Index of less than 10. Material for backfilling should not contain any debris, cobbles, or rock fragments larger than 4 inches in diameter, organic matter, or expansive clay soils. On-site soils that meet these requirements can be used as backfill material. We anticipate that the existing embankment fill soils (sands) can be reused for fill. However, because of their organic content, existing native sands should not be reused as backfill. Backfill material should be approved by the geotechnical engineer prior to being placed at the site. Backfill material should also meet the specifications of the CON/SPAN manufacturer.

5.2.4 Compaction of Backfill

All backfill materials should be placed in thin layers not exceeding 8 inches in uncompacted thickness, moisture conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Soils in the upper 6 inches below the pavements should be compacted to least 95 percent relative compaction.

5.2.5 Utility Trench Backfill

Bedding for utilities should be in accordance with the requirements of governing agencies. In the absence of such requirements, utilities should be bedded in granular materials such as sand or gravel, extending at least one foot above the pipe or conduit. Utility trenches in non-structural areas can be backfilled with fill and compaction achieved by mechanical means. Jetting should not be allowed for compaction. Utility trench backfill should be placed in thin lifts of 6 inches or less loose thickness, uniformly moisture conditioned as described above, and compacted to 90 percent relative compaction and to at least 95 percent relative compaction in the upper 6 inches below pavements.

5.3 Shallow Foundations

5.3.1 General

Spread footings bottomed in existing medium dense native sands can be used to support the proposed new bridge. The top of the footings should be placed below the scour elevation (or 3 feet below the bottom of the existing creek, whichever is greater), and should be a minimum of 18 inches in width and thickness.

5.3.2 Bearing Pressures

Shallow footings, founded on a compacted native sand subgrade, can be designed using an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus long-term live loads (Factor of Safety, FS=3). For total loading conditions, including seismic or wind forces, this allowable bearing pressure value can be increased to 4,500 psf (FS=2).

Standing water should not be allowed to collect in foundation excavations. If ponding does occur, excavations should be pumped free of standing water and checked for soft zones. Prior to concrete placement, any soft or disturbed zones should be over-excavated and replaced with compacted fill or lean concrete.

5.3.3 Settlement

For the allowable bearing pressures given above, static settlements are expected to be less than approximately one inch. Differential settlements should be less than half an inch. As discussed in Section 5.1, the new footings could additionally settle as much as an inch during a strong earthquake (and perhaps as much as 6 inches if the native medium dense sands are greater than 20 feet deep).

5.3.4 Lateral Resistance

Resistance to lateral loads can be derived from a combination of: 1) passive resistance acting on the faces of foundation elements perpendicular to the direction of motion, and 2) friction acting between the bottom of the foundation and the supporting subgrade. We recommend using an equivalent fluid pressure of 300 pounds per cubic foot (pcf) to compute passive resistance, and a friction coefficient of 0.30 applied to dead loads to compute base friction for the native soil or new-engineered fill. The above values include a

FS of 1.5 and assume that the soil adjacent to and below the foundation consists of native sands or compacted fill. At the perimeter of foundations, not adjacent to the roadway embankment, passive resistance from the soils above scour elevation (or minimum the top 12 inches) should be ignored when calculating passive resistance.

5.4 Lateral Earth Pressures

Cantilevered permanent retaining wall (such as the wingwalls), free to displace or rotate, should be designed to resist active lateral earth pressures corresponding to an equivalent fluid density of 35 pcf. Walls fixed against rotation and translation should be designed to resist at-rest lateral earth pressures corresponding to an equivalent fluid density of 50 pcf. The above pressures are for positively drained walls (weepholes or full back drains) with level backfill and do not include hydrostatic pressure. Walls with sloping backfill should be evaluated on a case-by-case basis. The above at-rest and active lateral earth pressures do not include a factor of safety.

The retaining walls should be designed for traffic surcharge. Generally, retaining walls should be designed for increased lateral pressures due to vertical surcharge forces within a distance H (wall height in feet) from the back of the walls. For a uniform vertical surcharge pressure (Qs), we recommend assuming additional lateral pressures of 0.5Qs on the full height of the retaining wall.

Additional lateral pressure during earthquake shaking can be estimated with a triangular pattern with a zero pressure at the base and a maximum pressure of 15H psf per linear foot of wall at the top (where H is the height of the wall).

5.5 Flexible Asphalt Pavements

Pavement design thicknesses were calculated based on the Caltrans Design Procedure, and R-Value of 60 for the embankment soils. Resistance value tests (R-value). A range of traffic indices (TI) from 6 to 9 was used determine the following flexible pavement design thicknesses for sections with and without an aggregate base layer:

Traffic Index	Asphalt Concrete Thickness (in)	Aggregate Base Thickness (in)
6	5.0	-
	3.5	4.0
7	6.0	-
	4.0	4.0
8	7.0	-
	4.5	4.0
9	8.0	-
	5.5	4.0

Prior to subgrade preparation, all utility trench backfills should be properly placed and compacted. The upper six inches of subgrade soil should be rolled to provide a smooth, unyielding surface and compacted to at least 95 percent relative compaction. Class 2 Aggregate Base (if used) should have an R-value of at least 78 and conform to the requirements in the Caltrans Standard Specifications. Aggregate base should be placed in thin lifts (8-inch maximum loose lifts) in a manner to prevent segregation, uniformly moisture conditioned, and compacted to at least 95 percent relative compaction to provide a smooth, unyielding surface.

6.0 ADDITIONAL GEOTECHNICAL SERVICES DURING CONSTRUCTION

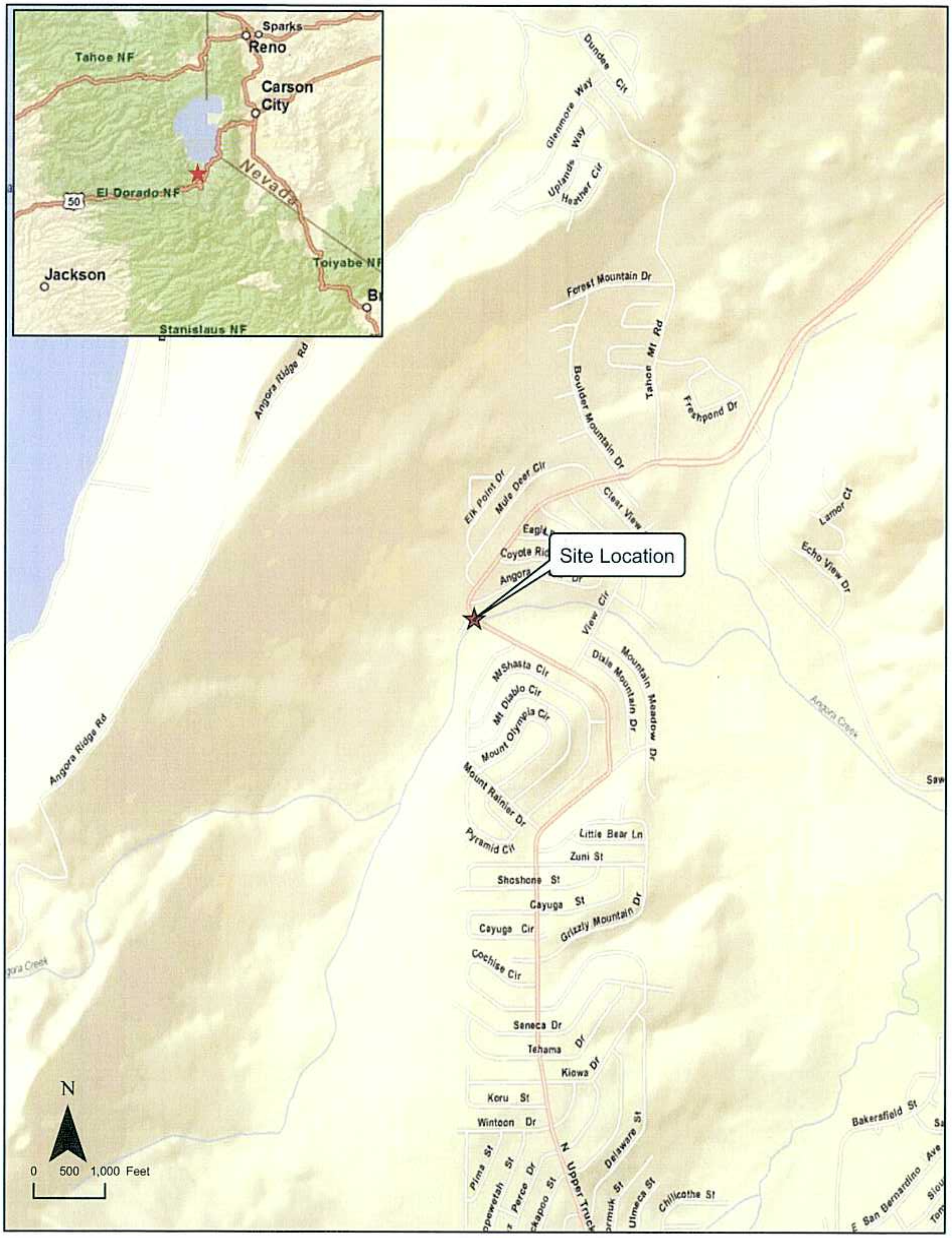
MACTEC should review the final plans and specifications during design to check for conformance with the intent of our geotechnical recommendations. We should also review the bid documents and bids during the Construction Administration Phase to check for items that could result in unnecessary risk of change orders during construction.

If changes are made in the project, the conclusions and recommendations presented in this report may not be applicable; therefore, we should review any changes to verify that our conclusions and recommendations are valid and modify them if required.

During construction, we should perform site visits as needed to check geotechnical aspects of the work and perform quality control testing of the following work items:

- Observe the stripping and excavation operations for proper removal of all unsuitable materials;
- Observe and test the compacted subgrades prior to placement of concrete or compacted fills;
- Evaluate the suitability of on-site and imported soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary;
- Observe and test backfilling for the CON/SPAN structure and for utility trenches; and
- Observe and test subgrade compaction, placement, and compaction of aggregate base for pavements.

PLATES



2006-07-01, "DigitalGlobe", 1:4000, 0.3m, "Color"
ESRI ArcGIS Online and data partners including USGS and © 2007 National Geographic Society

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Vicinity Map
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

Plate **1-1**

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
RL	4308080010	RH	01/2009	[signature]	1/09

\\Oakland-m1\projects\4308\080010_Angora Fisheries\GIS

Reference: Site Plan recieved from Ms. Amy Dillon of El Dorado County, on January 14, 2009.

NO.	DATE	REVISIONS	BY	CHK

DRAWN: RL | PROJECT NO: 4308-08-0010
ENGINEER: | SCALE: As Shown
CHECKED: RH | APPROVED:
DATE: 1/29/09 | DATE:

MACTEC

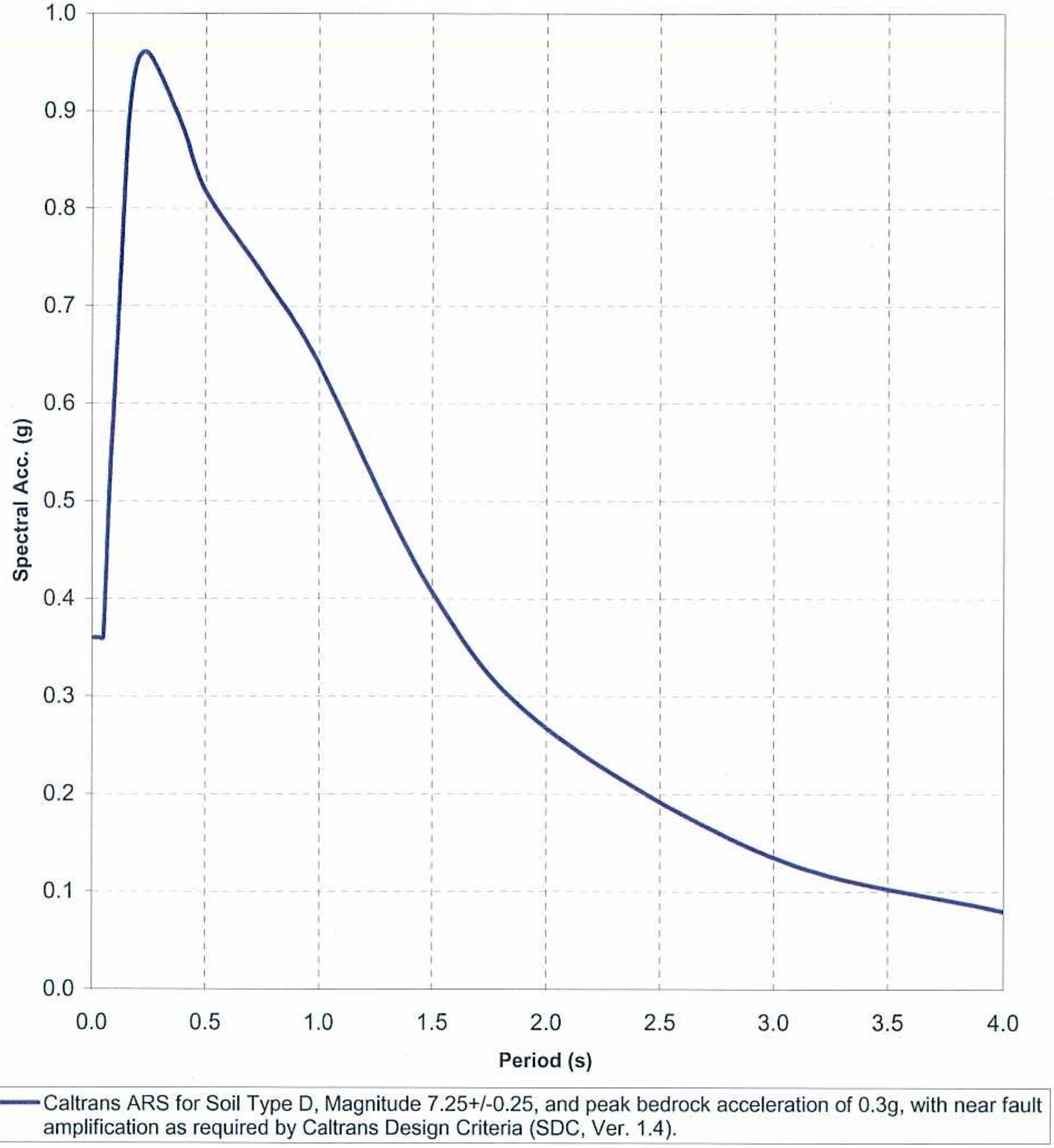
Angora Creek Fisheries/SEZ Enhancement Project

South Lake Tahoe, CA

Site Plan

PLATE: 1-2
SHEET: 1 OF 1
REVISION NUMBER: 0
DATE: 1/2009

PLANS-SITEPLAN.DWG 20.0



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Caltrans ARS Sepctrum
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

Plate **4-1**

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED DATE
RH	4308080010	RH/DKQ	1/09	

APPENDIX A

BORING LOGS

Checked RH

Approved DWJ

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488-93

MAJOR DIVISIONS			SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS OVER 50% RETAINED ON No.200 SIEVE SIZE	GRAVELS MORE THAN 1/2 OF COARSE FRACTION RETAINED ON No.4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVELS WITH OVER 15% FINES	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS MORE THAN 1/2 OF COARSE FRACTION PASSING No.4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW	Well-graded sand or gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH OVER 15% FINES	SM	Silty sand, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS OVER 50% PASSING No.200 SIEVE SIZE	SILTS & CLAYS LIQUID LIMIT 50% OR LESS		ML	Inorganic silts and sandy or gravelly silts, rock flour
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS & CLAYS LIQUID LIMIT GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays and silty clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

	NX Core Sampler
	SPT Sampler
	Sprague & Henwood Sampler
	Direct Push
	Pitcher Barrel
	Grab or Bulk Sample
	G.W. measured after water level stabilizes
	G.W. measured during or soon after drilling
Perm	Permeability
Consol	Consolidation
LL	Liquid Limit (%)
PI	Plasticity Index (%)
EI	Expansion Index (%)
Gs	Specific Gravity
MA	Particle Size Analysis
-200=55%	Percent Passing No. 200 Sieve

Shear Strength (psf) → ; Confining Pressure →

TxUU 3200 (2600) (FM) or (S)	-Unconsolidated Undrained Triaxial Shear (field moisture or saturated)
TxCU 3200 (2600) (P)	-Consolidated Undrained Triaxial Shear (with or without pore pressure measurement.)
TxCD 3200 (2600)	-Consolidated Drained Triaxial Shear
SSCU 3200 (2600) (P)	-Simple Shear Consolidated Undrained (with or without pore pressure measurement.)
SSCD 3200 (2600)	-Simple Shear Consolidated Drained
DSCD 2700 (2000)	-Consolidated Drained Direct Shear
UC 470	-Unconfined Compression
LVS 700	-Laboratory Vane Shear

KEY TO TEST DATA

Source: ASTM D 2488-93, based on Unified Soil Classification system

SOIL_CLASS_GEOTECH_MACTEC KET TO TEST DATA.GPJ GEOTECH.GDT 1/23/09

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Soil Classification Chart and Key to Test Data
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

PLATE **A-1**

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	APPRV'D DATE
RL	4308080010	RH	1/09	DWH	1/09

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	Standard Penetration Test Blow Count (blows per foot)
very loose	<4
loose	4-10
medium dense	10-30
dense	30-50
very dense	>50

CONSISTENCY OF FINE-GRAINED SOILS

Consistency	Approximate Blows/foot (SPT)	Undrained Shear Strength (psf)
very soft	<2	0 - 250
soft	2-4	250 - 500
medium stiff	4-8	500 - 1,000
stiff	8-15	1,000 - 2,000
very stiff	15-30	2,000 - 4,000
hard	>30	>4,000

NATURAL MOISTURE CONTENT

Dry	- Requires considerable moisture to obtain optimum moisture content for compaction
Moist	- Near the optimum moisture content for compaction
Wet	- Requires drying to obtain optimum moisture content for compaction

Note: Where laboratory data are not available, the field classifications given above provide a general indication of material properties; the classifications may require modification based on judgment or laboratory testing.

PHYSPROPS-SOIL.DWG 40.0
20070208.1259

MACTEC

Physical Properties Criteria for Soil Classification
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

PLATE: **A-2**

DRAWN	JOB NUMBER	CHECKED	CHECKED DATE	APPROVED	APPROVED DATE
RH	4308080010	RH	1/09	DMG	1/09

Date 1/5/09
Equipment
Drilling Method Hollow Stem Auger
Sampler S&H
Hammer Weight 140 lbs **Drop** 30 inches
Logged by DC **Datum** MSL
Surface Elevation 6342 feet

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Blows per Foot	Depth (ft.)	Sampler Type	Graphic Log	Description
						0			Asphalt Concrete (8")
R-Value, MA, -200=16.4%	9	106.4			46/5"*				LIGHT BROWN SILTY SAND (SM) with gravel, wet (FIll) Dark brown, dense
	12.5	103.9			51*	5			
-200=36.5%	14.4	106.6			18*				Medium dense
-200=13.4%	21.8	97.5			12*	10			DARK GRAY SILTY SAND (SM) with organics, medium dense, wet ▽ Groundwater measured @ 11 feet before backfilling
Organic Content= 9.34% -200=27.8%					15*				▽ Groundwater encountered @ 14 feet during drilling
-200=25%					15*	15			
-200=22.1%					10*				
						20			Bottom of boring at 20 feet below ground surface. Boring was backfilled with cement grout.

GEOTECH_BORING_NEW_MACTEC BORING LOG.GPJ GEOTECH.GDT 1/28/09

* S&H Sampler blow counts were converted to approximate SPT N-values using a conversion factor of 0.8.

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Log of Boring B-1
Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

PLATE **A-3**

DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
RH	4308080010	RH	1/09	DWQ	1/09

Date 1/5/09
Equipment
Drilling Method Hollow Stem Auger
Sampler S&H
Hammer Weight 140 lbs **Drop** 30 inches
Logged by DC **Datum** MSL
Surface Elevation 6343 feet

Other Tests/Drilling Notes	Moisture Content (%)	Dry Density (pcf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Blows per Foot	Depth (ft.)	Description
						0	Asphalt Concrete (8")
R-Value, MA, -200=18.2%							LIGHT BROWN SILTY SAND (SM), wet (FIll)
	9.8	97.6			40/3"*		Dark brown, dense
	11.9	103.6			48*	5	
-200=27.3%	12.8	103.6			16*		Dark Gray/Brown, Medium dense
						10	DARK GRAY SILTY SAND (SM) with organics, mdeium dense, wet
Organic Content=2.75% -200=21.1%					12*		
-200=20.3%	13.2	110.7			13*		
						15	
					26*		with gravel
					49*		LIGHT BROWN SAND (SP), dense, wet
						20	Bottom of boring at 20.25 feet below ground surface. Groundwater was not encountered. Boring was backfilled with cement grout.

* S&H Sampler blow counts were converted to approximate SPT N-values using a conversion factor of 0.8.

GEOTECH_BORING_NEW_MACTEC BORING LOG.GPJ GEOTECH.GDT 1/28/09

MACTEC

Log of Boring B-2

Angora Creek Fisheries/SEZ Enhancement Project
South Lake Tahoe, California

PLATE **A-4**

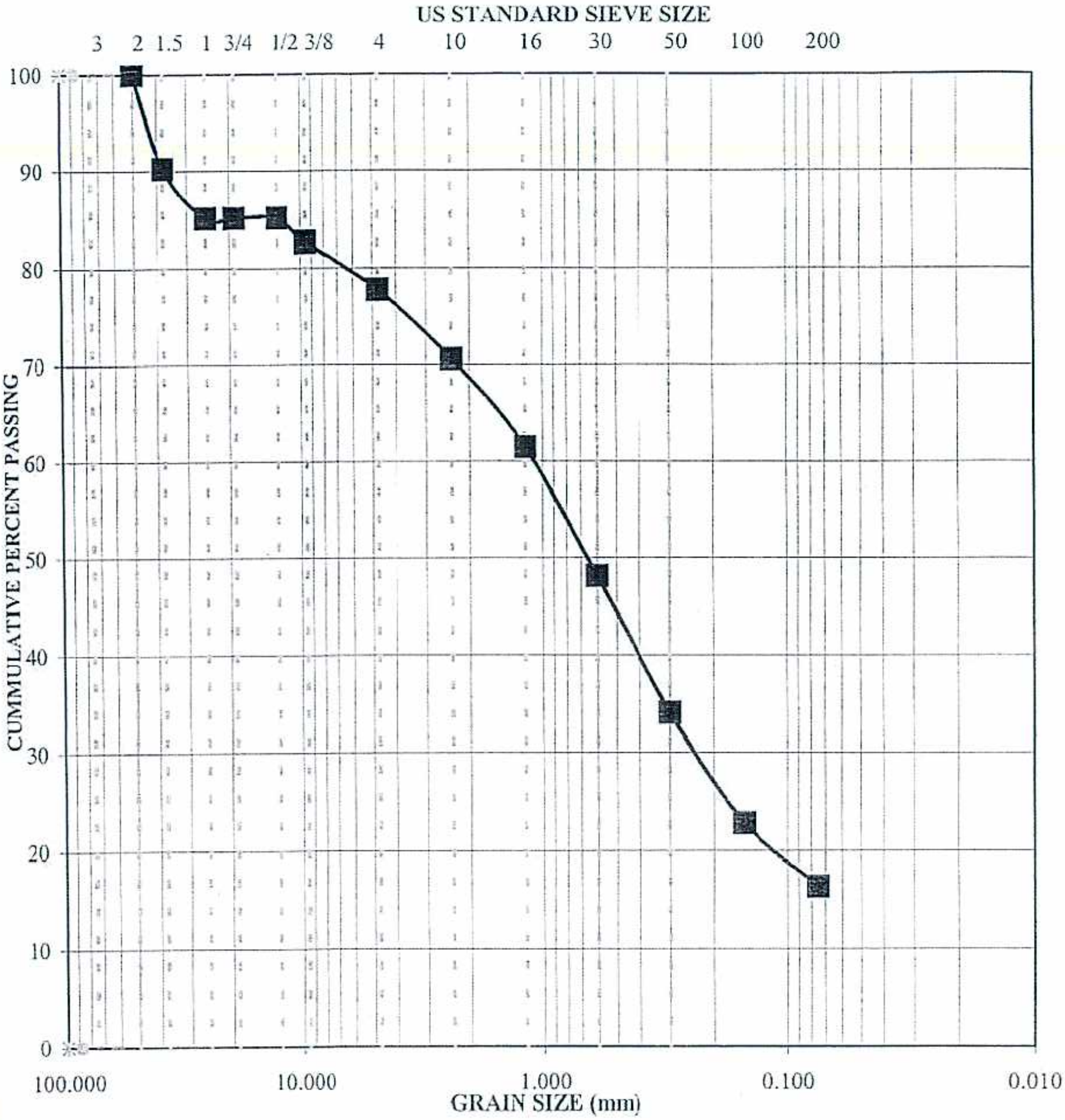
DRAWN	JOB NUMBER	CHECKED	CHCK'D DATE	APPROVED	CHCK'D DATE
RH	4308080010	RH	1/09	DRW	1/09

APPENDIX B

LORATORY TEST RESULTS

Checked RA

Approved DWQ



COBBLES	GRAVEL	SAND	SILT OR CLAY
	22.2	61.4	16.4

Symbol	Sample Source	Classification / Description
■	B-1 @ 0.8' - 1.3'	Sub-Base

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Fax (775) 322-9380

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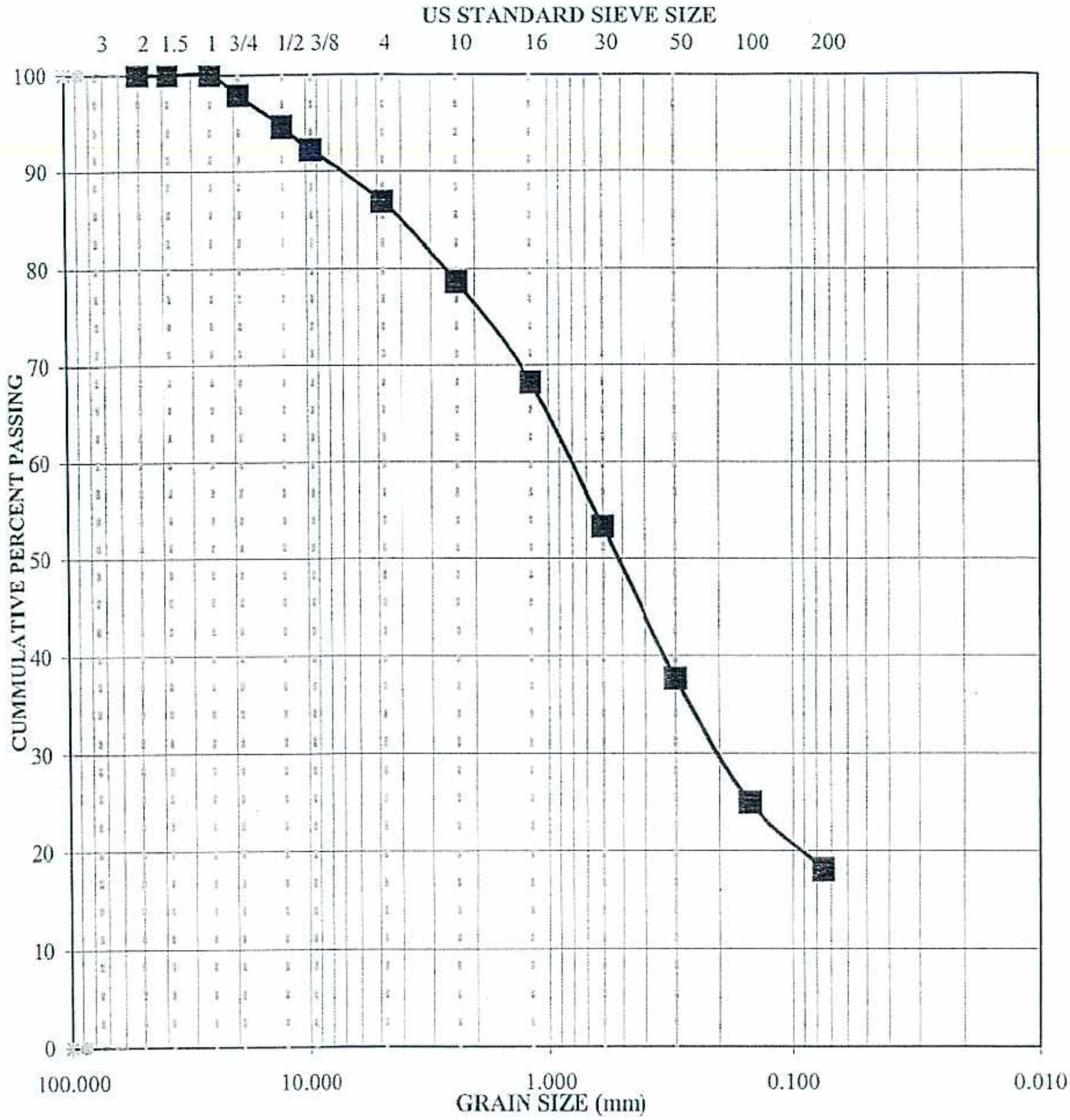
SIEVE ANALYSIS
Angora Creek Fisheries Bridge

JOB NUMBER	APPROVED	DATE	REVISED	DATE
4308080010	CE			D.C.



COBBLES	GRAVEL	SAND	SILT OR CLAY
	13.2	68.7	18.2

Symbol	Sample Source	Classification / Description
■	B-2 @ 0.8' - 1.5'	Sub-Base

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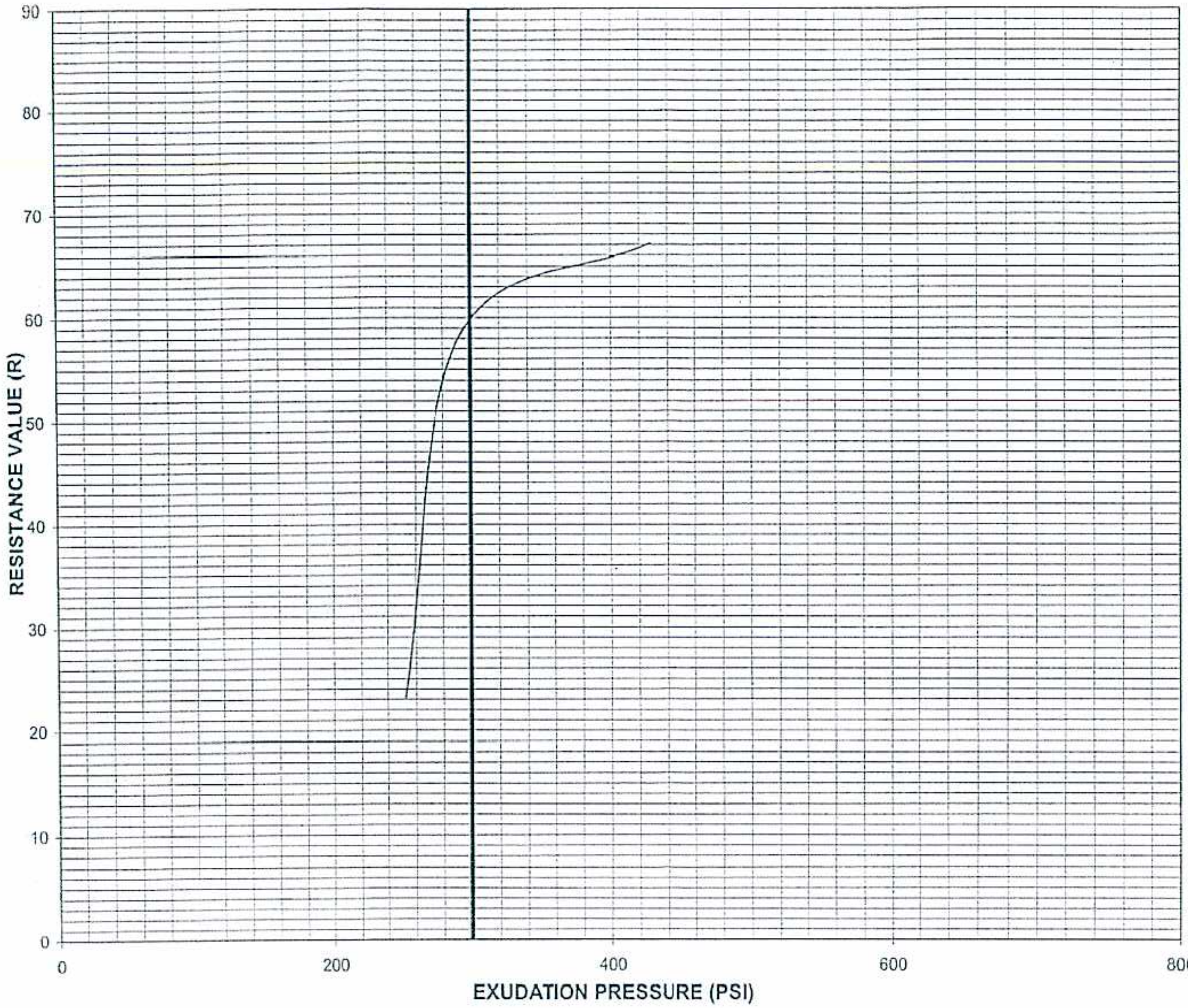
SIEVE ANALYSIS

Angora Creek Fisheries Bridge

JOB NUMBER	APPROVED	DATE	REVISED	DATE
4308080010	CR			D.C.

R-Value and Expansion Pressure of Compacted Soils AASHTO T190 / ASTM D2844



Bulk #	Sample Source	Classification	Expansion Pressure (psf) @ 300 (psi)	R-Value @ 300 (psi)
9267	B-1 @ 0.8' - 1.3'	Medium Brown Silty Sand With Gravel	0	60

POINT #	WATER CONTENT (%)	DRY DENSITY (PCF)	EXUDATION PRESS. (PSI)	EXPANSION PRESS. (PSF)	RESISTANCE VALUE (R)
1	11.0	126.2	253	0	23
2	9.9	125.7	293	0	58
3	9.3	129.7	428	0	67
4					
5					

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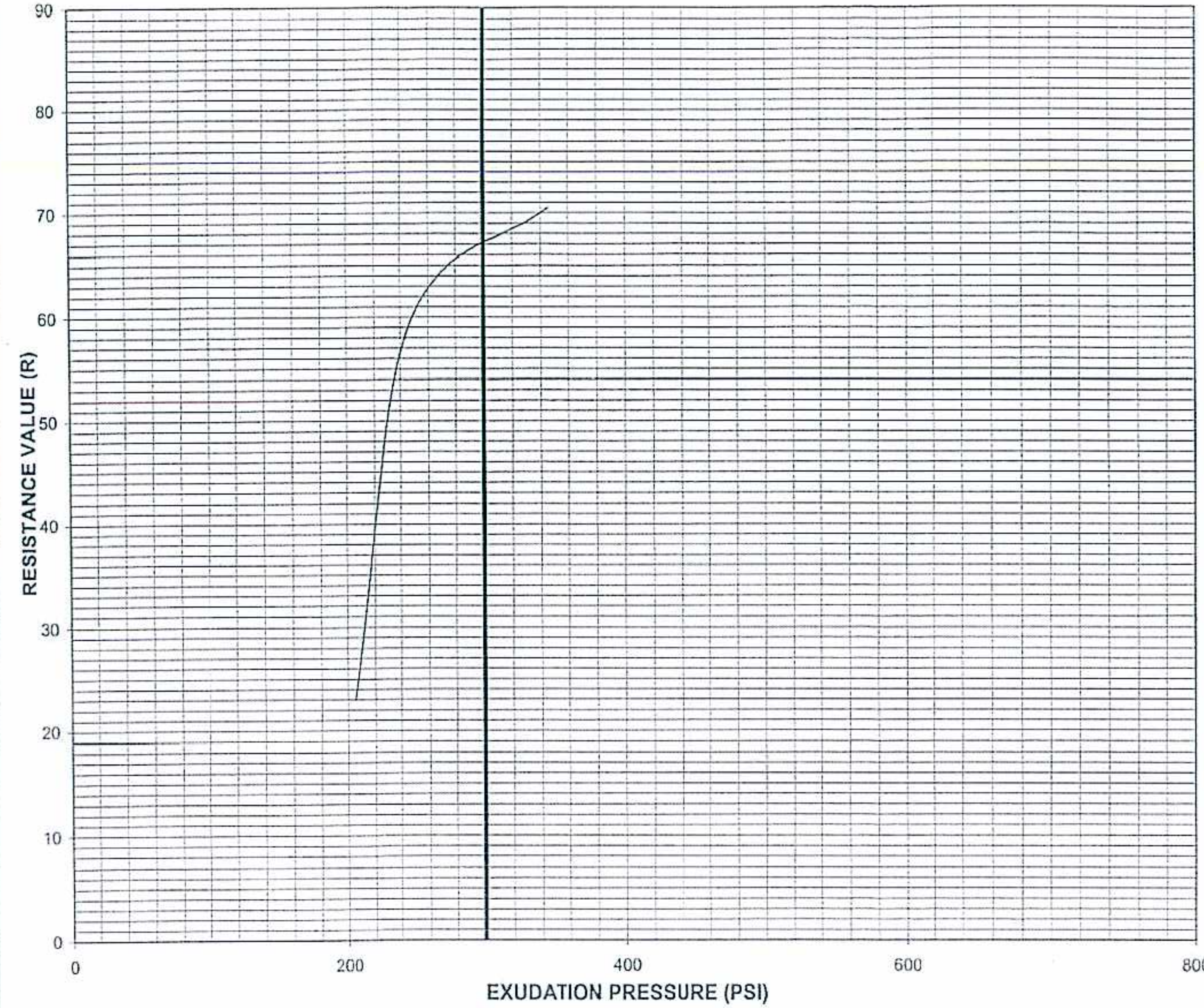
RESISTANCE VALUE TEST DATA
Angora Creek Fisheries Bridge

TESTED BY	JOB NUMBER	APPROVED	DATE	REVISED	DATE
CE	4308080010		1/12/2009		

R-Value and Expansion Pressure of Compacted Soils AASHTO T190 / ASTM D2844



Bulk #	Sample Source	Classification	Expansion Pressure (psf) @ 300 (psi)	R-Value @ 300 (psi)
9267	B-2 @ 0.8' - 1.5'	Medium Brown Silty Sand With Gravel	0	67

POINT #	WATER CONTENT (%)	DRY DENSITY (PCF)	EXUDATION PRESS. (PSI)	EXPANSION PRESS. (PSF)	RESISTANCE VALUE (R)
1	10.9	126.3	205	0	23
2	10.3	127.0	248	0	60
3	9.7	128.8	345	0	71
4					
5					

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RESISTANCE VALUE TEST DATA
Angora Creek Fisheries Bridge

TESTED BY	JOB NUMBER	APPROVED	DATE	REVISED	DATE
CE	4308080010		1/12/2009		