Appendix J1-3

Preliminary Geotechnical Investigation: Power Plant, Tracy, California

A California Corporation
Specializing in Geotechnical Engineering

Hultgren-Tillis Engineers

PRELIMINARY GEOTECHNICAL INVESTIGATION

POWER PLANT
TRACY, CALIFORNIA

Project No. 474.01
June 26, 2001

2520 Stanwell Drive, Suite 100 • Concord, California 94520
Phone (925) 685-6300 • Fax (925) 685-6768

A California Corporation
Specializing in Geotechnical Engineering

June 26, 2001
Project No. 474.01

GWF Power Systems Company, Inc.
4300 Railroad Avenue
Pittsburg, California 94565

Attention: Mr. Mark Kehoe

Preliminary Geotechnical Investigation
Power Plant
Tracy, California

Dear Mr. Kehoe:

We performed a preliminary geotechnical investigation for the power plant off Lammers Road in Tracy, California in accordance with the Contract Agreement No. 01-0517 dated June 14, 2001. The results of the investigation are presented in the attached report.

It was a pleasure working with you on this project and we look forward to working with you during final design and during construction. If you have any questions, please call.

Sincerely,

Hultgren – Tillis Engineers

Hugh D. Davis
Project Engineer

R. Kevin Tillis
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
R. KEVIN TILLIS
No. GE 2160
EXP. 3/31/03
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STATE OF CALIFORNIA

HDD:RKT:EH:la

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I. INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the planned power plant in Tracy, California. The project consists of constructing a 250 MW natural gas fueled electrical generation plant on a 40-acre parcel. The new plant will be designed as a "peaker" plant, delivering power during periods of high demand. The exact location of the site within the 40-acre parcel has not been determined. We understand that the plant will be sited on about 5 acres within the larger parcel. We understand that the power plant will consist of typical structures for plants including a turbine, cooling towers, pipe racks, buildings and various tanks and vessels. Portland cement concrete slabs will likely surround the structures. Asphalt paved roadways are planned. Storm water will be discharged into one or more basins that will be created on the site.

The site is located about 3,500 feet west of Lammers Road. The site is bounded by the Delta-Mendota Canal to the south, a railroad to the north and fallow fields to the east and west. The site is shown on the Site Plan, Plate 1. A Vicinity Map is provided on Plate 1.

Our scope of services was outlined in the Contract Agreement No. 01-0517 dated June 14, 2001 and included drilling borings and pushing cone penetration test (CPT) probes, performing percolation tests, performing laboratory tests and developing preliminary conclusions and recommendations regarding geotechnical aspects of the project. The investigation provides preliminary geotechnical information for the 40-acre parcel. A final investigation is needed after the location of the power plant within the site is defined. The results of our preliminary geotechnical investigation are presented in this report.

II. FIELD EXPLORATION AND LABORATORY TESTING

We explored subsurface conditions on June 11 through June 13, 2001 by drilling 4 borings to depths of 59 feet to 79.5 feet, pushing 6 CPT probes to depths ranging from 50 feet to 100 feet and performing 2 percolation tests. The locations of the borings, CPT probes and percolation tests are shown on the Site Plan, Plate 1. The locations shown on the Site Plan were surveyed by LCC, Inc. The borings were drilled with truck-mounted, auger and rotary wash drill equipment. Samples were collected with a Pitcher Barrel Sampler equipped with 3-inch diameter shelby tubes and also with a 2.5-inch outside diameter, 1.9-inch inside diameter sampler. The 2.5-inch sampler was driven with a 140-pound hammer falling approximately 30 inches for a penetration depth of 18 inches. The hammer utilized a rope-and-pulley system. Our engineer recovered samples from the borings and returned them to our office for further examination and selected testing. Stiffness of the materials was checked with a pocket penetrometer. The borings and CPT probes were backfilled with cement grout. The CPT data is presented on Plates 2 through 7 and Logs of Borings are presented on Plates 8 through 18.

The laboratory testing program consisted of Atterberg limits, moisture content, dry density, mechanical sieve analysis and unconsolidated-undrained triaxial shear strength tests. Soil descriptions, equivalent standard penetration resistance, strength measured by the pocket penetrometer and a summary of laboratory tests is presented on the Logs of Borings, Plates 8 through 18. The field blow counts were corrected to approximate Standard Penetration Test N-values by multiplying by 0.8 for presentation on the Logs of Borings. The mechanical sieve analysis and triaxial strength tests are presented on Plates 19 through 21.

Two percolation tests were performed for preliminary design of storm water detention ponds. The percolation tests consisted of drilling a 10-inch diameter boring to a depth of 10 feet and placing about 2 inches of sand at the bottom of the boring. A 4-inch diameter slotted PVC casing was inserted in the boring and the annular space around the pipe was filled with sand. We filled the pipes with water and left the water in the pipe overnight. We adjusted the water level to a depth of about 1 foot above the bottom of the casing on the following day. We then monitored the drop in the water level at about ½-hour intervals for 4 hours. The drop in water level was relatively consistent over the 4 hours. The test results indicate a percolation rate of 1.0 foot per day and 0.5 foot per day for percolation test sites 1 and 2, respectively.

III. SITE CONDITIONS

A. Regional Geology and Seismicity

We understand that geologic hazards are being addressed by others under separate cover. The following is a cursory description of the site geology and seismicity. The site is located on the western edge of the Great Valley of Central California, approximately astride the boundary between the Great Valley to the east, and the Coast Ranges to the west. The region is underlain by a complex series of sedimentary and volcanic rocks ranging in age from Jurassic to Tertiary. Since their deposition, these rocks have been extensively deformed by repeated episodes of folding and faulting.

Valleys within the region are generally filled with unconsolidated sedimentary deposits of Quaternary age. A thick sequence of alluvial fan deposits forms the west side of the Great Valley in the Tracy area. These sedimentary deposits probably consist of interbedded sand, gravel, silt, and clay.

B. Surface Conditions

The site is currently a fallow field. The field has not been farmed in recent years. Short grasses cover the site. The surface slopes gently down at about a one- percent grade to the northeast. The elevation of the site varies from about 180 feet to 155 feet. Levees for the Delta-Mendota Canal are present on the south edge of the site. A culvert extending beneath the canal is located on the west end of the site. Overhead electrical lines cross the southeast corner of the site. Three underground pipelines cross the middle of the site, trending southeast to northwest.

C. Subsurface Conditions

Subsurface conditions consist of a layer of moderately to highly expansive clay underlain by an alluvial sequence of silt, clay, sand and gravel. The surface expansive clay layer varies from about 2 feet to 7 feet thick. The material is loose to a depth of 1 feet to 1.5 feet and stiff to hard below this depth. The material directly below the clay consisted of 4 feet to 7 feet of silty sand at four locations and sandy silt or sandy clay at the other locations. This layer is underlain mainly by sandy silt to silty clay to the depths explored. The clay and silt is typically very stiff to hard and contained varying amounts of sand and gravel. Occasional layers of sand and gravel are present to the depths explored. We encountered two layers of dense sand and gravel at depths of about 30 feet and 50 feet below the ground surface. The two layers appear to be relatively continuous across the site. Two of the CPT probes met refusal in the layer at 50 feet.

We noted a slightly weaker clay at depths of 70 to 85 feet below ground surface. Occasional thinner zones of weaker soil were noted at various depths within the CPT probes. Although these materials are weaker than other areas, they are still classified as very stiff.

We used rotary wash drilling for the borings. The rotary method masks the groundwater levels and does not allow for a reliable measure of the groundwater level. To assess groundwater levels, we monitored the pore pressure dissipation at three of the CPT locations. We estimate the depth to groundwater is 50 feet at CPT 1, 34 feet at CPT 4 and 25 feet at CPT 6. This corresponds to groundwater elevations ranging from about 125 feet to 142 feet.

The above descriptions of soil and groundwater conditions summarize observations at the time of our investigation. Conditions are expected to vary across the site and with time and depend on several factors including changes in moisture content resulting from seasonal precipitation and land use changes.

IV. DISCUSSION AND CONCLUSIONS

A. General

The most significant geotechnical concerns for the project include the presence of expansive soil and the potential for compression of subsurface materials under heavy loads. These and other geotechnical considerations are discussed below. We conclude that conventional asphalt concrete pavements or portland cement concrete pavements may be used at the site. We can provide design thicknesses after traffic loading and traffic patterns are established during final design.

B. Expansive Soil

The soil profile consists of a surface expansive clay layer underlain by relatively dry soils to depths up to 50 feet. Atterberg limits tests indicate that the surface clay layer has a moderate to high expansion potential. The underlying materials have a lower potential for expansion. Expansive soils change volume with changes in their moisture content. As the moisture content is increased, expansive soil swells; as expansive soil dries, it shrinks. Moisture content increases during winter months and/or from heavy irrigation. Moisture content decreases from summer drying and/or extraction by tree root systems. Structures located directly on expansive soils will heave and settle in response to these movements. Placing a slab over expansive soil will cut down evapo-transpiration losses during dry months, tending to retain moisture content beneath the center portion of the slab. The moisture content near the edges of the slab tends to vary with the season and with irrigation practices.

We conclude that several alternatives are available to reduce the impacts of expansive soil on the plant to levels that would be considered acceptable to reasonable persons. The first measure is to soak and wet the existing soils to cause them to swell prior to constructing structures. Another measure includes removing the highly expansive surface layer and replacing it with material with low expansion potential. We believe that this alternative will be cost effective where the expansive soil is less than several feet thick.

Other methods include placing select fill of low expansion potential below the slabs and foundations. The purpose of the select fill is to provide a buffer zone between the expansive materials and concrete slabs. This zone will aid in reducing seasonal moisture variation in the expansive soils. The select fill will also help spread differential movement over a larger area and reduce the stress to the slabs. The material located below the expansive clay surface layer consists of lean clay and silty sands. The sands and possibly the lean clay will be suitable for use as select fill. The excavations for the storm water basin will extend into these materials and provide a potential source of select fill. Another method that could be considered is to treat the expansive soil with lime.

The various options will be considered in final design after the site is selected and additional exploration and laboratory testing is completed. Expansive surface soils are a typical condition in this portion of the coast range. The methods described above for controlling the effects of expansive soil are commonly used by engineers and contractors in this area.

C. Foundations

The subsurface conditions consist predominately of very stiff clay and dense sands and gravels. We judge that shallow footing and mat foundations will be suitable for support of the various structures. Detailed settlement analyses will be part of final design when estimates of structural loads are available and consolidation tests are performed on foundation materials at the selected plant site. Based on the stiffness and denseness of the materials encountered during this preliminary investigation, we conclude that foundation settlements will be small. We expect total settlements will be less than ½-inch for normally loaded footings and mats.

Though not expected, if unusually large loads need to be supported that cannot be otherwise made part of a footing or mat foundation, we conclude that these loads can be supported on deeper foundations such as drilled piers or driven piles. Drilled piers or piles can also be used to resist lateral and uplift loads for pipe racks and other structures. The type of foundation will be determined in final design after the locations and footprints are finalized.

Driven piles may be difficult to install due to the dense sand and gravel layers. Driven piles could meet refusal at relatively shallow depths and have limited axial capacities. For this reason, drilled pier foundations will probably be more cost effective and practical where deeper foundations are needed.

D. Seismicity

The site is not located within the one of the Alquist-Priolo Earthquake Fault

Zones delineated by the State of California around known active faults. The predominant seismic hazard for this site is strong groundshaking resulting from earthquakes. The building should be designed to accommodate such groundshaking in accordance with existing codes. No known active faults cross the site. We judge that the risk of fault rupture at the site is low. For use with the 1997 Uniform Building Code (UBC), the site can be classified as a S_D, a stiff soil site. The principal faults for use with the UBC include the Greenville Fault, a Type B fault, located 15 kilometers to the west, the Calaveras fault, a Type B fault, located 36 kilometers west, and the Hayward Fault, a Type A fault located 41 kilometers west. The designers can use UBC code factors of 1.0 for N_a and 1.0 for $N_{v.}$

Soil liquefaction is a phenomenon in which a loose- to medium-dense saturated granular soil undergoes reduction of internal strength as a result of increased pore water pressure generated by shear strains within the soil mass. This behavior is most commonly induced by strong groundshaking associated with earthquakes. Soil conditions consist predominately of clay and dense sands. We judge that the risk of liquefaction is low.

E. Storm Water Detention Basin

The site conditions consist primarily of fine-grained silts and clays. The presence of these materials will limit the vertical infiltration of detention ponds. The designers should consider this when using the percolation tests for preliminary design of the ponds.

The ponds will saturate the ground below and surrounding the ponds. We anticipate that the ground saturation will cause fine-grained materials to swell and heave the ground. We conclude that the basins should be sited well away from planned structures to limit the potential for heave of these structures due to infiltration of water and saturation of expansive material.

V. RECOMMENDATIONS

The following recommendations are intended for preliminary design and subject to revision during final design.

A. Earthwork

1. Site Preparation

The plant site should be stripped to sufficient depth to remove vegetation and soil containing roots. Stripped material should be removed from the plant site or stockpiled for the owner's use in landscaped or agricultural areas. These materials should not be reused as compacted fill. The upper 1 to 2 feet of loose material should be removed or recompacted prior to placing fills.

2. Fill Materials

Common fill placed at the site should be a soil or soil/rock mixture derived from onsite excavations, free of deleterious matter and contain no rocks or hard fragments larger than 4 inches in maximum dimension and less than 15 percent larger than 1-inch in maximum dimension.

In addition to meeting the requirements for common fill, *select fill* should have a low expansion potential, which for this site should be defined as having a Liquid Limit (LL) less than 40 and Plasticity Index (PI) less than 15. Select fill should be predominately granular with 100 percent passing a two-inch sieve and less than 30 percent passing the Number 200 sieve.

3. Compaction

Surfaces exposed by stripping and excavation of fill material should be scarified to a depth of at least 8 inches or the full depth of shrinkage cracks, whichever is deeper. The scarified soil should be moisture conditioned to at least 3 percent over optimum moisture content and compacted to at least 90 percent relative compaction. ASTM test D-1557 should be used to establish the reference values for computing optimum moisture content and relative compaction. If shrinkage cracks extend below 12 inches, some excavation in addition to scarifying will be required to adequately moisture condition and compact soils. If soft or yielding soils are present during subgrade preparation or fill compaction, they should be scarified, air-dried and compacted or removed by excavating to expose firm soil.

Fill should be placed in lifts 8 inches or less in loose thickness and moisture conditioned to at least optimum moisture content for select fill and at least 3 percent over optimum for common fill. Moisture conditioning should be performed prior to compaction. Each lift should be methodically compacted to at least 90 percent relative compaction. A sheepsfoot compactor or equivalent equipment should be used for compacting clay soils. Material that fails to meet the moisture or compaction criteria should be loosened by ripping or scarifying, moisture conditioned, and then recompacted.

In pavement areas, the upper 6 inches of subgrade should be compacted to at least 95 percent relative compaction and rolled to provide a smooth firm surface. Subgrade soils should be proof-rolled prior to placing aggregate base. Proof-rolling should be performed with the heaviest available rubber-tired construction equipment and should be observed by the geotechnical engineer. Soft or pumping areas should be aerated or excavated and recompacted.

Aggregate base should be placed in thin lifts no greater than 8 inches in loose thickness and in a manner that avoids segregation, moisture conditioned as necessary, and compacted to at least 95 percent relative compaction.

4. Utility Trenches

In the absence of local agency or utility company requirements, the following criteria for bedding and backfilling utility lines may be used. For terra cotta, plastic and/or metal pipes, a bedding layer consisting of clean sand or fine gravel should be placed below and around pipes and extend at least six inches above their tops. The bedding thickness below the bottom of the pipe should be at least three inches. For concrete storm drains, the above bedding criteria may be modified by extending the sand or fine gravel bedding material only up to the spring line of the pipe provided care is taken during placement and compaction of the fill around and above the pipe. Jetting should not be allowed for compacting backfill.

B. Slabs-On-Grade

Concrete slabs-on-grade should be underlain by 6 inches of aggregate base where storage and/or traffic is planned. Within areas sensitive to moisture transmission through the slab such as a control room, the concrete slabs-on-grade should be underlain by at least 4 inches of capillary gravel. Capillary gravel used to create a moisture break beneath slabs-on-grade should be a clean, uniform graded crushed aggregate satisfying the requirements of Standard Aggregate No. 67 in accordance with ASTM test D-448. A vapor barrier such as a plastic membrane should be installed between the slab and capillary gravel. As an option, the vapor barrier may be overlain by a 2-inch-thick sand blanket to aid in protecting the barrier against puncture and to improve concrete curing. Even with these measures, some minor moisture transmission should be expected.

Special site preparation measures may be needed below the capillary barrier or aggregate base to reduce the potential for heave of the expansive soil. These measures will be developed during final design.

C. Foundations

1. Footings and Mats

Continuous footings should be at least 12 inches wide, and individual footings should be at least 18 inches wide. Footings should be founded at least 24 inches below lowest adjacent finished grade. Mats should be founded at least 12 inches below grade. Footings and mats should be well reinforced. The faces of foundation excavations should be cut vertical. The bottom of excavations should be firm and free of water, debris, and loose or soft soils. Excavations should be kept moist until concrete is placed.

Footings should be designed using allowable bearing pressures of 3,500 pounds per square foot (psf) for dead loads, 4,000 psf for dead plus sustained live loads with a one-third increase for total loads including wind or seismic forces. A modulus of subgrade reaction of 100 tons per cubic foot can be used for preliminary design.

Resistance to lateral loads can be developed by friction at the base of foundations and passive pressures acting against the vertical faces of below grade foundation elements. Frictional resistance on the base of foundations can be calculated using a frictional coefficient of 0.40 multiplied by the vertical dead load. An equivalent fluid weight of 350 pounds per cubic foot (pcf) may be used to calculate sustained passive resistance and 450 pcf may be used for passive resistance to transient loads. Passive pressure should be neglected in the upper one foot of soil unless concrete slabs or pavements confine the adjacent surface. These lateral resistance values do not include a factor of safety.

2. Drilled Piers and Driven Piles

Drilled piers should have a minimum diameter of 18 inches and should extend at least 15 feet below lowest adjacent finished grade. Precast prestressed concrete piles should be at least 14-inch square and extend at least 25 feet below grade. Drilled piers or driven piles should be designed to resist vertical loads by skin friction between the perimeter of the pier or piles and the surrounding ground. For dead plus live loads, we recommend using an allowable skin friction value of 700 psf beginning five feet below final grade. This value may be increased by one third for transient forces, including earthquake and wind loads. Piers or piles should be spaced no closer than 3 pier widths, measured center to center.

Passive pressure acting on the face of the piers can resist lateral loads on the piers. An equivalent fluid weight of 350 pounds per cubic foot (pcf) up to a maximum of 1,500 psf may be used to calculate sustained passive resistance. The passive pressure may be assumed to act over two pier diameters. In expansive clay areas the upper three feet should be ignored when calculating passive resistance. The lateral resistance values should be used with an appropriate factor of safety when assessing the lateral capacity of the pier foundation.

Pier holes should be cleaned of loose material prior to casting the piers. Concrete for the piers should be placed within 24 hours of drilling the piers and the holes should be covered for the interim.

If water is encountered in the pier holes, the water should be removed from the pier excavations prior to placing concrete or the concrete should be placed through a tremie pipe placed to the bottom of the pier excavation.

D. Retaining Walls

Retaining walls should be designed for lateral soil pressures. Where walls are free to rotate at their top, we recommend that the walls be designed for an equivalent fluid pressure of 40 pcf. For walls that are restrained at the top we recommend that the walls be designed for an equivalent fluid pressure of 60 pcf. These values assume that drainage is provided behind the wall and that wall backfill is level and consists of select fill. It does not include an allowance for surcharge loads or earthquake pressures.

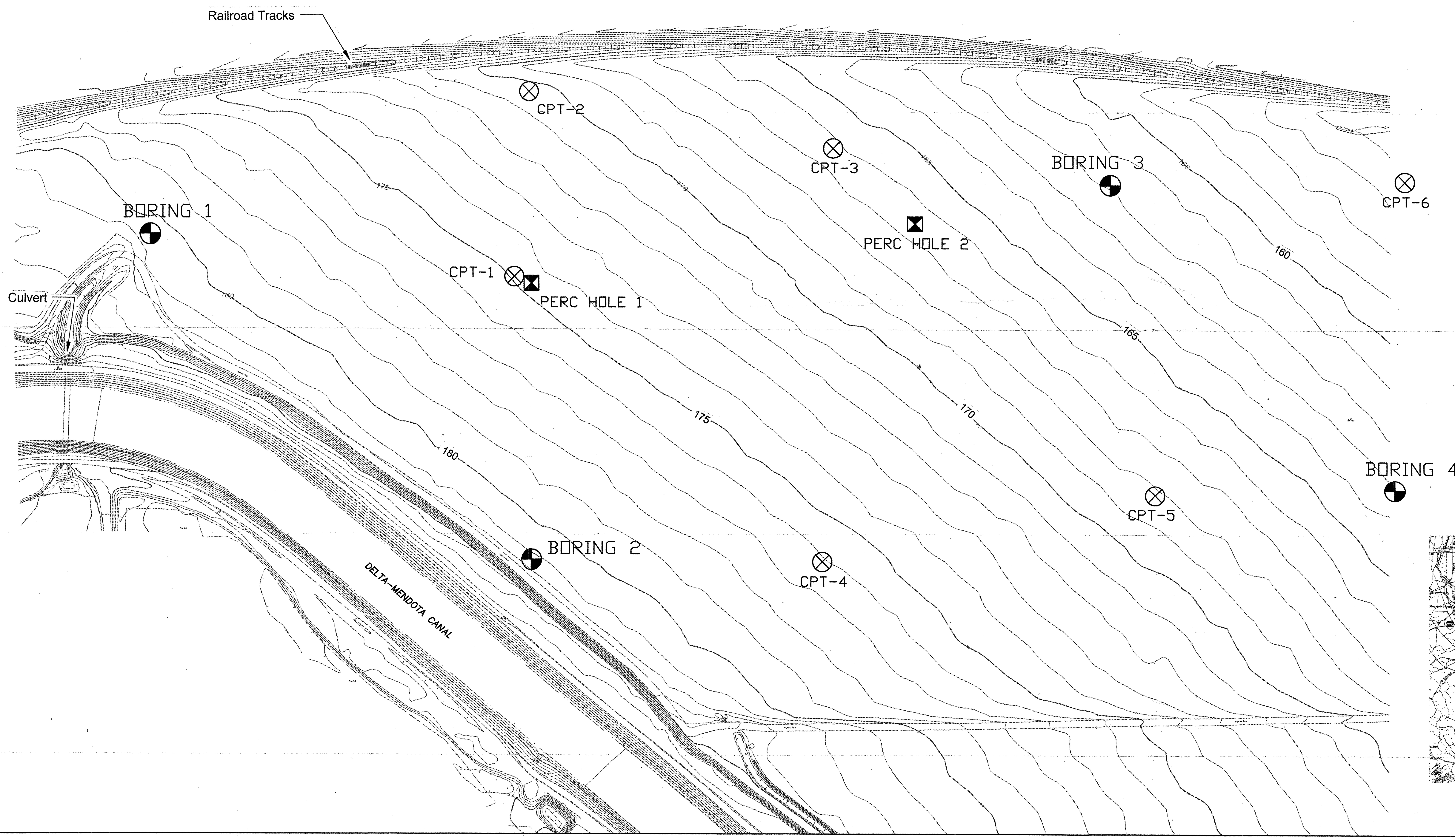
To account for increased loading due to seismic forces, an additional pressure of 10H psf (rectangular distribution) may be added to the pressure given above where H is the height in feet of the retaining wall. A surcharge pressure of 200 psf extending to a depth of 5 feet should be applied to walls founded within 5 feet of traffic lanes or parking areas.

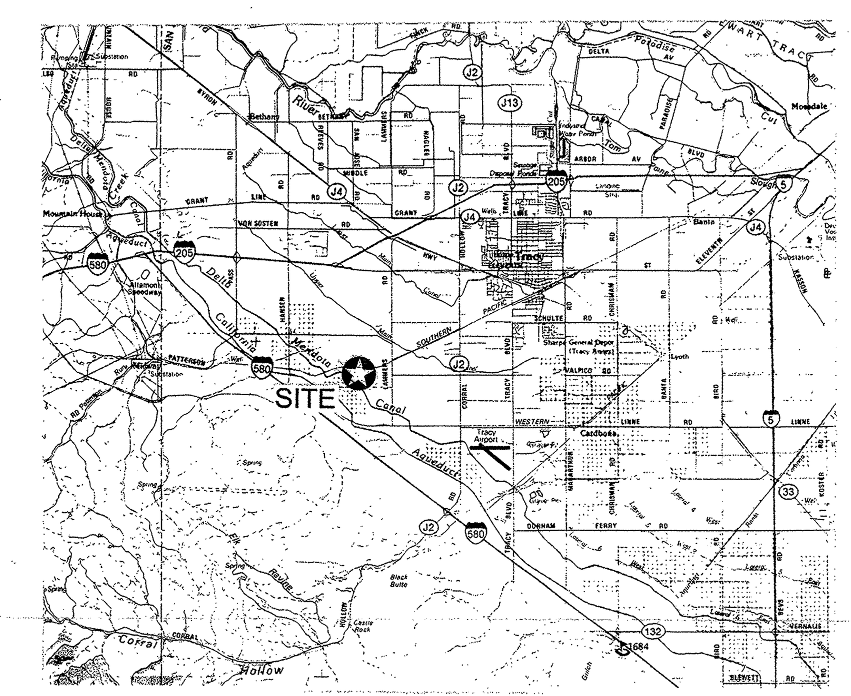
Retaining walls may be founded on shallow footings or drilled piers meeting the foundation criteria described above. Drainage material should be placed behind the wall. Fill used to backfill behind drains should be material that meets the criteria for select fill described above.

E. Surface Drainage

Proper surface drainage is important to minimize changes in soil moisture content. Ground surfaces in the vicinity of structures and flatwork, should slope away from the structures and flatwork; no ponding of surface water should be allowed especially adjacent to the structures. The site should be graded to drain towards swales and/or into a storm drain system. Roofs should be provided with gutters and downspouts that discharge into a closed storm drain system or on to paved areas.

PLATES





VICINITY MAP

Not to Scale

KEY

- Location of boring
- Location of CPT
- Location of percolation test hole

Reference: Topographic Plan by LCC, Inc., undated

REV	DESCRIPTION	DATE	APPROVED

REVISIONS

Hultgren-Tillis Engineers

2520 Stanwell Drive, Suite 100
Concord, California 94520

Phone (925) 685-6300
Fax (925) 685-6768

Power Plant
Tracy, California

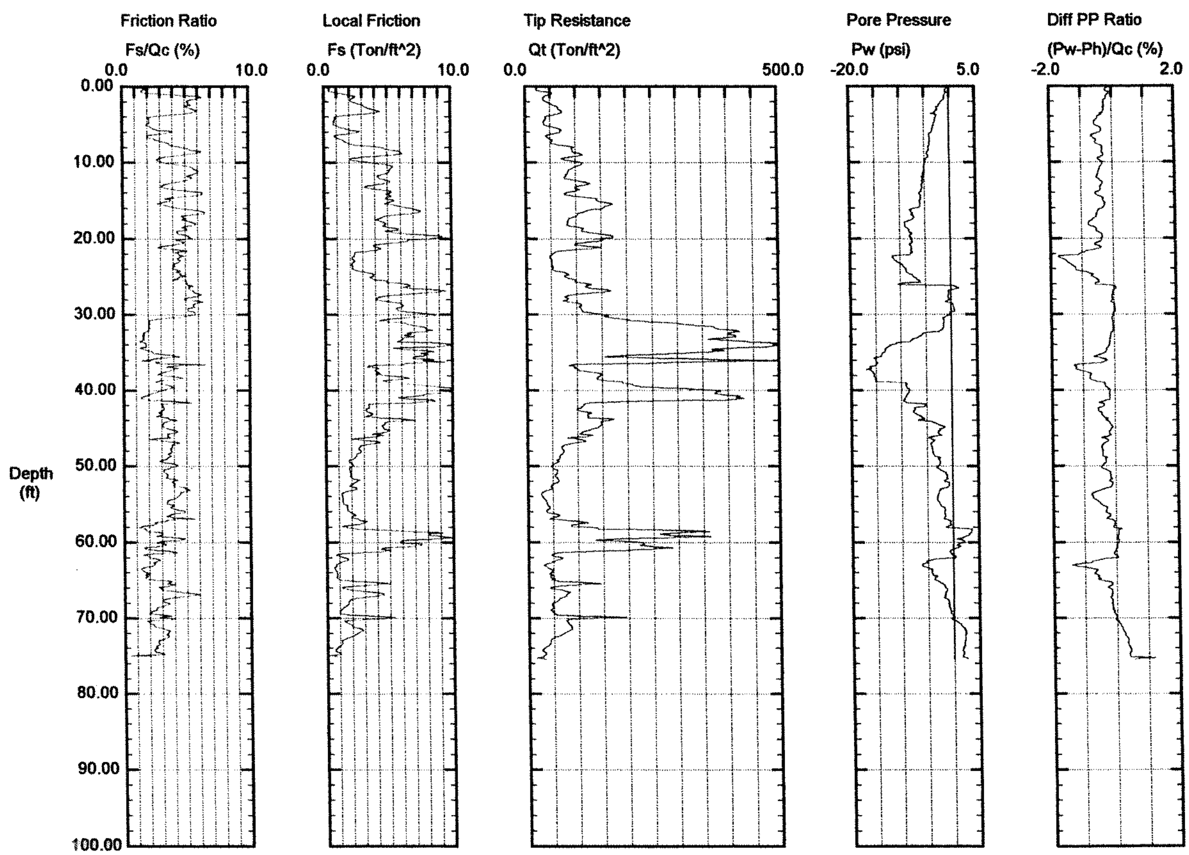
Site Plan

SCALE:	PROJECT NO.	DWG NO.	REV
1"=100'	474.01	1	
DWN BY: HDD	APP. BY: RKT	SHEET 1 OF 1	

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W158
Cone Used: HO738TC U2

CPT Date/Time: 06-11-01 08:50
Location: CPT-1
Job Number: 474.01



Maximum Depth = 75.46 feet

Depth Increment = 0.16 feet

Power Plant
Tracy, California

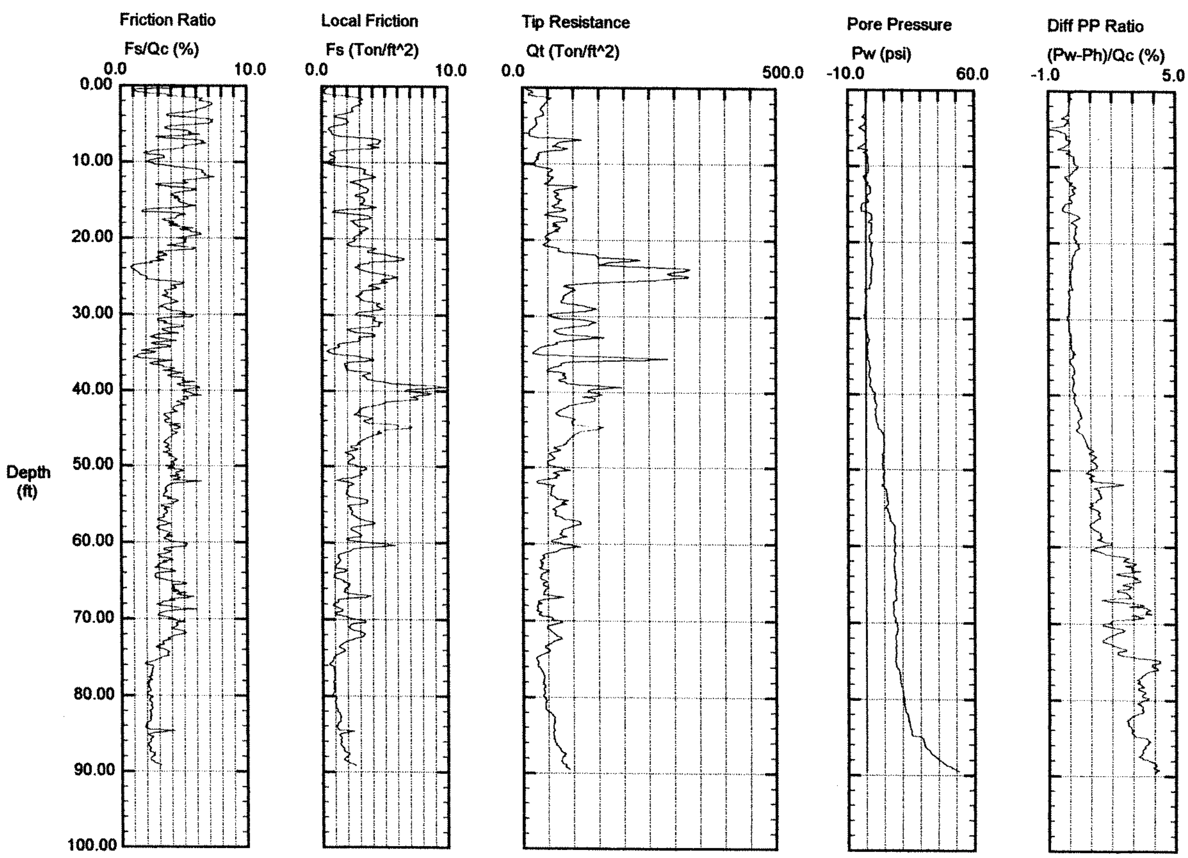
Log of CPT-1

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 2

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W161
Cone Used: HO738TC U2

CPT Date/Time: 06-11-01 13:40
Location: CPT-2
Job Number: 474.01



Maximum Depth = 89.57 feet

Depth Increment = 0.16 feet

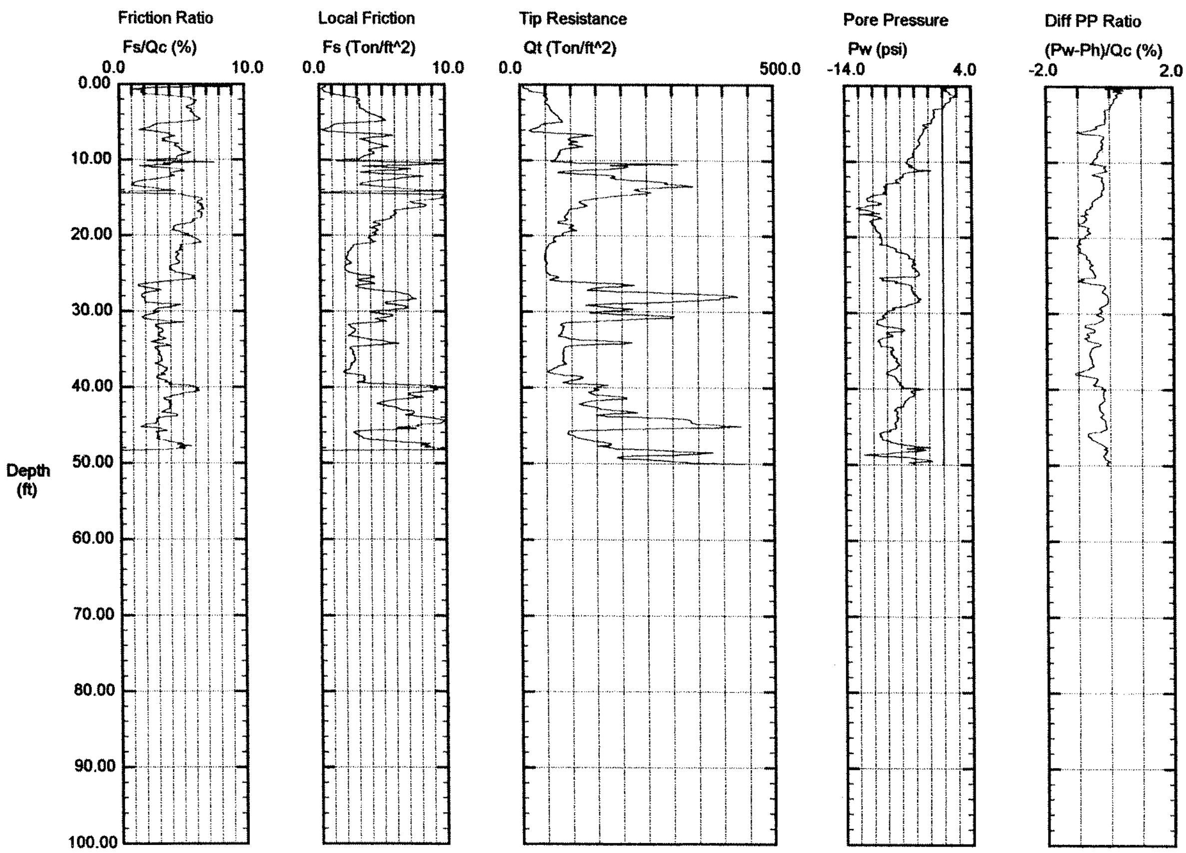
Power Plant *Tracy, California*	**Log of CPT-2**

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 3

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W162
Cone Used: HO738TC U2

CPT Date/Time: 06-12-01 08:12
Location: CPT-3
Job Number: 474.01



Maximum Depth = 50.03 feet

Depth Increment = 0.16 feet

Power Plant
Tracy, California

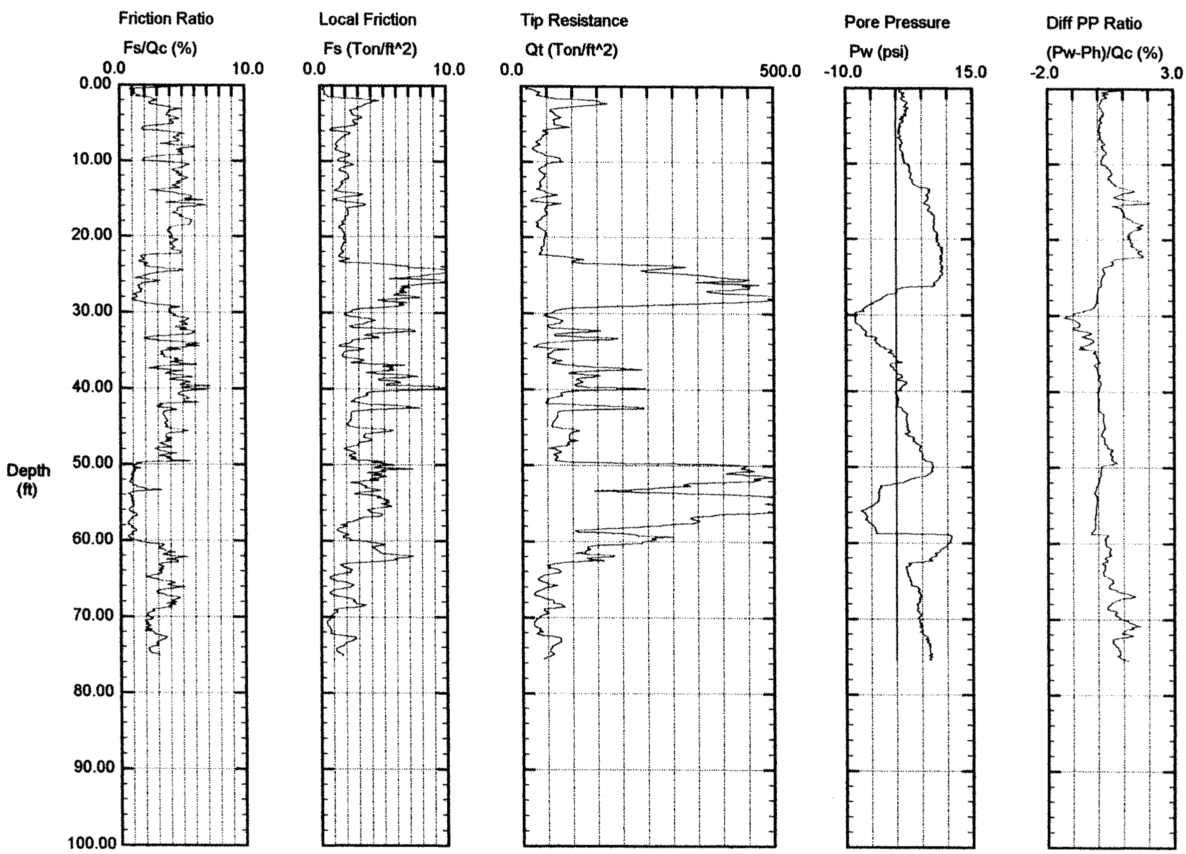
Log of CPT-3

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 4

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W164
Cone Used: HO738TC U2

CPT Date/Time: 06-12-01 10:25
Location: CPT-4
Job Number: 474.01



Maximum Depth = 75.46 feet

Depth Increment = 0.16 feet

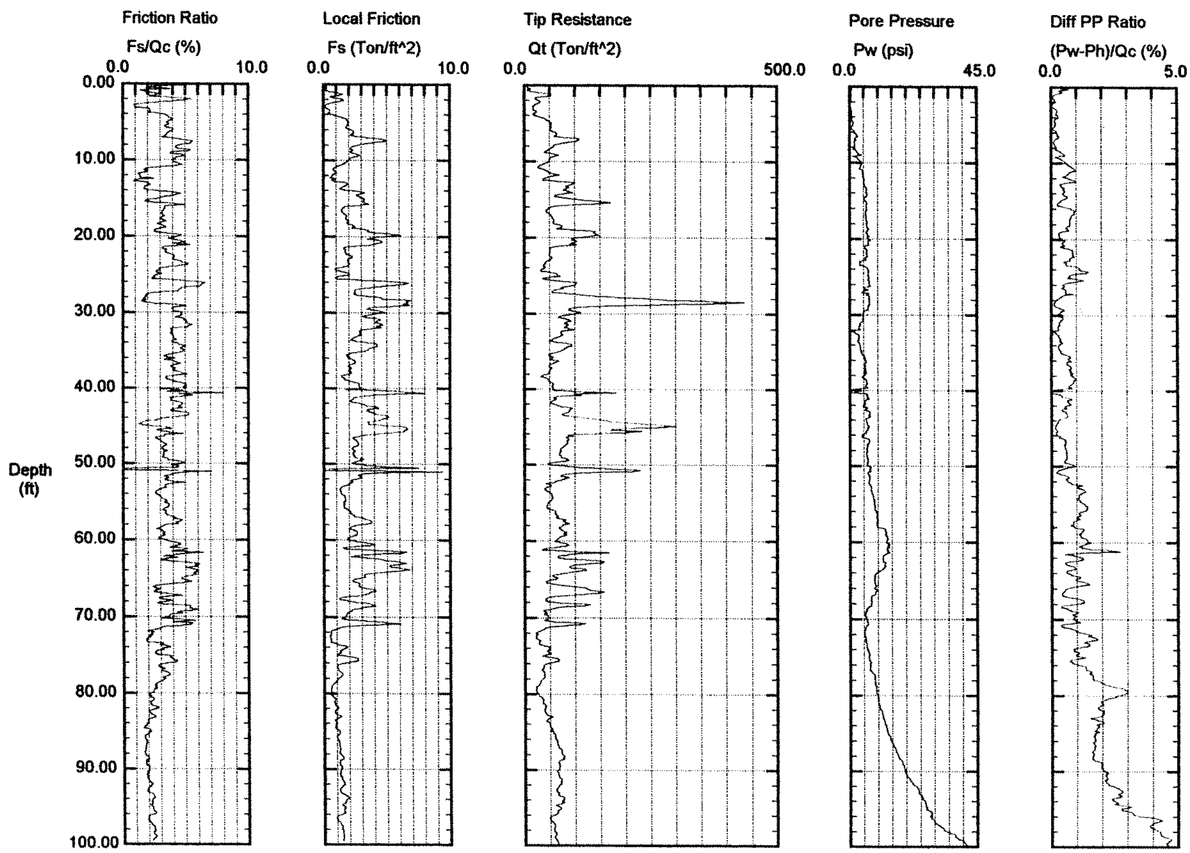
Power Plant *Tracy, California*	**Log of CPT-4**

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 5

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W165
Cone Used: HO738TC U2

CPT Date/Time: 06-12-01 12:39
Location: CPT-5
Job Number: 474.01



Maximum Depth = 100.23 feet

Depth Increment = 0.16 feet

Power Plant
Tracy, California

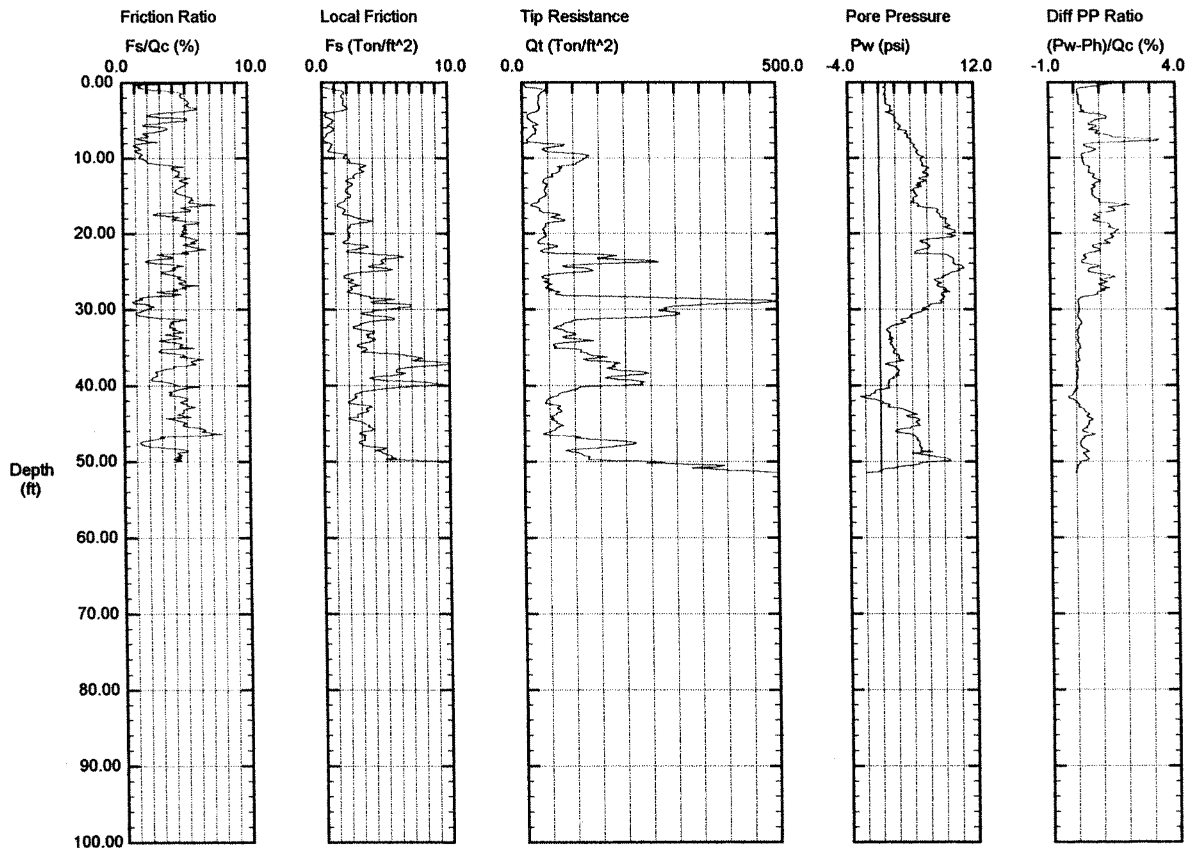
Log of CPT-5

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 6

VBI In-Situ Testing

Operator: TIM d'ARCY
Sounding: 01W166
Cone Used: HO738TC U2

CPT Date/Time: 06-12-01 15:02
Location: CPT-6
Job Number: 474.01



Maximum Depth = 51.51 feet

Depth Increment = 0.16 feet

Power Plant
Tracy, California

Log of CPT-6

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 7

Drilling Method: Rotary Wash
Date: June 12, 2001
Elevation: 180 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				0	CH	Gray Fat Clay (CH), stiff, dry to moist			
2.5		22	■				28 9	84 92	LL=61% PI=42%
				5	SP	Tan Poorly Graded Sand (SP), medium dense, dry			
					ML	Tan Sandy Silt (ML), very stiff, dry, with interbeds of Silty Sand (SM), with caliche			
		28	■				7	104	Sieve See Plate 21
				10	ML	Tan Silt (ML), dense, dry, with caliche			
3.5		41	■						
				15	ML	Tan Sandy Silt (ML) with clay, very stiff, moist			
					CL	Tan Lean Clay (CL) with silt, hard, moist			
4.0		35	■	20			30 18	91 107	TxUU See Plate 20
					ML	Light Brown Silt (ML), very stiff to hard, moist			
4.0		49	■	25		with Sandy Silt	20	108	
					SW	Light Brown Well-Graded Sand (SW), very dense, moist			
		58	■	30	ML	Light Brown Silt (ML), hard, moist			
					ML SM	Tan Sandy Silt (ML) and Tan Silty Sand (SM) with occasional fine gravel, very stiff, moist			
2.0 2.5		33	■	35			13	112	

Power Plant
Tracy, California

Log of Boring 1

Hultgren – Tillis Engineers	**Project No. 474.01**	**Plate No. 8**

Drilling Method: Rotary Wash
Date: June 12, 2001
Elevation: 180 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				35		ML / SM	Tan Sandy Silt (ML) and Tan Silty Sand (SM) with occasional fine gravel, very stiff to hard, moist			
>4.5		51		40						
						CL	Tan Lean Clay (CL) with sand, hard, moist			
>4.5		53		45				26	99	
						GP	Gravel (GP)			
						CL	Light Brown Lean Clay (CL) with sand, hard, moist			
>4.5		73/ 11"		50						
						SM	Brown Silty Sand (SM), very dense, saturated cemented into isolated fine gravel size particles			
		65		55						
						GP	Gravel (GP)			
						ML	Tan Silt (ML) with gravel, stiff, moist to wet, with abundant clay			
2.3		47		60				27	98	
3.0		81		65						
						CL	Olive-Gray and Rust-Brown Lean Clay (CL), stiff to very stiff, moist, with abundant silt			
3.7		47		70				30	92	

Power Plant
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Log of Boring 1

Hultgren - Tillis Engineers	**Project No. 474.01**	**Plate No. 9**

Drilling Method: Rotary Wash
Date: June 12, 2001
Elevation: 180 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				70		CL	Olive-Gray and Rust-Brown Lean Clay (CL), stiff to very stiff, moist, with abundant silt			
3.2		63		75						
							Bottom of boring at 75.5 feet			
							Groundwater level obscured by rotary drilling			
				80						
				85						
				90						
				95						
				100						
				105						

Power Plant
Tracy, California

Log of Boring 1

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 10

Drilling Method: Rotary Wash
Date: June 12, 2001
Elevation: 181 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Depth (feet)	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
			0	CL	Light Brown Lean Clay (CL) with gravel, very stiff, moist			
2.7		58						Sieve See Plate 21
				SM	Tan Silty Sand (SM) with gravel, very dense, dry			
					with caliche			
			5					
				SP	Brown Poorly Graded Sand (SP) with fine gravel, dense, wet			
>4.5		51				16	113	
				CL	Light Brown Lean Clay (CL), hard, moist			
				SM	Brown Silty Sand (SM) with gravel, dense, moist, with Sandy Silt			
	Pitcher Barrel		10					
	Pitcher Barrel		15	ML	Tan Sandy Silt (ML) with gravel, very stiff, moist			
				ML	Light Brown Sandy Silt (ML) with gravel, hard, moist			
		75	20			16	115	
					interbedded with Silty Sand			
>4.5		45	25			19	108	TxUU See Plate 20
>4.5		64	30			16	104	
				ML	Tan Sandy Silt (ML) with gravel, hard, moist, abundant sand, with caliche			
4.5		54	35					

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Log of Boring 2

Hultgren - Tillis Engineers | **Project No. 474.01** | **Plate No. 11**

Drilling Method: Rotary Wash
Date: June 12, 2001
Elevation: 181 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Depth (feet)	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
			35	ML	Tan Sandy Silt (ML) with gravel, hard, moist, abundant sand, with caliche			
				ML	Tan Silt (ML) with sand, stiff to very stiff, wet			
		67	40			26	95	
				ML	Tan Sandy Silt (ML) with abundant clay, hard, moist			
>4.5		66	45					
				SM	Brown Silty Sand (SM) with gravel, very dense, saturated			
		41/ 6"	50					
		41/ 5"	55					
			60		Bottom of boring at 59 feet			
			65					
			70					

Bottom of boring at 59 feet

Gravel caved into hole below 55 feet, driller unable to remove gravel

Groundwater level obscured by rotary drilling

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Log of Boring 2

Hultgren - Tillis Engineers | **Project No. 474.01** | **Plate No. 12**

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 162 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				0		CH	Brown Fat Clay (CH) with gravel, hard, dry			
>4.5		34						16	92	
				5						
						ML	Tan Silt (ML) with clay, very stiff, moist			
3.2 Pitcher Barrel								18	99	TxUU See Plate 20
				10						
						ML/SM	Tan Sandy Silt (ML) to Silty Sand (SM), very stiff, moist			
2.5 3.0		25						17	101	
							fine-grained sand			
				15						
						CL	Tan Lean Clay (CL) with abundant silt, very stiff, moist, occasional gravel			
3.0 Pitcher Barrel								15	106	TxUU See Plate 20
				20						
						ML/SM	Tan Sandy Silt (ML) to Silty Sand (SM) with gravel, very stiff, moist			
3.5		37								Sieve See Plate 21
				25						
						ML	Tan Sandy Silt (ML), very stiff to hard, moist			
4.5 Pitcher Barrel										
				30						
>4.5		51					becoming hard	16	116	
				35						

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Log of Boring 3

Hultgren – Tillis Engineers	**Project No. 474.01**	**Plate No. 13**

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 162 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				35		ML	Tan Sandy Silt (ML), hard, moist			
						ML	Tan Sandy Silt (ML) with gravel, hard, moist			
>4.5		75								
				40						
						SM	Silty Sand (SM) with clay, very dense, moist, with gravel			
2.5		41/6"						11	104	
				45						
						CL	Tan Sandy Lean Clay (CL) with abundant silt, hard, moist			
>4.5		47								
				50						
4.0		41/6"					with caliche	20	106	
				55						
						ML/SM	Tan Sandy Silt (ML) to Silty Sand (SM) with abundant sand, dense, saturated			
>4.5		70						21	106	
						CL	Tan Sandy Lean Clay (CL) with abundant silt, hard, moist			
				60						
						ML	Tan Sandy Silt (ML), very stiff, moist to wet, non-cohesive			
		67								
				65						
		62								
						CL	Tan Lean Clay (CL) with silt, very stiff, moist			
				70						

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Log of Boring 3

Hultgren – Tillis Engineers | **Project No. 474.01** | **Plate No. 14**

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 162 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				70		CL	Tan Lean Clay (CL) with silt, very stiff, moist			
		51						20	110	
							Bottom of boring at 72.5 feet			
							Groundwater obscured by rotary drilling			
				75						
				80						
				85						
				90						
				95						
				100						
				105						

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Log of Boring 3

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 15

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 163 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Depth (feet)	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
			0	CH	Brown Fat Clay (CH) with caliche, hard, moist			
>4.5		46				14	97	LL=50% PI=30%
			5	CL	Tan Sandy Lean Clay (CL), hard, dry			
>4.5		45				12	92	LL=30% PI=12%
			10	CL	Tan Sandy Lean Clay (CL) with gravel, very stiff to hard, dry,			
3.0 >4.5		69				13	95	
			15	ML	Tan Sandy Silt (ML) with abundant sand, stiff to hard, dry, with occasional gravel			
>4.5		63						
			20	ML	Tan Sandy Silt (ML) with caliche and gravel, very stiff, moist			
2.7		47				17	99	
			25	SM CL	Tan Silty Sand (SM) with gravel, dense, moist, and Tan Lean Sandy Clay (CL) with silt, hard, moist			
>4.5		51						
			30	SM	Tan Silty Sand (SM) with abundant silt, dense, dry			
3.5		34				15	101	
			35					

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Log of Boring 4

Hultgren - Tillis Engineers | **Project No. 474.01** | **Plate No. 16**

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 163 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
				35		SM				
						ML SM	Sandy Silt (ML), hard, moist and Silty Sand (SM) with gravel, very dense, moist			
4.0		54								
				40						
						CL	Tan Lean Clay (CL) with abundant silt, very stiff to hard, moist, with gravel			
4.3		28						12	91	
				45						
						SM	Gray and Brown Silty Sand (SM) with gravel, very dense, dry			Sieve See Plate 22
		41/ 6"								
							gravel to 1-inch size			
				50		CL	Light Brown Sandy Clay (CL) with abundant silt, hard, moist, with gravel and caliche, gravel to 2-inches in diameter			
		63/ 11"						16	104	
				55						
						GC	Brown and Gray Clayey Gravel (GC) with sand, very dense, saturated			
		41/ 5"								
				60		ML	Brown Sandy Silt (ML) with gravel, stiff, moist			
1.7		41/ 6"								
				65						
		44								
						ML				
				70						

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Log of Boring 4

Hultgren - Tillis Engineers	**Project No. 474.01**	**Plate No. 17**

Drilling Method: Rotary Wash
Date: June 13, 2001
Elevation: 163 feet

Pocket Penetrometer (TSF)	Drilling Rate (min/ft)	N (blows/ft.)	Samples	Depth (feet)	Graphic Log	USCS	Material Description	Moisture Content (%)	Dry Density (pcf)	Other Laboratory Tests
1.0 1.5		29		70		ML	Tan Silt (ML) with clay, stiff, moist	25	103	0.65 TSF Torvane
				75						
2.7		41/ 5"				CL	Gray and Yellow-Brown Lean Clay (CL) with abundant silt, very stiff, moist	29	92	
				80			Bottom of boring at 79.5 feet Groundwater obscured by rotary drilling			
				85						
				90						
				95						
				100						
				105						

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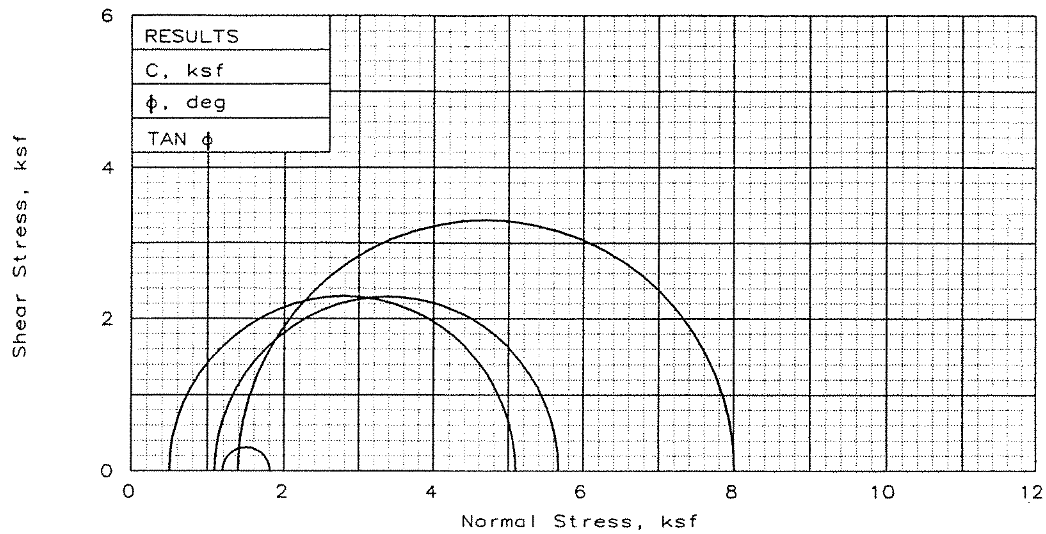
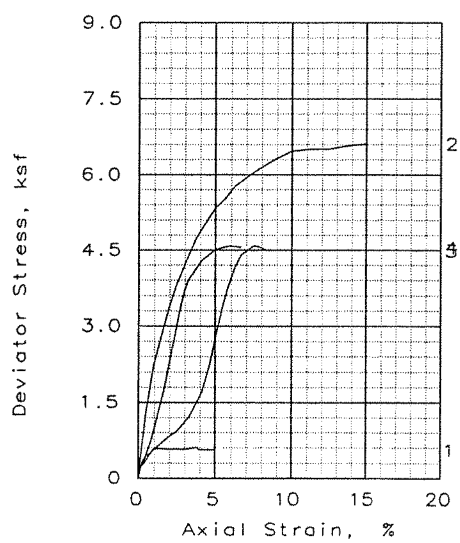
Log of Boring 4

Hultgren – Tillis Engineers	**Project No. 474.01**	**Plate No. 18**





SAMPLE NO.:		1	2	3	4
INITIAL	WATER CONTENT, %	30.2	19.3	17.7	15.3
	DRY DENSITY, pcf	90.6	107.5	98.6	105.8
	SATURATION, %	94.7	91.5	67.3	69.6
	VOID RATIO	0.860	0.568	0.710	0.593
	DIAMETER, in	1.93	1.93	2.87	2.88
	HEIGHT, in	4.00	4.00	6.00	6.00
AT TEST	WATER CONTENT, %	30.2	19.3	17.7	15.3
	DRY DENSITY, pcf	90.6	107.5	98.6	105.8
	SATURATION, %	94.7	91.5	67.3	69.6
	VOID RATIO	0.860	0.568	0.710	0.593
	DIAMETER, in	1.93	1.93	2.87	2.88
	HEIGHT, in	4.00	4.00	6.00	6.00
Strain rate, %/min		1.00	1.00	1.00	1.00
BACK PRESSURE, ksf		0.00	0.00	0.00	0.00
CELL PRESSURE, ksf		1.20	1.40	0.50	1.09
DEVIATOR STRESS, ksf		0.62	6.61	4.59	4.58
STRAIN, %		3.8	15.0	7.5	5.8
ULT. STRESS, ksf					
STRAIN, %					
σ_1 FAILURE, ksf		1.81	8.01	5.10	5.67
σ_3 FAILURE, ksf		1.20	1.40	0.50	1.09

TYPE OF TEST:
Unconsolidated Undrained
SAMPLE TYPE: Undisturbed
DESCRIPTION: See Remarks

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: 1)brn. sandy CLAY 2)brn. clayey SAND 3)brown tan sandy CLAY 4)brn. sandy CLAY w/ pockets of clayey sa.

Fig. No.: ______

CLIENT: Hultgren-Tillis

PROJECT: 474.01

SAMPLE LOCATION: 1)B1 @ 19-19.5' 2)B2 @ 24-24.5' 3)B3 @ 7-9' 4)B3 @ 17-19

PROJ. NO.: 212-039 DATE: 06/18/01

TRIAXIAL SHEAR TEST REPORT

COOPER TESTING LABORATORY

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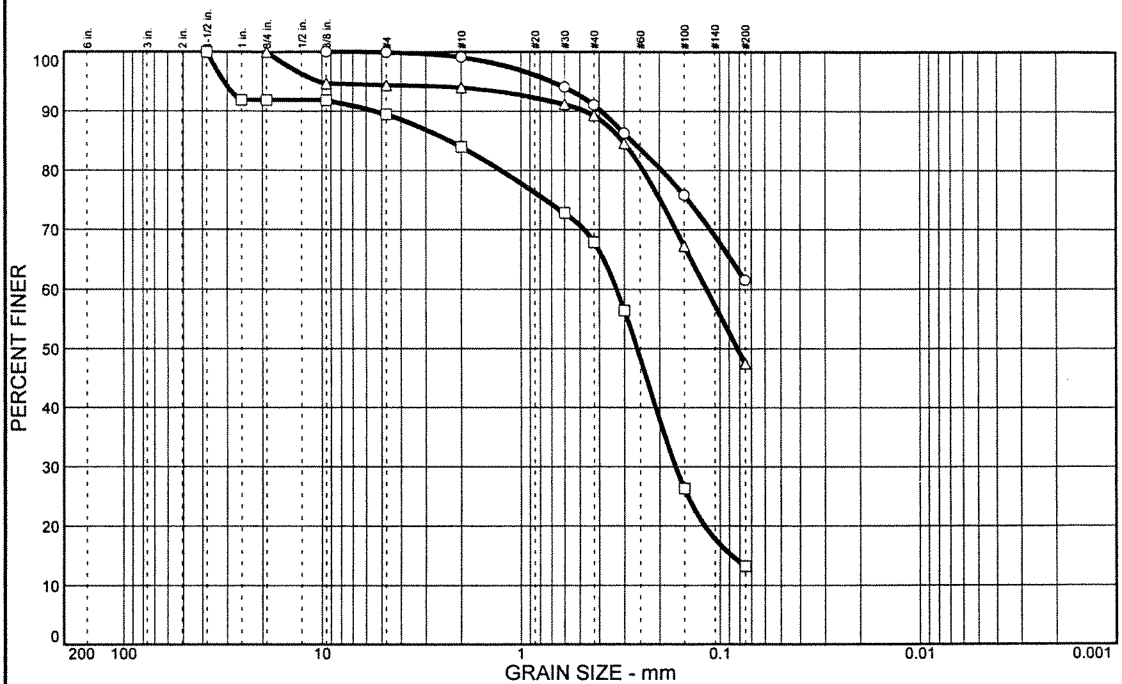
UU Triaxial Test Results

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 19

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○		0.1	38.4			61.5	ML			
□		10.5	76.3			13.2	SM			
△		5.6	46.9			47.5	SM			

SIEVE inches size	PERCENT FINER ○	□	△
1.5		100.0	
1		91.9	
3/4		91.9	100.0
3/8	100.0	91.9	94.7
	GRAIN SIZE		
D_{60}		0.328	0.116
D_{30}		0.166	
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIZE	PERCENT FINER ○	□	△
#4	99.9	89.5	94.4
#10	99.1	84.0	94.0
#30	94.1	72.8	91.2
#40	91.1	67.9	89.3
#50	86.3	56.4	84.6
#100	75.8	26.3	67.3
#200	61.5	13.2	47.5

SOIL DESCRIPTION

○ brown sandy CLAY

□ brown clayey SAND

△ brown clayey SAND

REMARKS:

○

□

△

○ Source: B1 — Elev./Depth: 8.5-9.0

□ Source: B2 — Elev./Depth: 3-3.5

△ Source: B3 — Elev./Depth: 23.4-24

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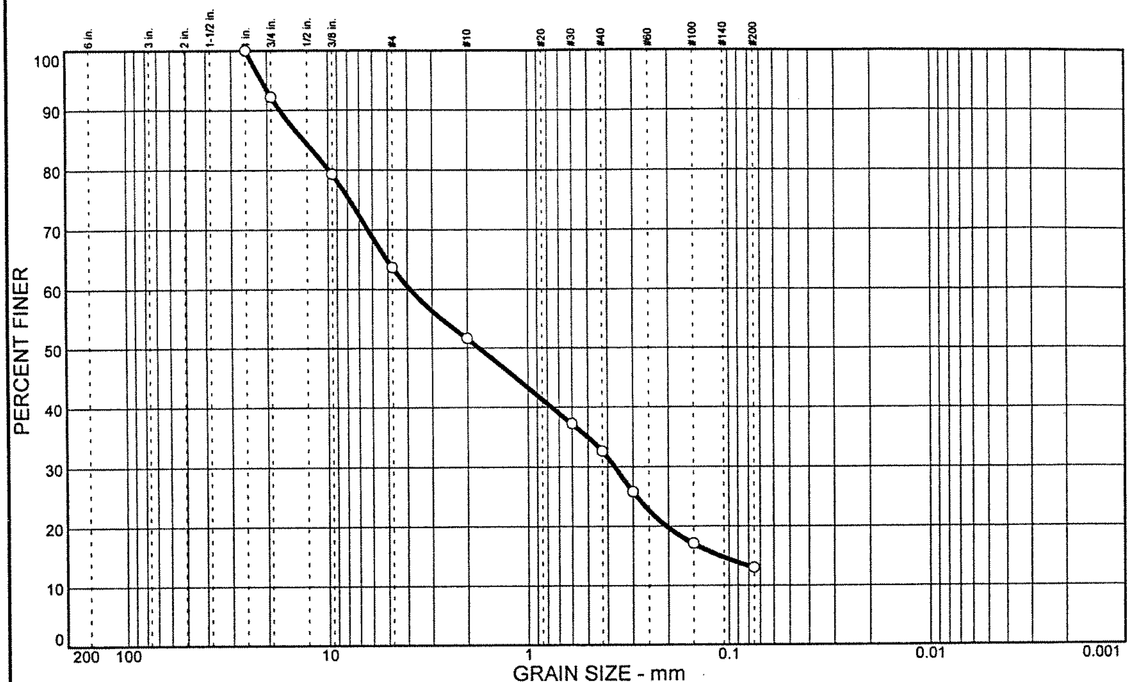
Sieve Analysis Results

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 20

PARTICLE SIZE DISTRIBUTION TEST REPORT



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	% FINES	USCS	AASHTO	PL	LL
○		36.3	50.7			13.0	SM			

SIEVE inches size	PERCENT FINER ○		
1	100.0		
3/4	92.2		
3/8	79.3		
	GRAIN SIZE		
D_{60}	3.86		
D_{30}	0.370		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

○ Source: B4

SIZE	PERCENT FINER ○		
#4	63.7		
#10	51.6		
#30	37.2		
#40	32.6		
#50	25.8		
#100	17.1		
#200	13.0		

Sample No.: 47-47.5

SOIL DESCRIPTION

○ brown silty SAND with gravel

REMARKS:

○

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Sieve Analysis Results

Hultgren - Tillis Engineers	Project No. 474.01	Plate No. 21