DATE DRILLED 12/15/94 BORING NO. B-10
GROUND ELEVATION 488' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs DROP 30"
SAMPLED BY HV LOGGED BY HV REVIEWED BY DR/RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						■		**ASPHALTIC CONCRETE: Approximately 8 inches thick.**
						▦	SM	BASE: Yellowish brown, damp to moist, medium dense to dense, silty fine to coarse SAND with gravel.
			12	11.1	122.0	▨	CL	FILL: Dark reddish brown, wet, stiff, fine sandy CLAY with silt.
						▦	SM	Light yellowish brown, moist, loose, silty fine to medium SAND; scattered fine gravel and coarse sand.
5			131/9"	17.7	105.8	▨	CL/CH	TOPSOIL: Light brown to yellowish brown, moist, hard, silty CLAY; trace fine sand.
						░		LINDAVISTA FORMATION: Brown to grayish brown, moist, strongly to moderately cemented, silty fine-grained SANDSTONE with clay.
10			136/9"	15.0	102.4	░		Scattered interbeds of silty claystone.
						░		Abundant gravel.
15			131	15.1	117.8	■		Light brown with olive-gray mottles, wet, moderately indurated, clayey SILTSTONE with fine sand.
								Total Depth = 16.5 Feet. Groundwater not encountered. Backfilled and pavement patched on 12/15/94.
20								

Ninyo & Moore

BORING LOG

Miramar Road Subsystem Extension
San Diego, California

PROJECT NO.	DATE	FIGURE
102746-01	2/95	A-10

DATE DRILLED 12/15/94 BORING NO. B-11
GROUND ELEVATION 458' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 8" Diameter Hollow-Stem Auger
DRIVE WEIGHT 140 lbs DROP 30"
SAMPLED BY HV LOGGED BY HV REVIEWED BY DR/RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	<u>ASPHALTIC CONCRETE</u>: Approximately 2-1/2 inches thick.
								<u>FILL</u>: Yellowish brown, damp, loose to medium dense, silty fine to medium SAND.
							SM	<u>ASPHALTIC CONCRETE</u>: Approximately 6-1/2 inches thick.
			10	12.0	116.9		SM	<u>BASE</u>: Yellowish brown, moist, medium dense to dense, silty fine to coarse SAND with gravel.
							CL/CH	<u>FILL</u>: Dark reddish brown, moist, loose, silty fine SAND with clay.
								<u>TOPSOIL</u>: Dark reddish brown, moist, stiff, silty CLAY.
5								<u>LINDAVISTA FORMATION</u>: Brown to grayish brown, damp to moist, moderately cemented, clayey fine-grained SANDSTONE with silt.
			171	10.1	111.7			
								Yellowish brown.
10								
			51	14.0	113.2			Fine- to medium-grained.
								Olive-gray.
15			134/ 11"	7.2	—			
								Red, damp, moderately cemented, sandy CONGLOMERATE; matrix comprised of silty fine- to coarse-grained sandstone.
								Total Depth = 15.9 Feet. Groundwater not encountered. Backfilled and patched on 12/15/94.
20								

Ninyo & Moore

BORING LOG

Miramar Road Subsystem Extension
San Diego, California

PROJECT NO.	DATE	FIGURE
102746-01	2/95	A-11

DATE DRILLED 12/15/94 BORING NO. B-12

GROUND ELEVATION 446' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Diameter Hollow-Stem Auger

DRIVE WEIGHT 140 lbs DROP 30"

SAMPLED BY HV LOGGED BY HV REVIEWED BY DR/RI

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								**ASPHALTIC CONCRETE:** Approximately 10 inches thick.
							SP-SM	**BASE:** Brown, damp, dense, fine to coarse SAND with fine gravel and silt.
			134/9"	12.2	115.6		SC	**FILL:** Dark reddish brown, moist, medium dense, clayey fine to coarse SAND; scattered gravel.
								LINDAVISTA FORMATION: Reddish brown, damp to moist, moderately cemented, clayey fine- to coarse-grained SANDSTONE; intensely weathered; scattered manganese stains.
5			63/4"	14.9	---			
								Scattered layers of sandy claystone; scattered fine gravel.
			42/2"					Abundant gravel.
10								
								Yellowish brown; moderately weathered.
15			186/ 11"	11.0	—			**STADIUM CONGLOMERATE:** Olive, damp, moderately cemented, silty CONGLOMERATE.
								Total Depth = 15.9 Feet. Groundwater not encountered. Backfilled and pavement patched on 12/15/94.
20								

Ninyo & Moore

BORING LOG

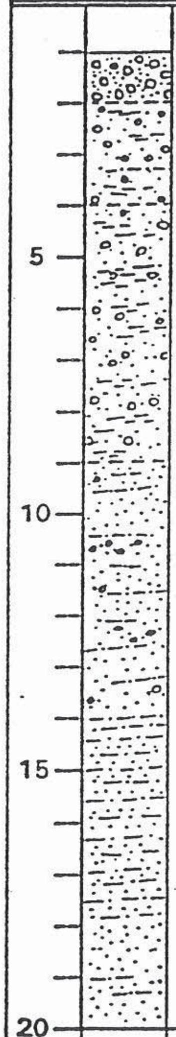
Miramar Road Subsystem Extension
San Diego, California

PROJECT NO.	DATE	FIGURE
102746-01	2/95	A-12

LOGS OF EXCAVATIONS
FROM
SOUTHLAND GEOTECHNICAL
1993

EXPLANATION OF GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II	F&C Drilling Company, Inc.	Boring No. 1
Project No. 39C61	Mobile Drill Rig B-61, Hollow Stem Auger	Sheet 1 of 1
November 8, 1993	Boring Diameter: 8"	Logged by SET
Top of Hole Elevation: 388.95'	Sample Driver: 140 lbs, 30" drop	Sampled by SET



Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	
						SM	
						CL-SC	
5		1	20	103.6	23.4		Relatively undisturbed drive sample (Modified California Sampler) (number to left indicates sample number)
		①					
						SM	
10		2	60/6"	105.3	8.7		
		②					Bulk sample (circled)
							GRAPHIC LOG sand silt clay clast
15		3	96	110.0	14.8	ML-SM	
						SM.	
		③					
20							Total Depth = 20' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-08-93
25							

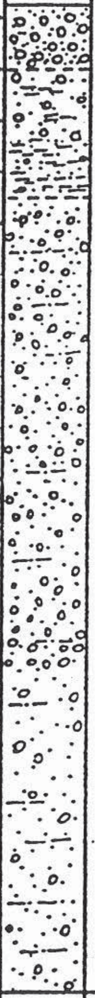
SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 8, 1993
Top of Hole Elevation: 395.38'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 2
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	**ASPHALTIC CONCRETE** **AGGREGATE BASE** @9" Aggregate base
						SM CL-SC	FILL @2' Red-brown, damp, medium dense, silty fine to medium sand; slightly clayey, with gravels @3.5' Brown to red-brown, damp, stiff to medium dense, sandy clay to clayey sand; with gravels, appears to be derived from residual soil
5		①				GM	LINDAVISTA FORMATION @4.5' Light brown, dry to slightly damp, very dense, silty fine sandstone with gravels; cemented
10							@10' Well-cemented zone
							@13' More gravels
						SM	@13.5' Light orange-brown, dry to slightly damp, very dense to hard, silty fine to medium sandstone; with gravels
15		②					
20							Total Depth = 20' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-08-93
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 8, 1993
Top of Hole Elevation: 399.69'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 3
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							ASPHALTIC CONCRETE
						GP	**AGGREGATE BASE** @15.5" Aggregate base
						CL-CH	**FILL** @2.5' Gray-brown, damp to moist, stiff, sandy clay
						GM-SM	**LINDAVISTA FORMATION** @3.5' Light brown, dry to slightly damp, very dense, silty fine sandstone with gravels; cemented
5							@5' Turns reddish brown
		①					
							@8-11' Very difficult drilling
10							
15		②					
20							Total Depth = 20' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-08-93
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 15, 1993
Top of Hole Elevation: ±405'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 4
Sheet 1 of 1
Logged by SET
Sampled by SET

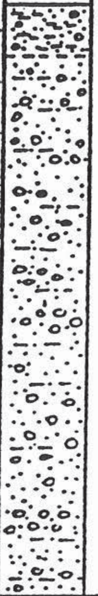
Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							ASPHALTIC CONCRETE; CONCRETE @10" (no aggregate base)
	░					CL-SC	RESIDUAL SOIL @1' Brown, damp, stiff to medium dense, sandy clay to clayey sand; with gravels
	░					GM	LINDAVISTA FORMATION@2.5' Light brown, dry, hard, silty fine sandstone with gravels, well-cemented, difficult drilling
5							Total Depth = 3.5' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-15-93 Refusal on very hard, cemented conglomerate
10							
15							
20							
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 15, 1993
Top of Hole Elevation: 405.35'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 5
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							ASPHALTIC CONCRETE
						CL-SC	RESIDUAL SOIL @6" Brown, damp, stiff to medium dense, sandy clay to clayey sand; with gravels
						GM	LINDAVISTA FORMATION @1.5' Light brown, dry, hard, silty fine sandstone with gravels, well-cemented, difficult drilling
5							
							Very difficult drilling
10							
							Total Depth = 12' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-15-93 Refusal on very hard, cemented conglomerate
15							
20							
25							

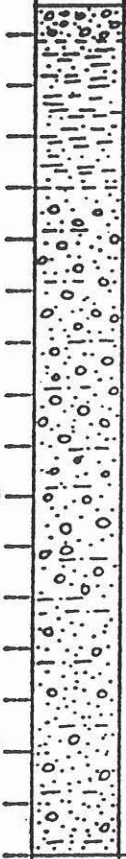
SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 9, 1993
Top of Hole Elevation: 434.38'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 6
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	**ASPHALTIC CONCRETE; AGGREGATE BASE** @6"
						CL-CH	**RESIDUAL SOIL** @14" Brown to orange-brown, damp, stiff, sandy, silty clay
						GM	**LINDAVISTA FORMATION** @4' Light brown, dry, very dense to hard, slightly silty fine to medium sandstone with gravels; cemented zones
5							
		1	105/6"		8.3		
10		①					
						SM	@12.5' Brown, damp to slightly moist, very dense, silty, clayey, fine to medium sandstone; with occasional gravels and cemented zones
15							
						GM	@17' More gravels, well cemented, difficult drilling
							Total Depth = 17' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-09-93 Refusal on very hard, cemented conglomerate *disturbed sample
20							
25							

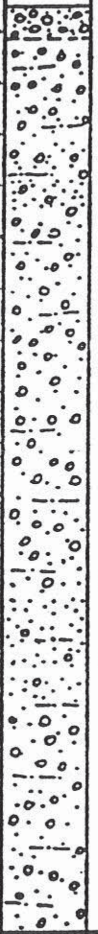
SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 9, 1993
Top of Hole Elevation: 440.20'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 7
Sheet 1 of 1
Logged by SET
Sampled by SET

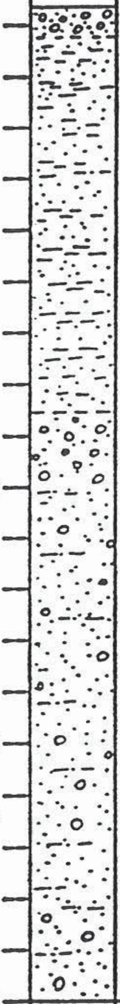
Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	<u>ASPHALTIC CONCRETE</u>; <u>AGGREGATE BASE</u> @6"
							<u>LINDAVISTA FORMATION</u>
						GM	@14" Light brown, dry, medium dense to dense, silty fine to medium sandstone with gravels
		①				SM-GM	@3.5-4.5' Dark red-brown, dry, silty fine sandstone with gravels
5						GM	@4.5' Cemented zone of light brown conglomerate, difficult drilling
10							
		②				GM	@11' Red-brown, dry, very dense, silty fine to coarse sandstone with gravels
		③				SM	@12' Red-brown, dry, very dense, silty fine to medium sandstone; with occasional gravel
						GM	@14' Light brown, dry, very dense to hard, silty fine sandstone with gravels; well cemented
15							
						GM-	@17' Well cemented conglomerate
							Total Depth = 18.5' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-09-93 Refusal on very hard, cemented conglomerate
20							
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 9, 1993
Top of Hole Elevation: 443.20'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 8
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.		Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							GP	ASPHALTIC CONCRETE; AGGREGATE BASE @7"
								RESIDUAL SOIL
							SC-CL	@15" Brown to orange-brown, damp, medium dense to dense, clayey fine to medium sand to sandy clay; less clayey, more dense with depth, gradational to:
5		1	■	50	110.2	16.3		
		①						
							CL-CH	@7' Brown, damp, very stiff, sandy clay horizon
								LINDAVISTA FORMATION
							GM	@8.5' Light brown, dry, very dense, silty fine sandstone with gravels
10							SM	@10' Light brown, dry, very dense to hard, silty fine sandstone; less clasts, well cemented
		②						Well cemented
15								Well cemented
							SM	@18' Red-brown, dry, very dense, silty fine to coarse sandstone; with occasional gravel, less well cemented, more coarse sandstone
		③						@20' Very hard, well cemented
20								Total Depth = 20' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-09-93
25								

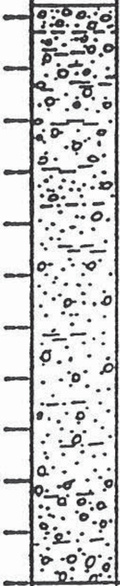
SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
'roject No. 39C61
November 10, 1993
Top of Hole Elevation: ±437'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 9
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	**ASPHALTIC CONCRETE; AGGREGATE BASE @8"**
						GM	**LINDAVISTA FORMATION** @16" Dark brown, damp, dense to very dense, slightly clayey fine to medium sandstone with gravels
5		①				SM	@4' Orange-brown, dry to slightly damp, very dense, slightly clayey, silty fine to medium sandstone; with occasional gravels
10						GM	@10' Brown, dry, very dense to hard, silty fine sandstone with gravels; becoming more gravelly, with larger clasts
15							Total Depth = 12' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-10-93 Refusal on large clasts
20							
25							

SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II	F&C Drilling Company, Inc.	Boring No. 10
Project No. 39C61	Mobile Drill Rig B-61, Hollow Stem Auger	Sheet 1 of 1
November 10, 1993	Boring Diameter: 8"	Logged by SET
Top of Hole Elevation: ±437'	Sample Driver: 140 lbs, 30" drop	Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	ASPHALTIC CONCRETE; AGGREGATE BASE @9" AGGREGATE BASE/LINDAVISTA FORMATION MIX @11"
						GM	LINDAVISTA FORMATION @17" Dark red-brown, damp, very dense to hard, slightly clayey fine to medium sandstone with gravels
5							Total Depth = 4.5' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-10-93 Refusal on large clasts
10							
15							
20							
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 10, 1993
Top of Hole Elevation: ±437'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 11
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
						GP	**ASPHALTIC CONCRETE; AGGREGATE BASE @9"**
	▨					GM	**LINDAVISTA FORMATION** @15" Dark red-brown, damp, very dense to hard, slightly clayey fine to medium sandstone with gravels
5							Total Depth = 5' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-10-93 Refusal on large clasts
10							
15							
20							
25							

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 10, 1993
Top of Hole Elevation: 430.95'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 12
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							ASPHALTIC CONCRETE (no aggregate base)
						GM	**FILL** @1' Brown, damp to moist, medium dense, slightly clayey medium sand with gravels
5		①				GC	@5' Becomes more clayey, lighter in color
10		②					
						GC-GM	@12' Dark gray-brown, damp to slightly moist, dense, clayey fine sand with gravels; less clasts than above
						GM	@13' Less clayey fill
15		③					
						SM	**LINDAVISTA FORMATION** @20' Light orange-brown, dry to slightly damp, very dense to hard, silty fine sandstone; well cemented
20							Total Depth = 20' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-10-93 Refusal on well cemented Lindavista Formation
25							

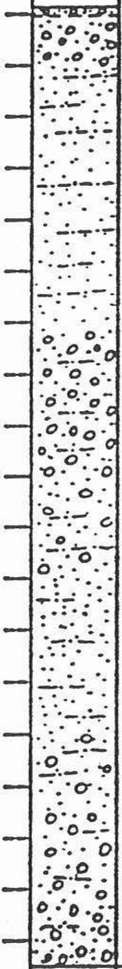
SGC

GEOTECHNICAL BORING LOG

Miramar Road Reclaimed Water Pkg II
Project No. 39C61
November 10, 1993
Top of Hole Elevation: 445.67'

F&C Drilling Company, Inc.
Mobile Drill Rig B-61, Hollow Stem Auger
Boring Diameter: 8"
Sample Driver: 140 lbs, 30" drop

Boring No. 13
Sheet 1 of 1
Logged by SET
Sampled by SET

Depth in Feet	Graphic Log	Sample No.	Blows Per Foot	Dry Density (pcf)	Water Content (%)	U.S.C.S. Soil Type	Geotechnical Description
							ASPHALTIC CONCRETE; AGGREGATE BASE @10"
							**LINDAVISTA FORMATION**
						GM	@1' Brown, dry to slightly damp, very dense, silty fine to medium sandstone with gravels
						SM	@2' Orange-brown, dry to slightly damp, very dense to hard, silty fine sandstone
5		①					
						GM	@7' Light brown, dry, very dense to hard, silty fine to medium sandstone with gravels
10							@10' Less gravels
		②				SM	@12' Orange-brown, dry, very dense to hard, silty fine sandstone
		③					
15						GM	@15' Brown, dry, very dense, silty fine to medium sandstone with gravels
							@19.5' Very difficult drilling, lots of clasts
20							Total Depth = 19.5' No ground water encountered at time of drilling Backfilled (and asphalt patched) on 11-10-93 Refusal on clasts within Lindavista Formation
25							

LOGS OF EXCAVATIONS
FROM
GEOBASE
1993 - 1994

TerraCosta
Consulting Group

The terms and symbols used on the Log of Borings to summarize the results of the field investigation and subsequent laboratory testing are described in the following:

It should be noted that materials, boundaries, and conditions have been established only at the boring locations, and are not necessarily representative of subsurface conditions elsewhere across the site.

A. PARTICLE SIZE DEFINITION (ASTM D2487 and D422)

Boulder	-- larger than 12-inches	Sand, medium	-- No. 40 to No. 10 sieves
Cobble	-- 3-inches to 12-inches	Sand, fine	-- No. 200 to No. 40 sieves
Gravel, coarse	-- 3/4-inch to 3-inches	Silt	-- 5µm to No. 200 sieves
Gravel, fine	-- No. 4 sieve to 3/4-inch	Clay	-- smaller than 5µm
Sand, coarse	-- No. 10 to No. 4 sieve		

B. SOIL CLASSIFICATION

Soils and bedrock are classified and described according to their engineering properties and behavioral characteristics. The soil of each stratum is described using ASTM D2487 and D2488.

The following adjectives may be employed to define percentage ranges by weight of minor components:

trace	-- 1-10%
little	-- 10-20%
some	-- 20-35%
"and" or "y"	-- 35-50%

The following descriptive terms may be used for stratified soils:

parting	-- 0 to 1/16-in. thickness;
seam	-- 1/16 to 1/2-in. thickness;
layer	-- 1/2-in. to 12-in. thickness;
stratum	-- greater than 12-in. thickness.

C. SOIL DENSITY AND CONSISTENCY

The density of coarse grained soils and the consistency of fine grained soils are described on the basis of the Standard Penetration Test:

COARSE GRAINED SOILS		FINE GRAINED SOILS		
Density	SPT Blows per foot	Estimated Consistency	SPT Blows per foot	Estimated Range of Unconfined Compressive Strength (tsf)
very loose	less than 4	very soft	less than 2	less than 0.25
loose	4 to 10	soft	2 to 4	0.25 to 0.50
medium	10 to 30	firm (medium)	4 to 8	0.50 to 1.0
dense	30 to 50	stiff	8 to 15	1.0 to 2.0
very dense	over 50	very stiff	15 to 30	2.0 to 4.0
		hard	over 30	over 4.0

GEOBASE, INC.

EXPLANATION OF TERMS
AND SYMBOLS

FIGURE B-2

D: STANDARD PENETRATION TEST (SPT) -- D1586

The SPT test involves failure of the soil around the tip of a split spoon sampler for a condition of constant energy transmittal. The split spoon, 2-inches outside diameter and 1 3/8-inches inside diameter, is driven eighteen (18) inches. The sampler is seated in the first six (6) inches and the number of blows required to drive the sampler the last foot is recorded as the "N" value or SPT blow count. The driving energy is provided by a 140 pound weight dropping thirty (30) inches.

E. SAMPLE TYPE

■ Thin walled tube
◫ SPT split spoon
☒ California modified sampler
▤ Disturbed
⧄ No recovery
▦ Core

F. ABBREVIATION OF LABORATORY TEST DESIGNATIONS

C	Consolidation	PP	Pocket Penetrometer
CBR	California Bearing Ratio	PS	Particle Size
Ch	Water Soluble Chlorides	RV	R-Value
DS	Direct Shear	SE	Sand Equivalent
EI	Expansion Index	SG	Specific Gravity
ER	Electrical Resistivity	SO_4	Water Soluble Sulfates
k	Permeability	TX	Triaxial Compression
MD	Moisture/Density Relationship	TV	Torvane Shear
O	Organic Content	U	Unconfined Compression
pH	pH		

G. STRATIFICATION LINES

The stratification lines indicated on the boring logs and profiles represent the approximate boundary between material types and the transition may be gradual.

GEOBASE, INC.

EXPLANATION OF TERMS
AND SYMBOLS

FIGURE B-2

SOIL CLASSIFICATION SYSTEM (ASTM D2487)

MAJOR DIVISION			GROUP SYMBOL	GRAPHIC SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
HIGHLY ORGANIC SOILS			Pt	▤	Peat and other highly organic soils	Strong color or odor and often fibrous texture	
COARSE-GRAINED SOILS (More than half by weight larger than No. 200 sieve size)	GRAVELS (More than half coarse fraction larger than No. 4 sieve size)	CLEAN GRAVELS	GW	⁘	Well-graded Gravels, Gravel-Sand mixtures (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 6$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP	▲	Poorly-graded Gravels and Gravel-Sand mixtures (<5% fines)	Not meeting all above requirements	
		DIRTY GRAVELS	GM	▲	Silty Gravels, Gravel-Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$	
			GC	▨	Clayey Gravels, Gravel-Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$	
	SANDS (More than half coarse fraction smaller than No. 4 sieve size)	CLEAN SANDS	SW	∘	Well-graded Sands, Gravelly Sands (<5% fines)	$C_u = \frac{D_{60}}{D_{10}} > 4$ $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		Poorly-graded Sands or Gravelly Sands (<5% fines)	Not meeting all above requirements	
		DIRTY SANDS	SM	▥	Silty Sands, Sand-Silt mixtures (>12% fines)	Atterberg limits below "A" line or $I_p < 4$	
			SC	▨	Clayey Sands, Sand-Clay mixtures (>12% fines)	Atterberg limits above "A" line or $I_p > 7$	
FINE-GRAINED SOILS (More than half by weight passes No. 200 sieve size)	SILTS Below "A" line on plasticity chart: negligible organic content		ML	▥	Inorganic Silts and very fine Sands, Rock Flour, Silty Sands of slight plasticity	$W_L < 50$	See chart below
			MH	▥	Inorganic Silts micaceous or diatomaceous, fine Sandy or Silty soils	$W_L > 50$	
	CLAYS Above "A" line on plasticity chart: negligible organic content		CL	▨	Inorganic Clays of low plasticity, Gravelly, Sandy, or Silty Clays, lean Clays	$W_L < 30$	
			CI	▨	Inorganic Clays of medium plasticity, Silty Clays	$W_L > 30, < 50$	
			CH	▨	Inorganic Clays of high plasticity, fat Clays	$W_L > 50$	
	ORGANIC SILTS & ORGANIC CLAYS Below "A" line on plasticity chart		OL	▥	Organic Silts and organic Silty Clays of low plasticity	$W_L < 50$	
			OH	▨	Organic Clays of high plasticity	$W_L > 50$	

The soil of each stratum is described using ASTM D2487 and D2488 modified slightly so that an inorganic clay of "medium plasticity" is recognized.

ADDITIONAL SOIL CLASSIFICATION

▦ Fill Soil

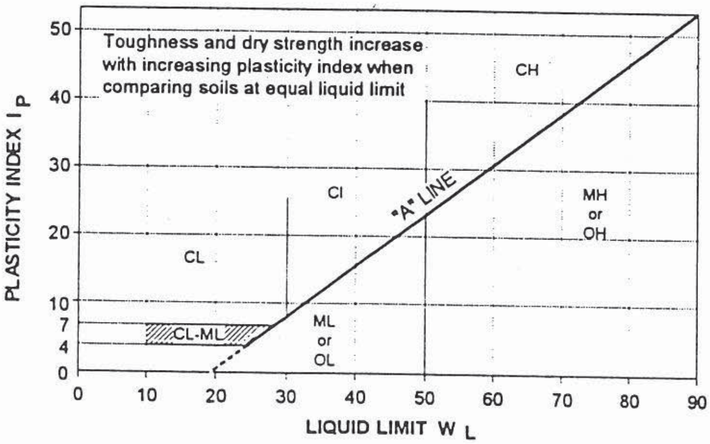
▤ Ss Sandstone

▩ Cs Claystone

▤ Ms Siltstone

v v Jsp Undifferentiated Metavolcanics

PLASTICITY CHART


Toughness and dry strength increase with increasing plasticity index when comparing soils at equal liquid limit

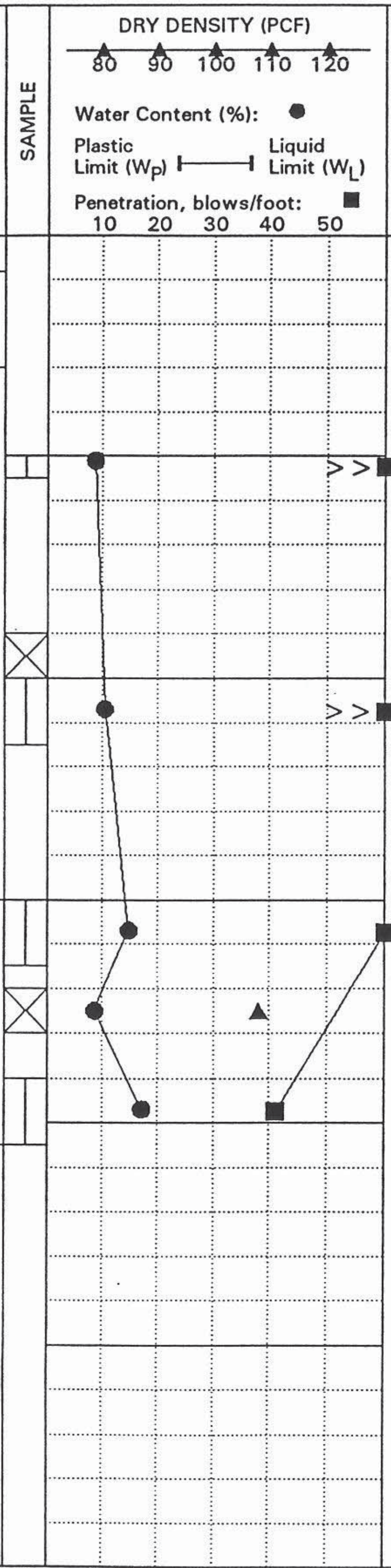
GEOBASE, INC.

EXPLANATION OF TERMS AND SYMBOLS

Figure B-2

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ▯ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ▤ DISTURBED ⧅ NO RECOVERY ▦ CORE



DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	<u>ASPHALTIC CONCRETE</u> (9.0").		
	<u>AGGREGATE BASE</u> (Fill) brown, sand, trace clay, little gravel, moist.	GP	
	<u>GRAVEL</u> (Lindavista Formation) reddish brown, sand matrix, fine to medium grained, silty, cobble and boulder sizes, very dense damp. (Conglomerate).	GM	
5	... tan to light brown, trace clay.		50 blows for 6 inches
	... light brown, coarse grained.		
			Corrosivity tests
10	... tan, fine grained.		N=94
15	<u>SANDSTONE</u> (Lindavista Formation) tan, fine grained, trace silt, very weathered, slightly laminated, cementeddense, moist.	Ss	N=60
	... little silt.		
20			
	• End of Boring at 20.5 feet. • Boring dry at completion of drilling.		
25			
30			

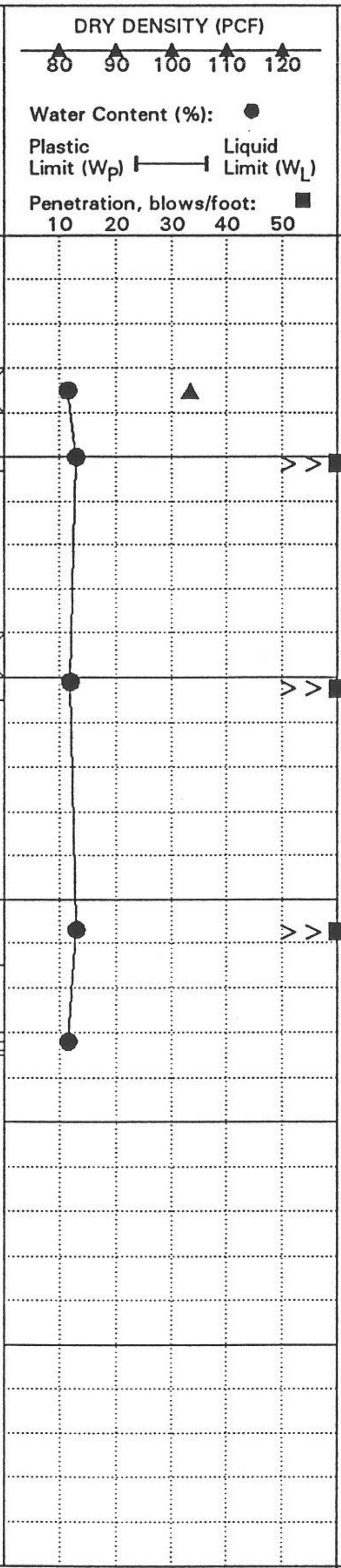
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-3
	DEPTH TO WATER ⊻	SURFACE ELEV. ± 394 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ⊼	DRILL CME-75	DATE LOGGED 11/23/93	FIGURE NO. B-5

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ▦ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (12.0").			
	AGGREGATE BASE (Fill) brown, sand, silty, trace gravel, damp.	GM		
	SAND (FILL) brown/greenish brown, clayey, trace asphalt pieces, trace gr avel, moist.	SC	☒	
5	GRAVEL (Lindavista Formation) orange brown, sand matrix, fine to medium grained, silty, trace clay, some cobbles and boulders, very dense, moist. (Interbedded Conglomerate and Sandstone). ... boulders. ... boulders. ... boulders. ... brown, coarse grained.	GM	◫	50 blows for 2 inches
10	... orange brown, medium to coarse grained, gravelly. ... gravelly.		☒ ◫	Poor sample 50 blows for 6 inches
15	SANDSTONE (Lindavista Formation) brown/orange brown, fine to medium grained, massive, trace silt, trace to little gravel, very dense, moist.	Ss	◫ ☰	N=77
20	• End of Boring at 19.0 feet. • Boring dry at completion of drilling. • Concrete pipe of approximately 42 inches diameter was encountered at about 4.0 feet below existing ground surface.			
25				
30				



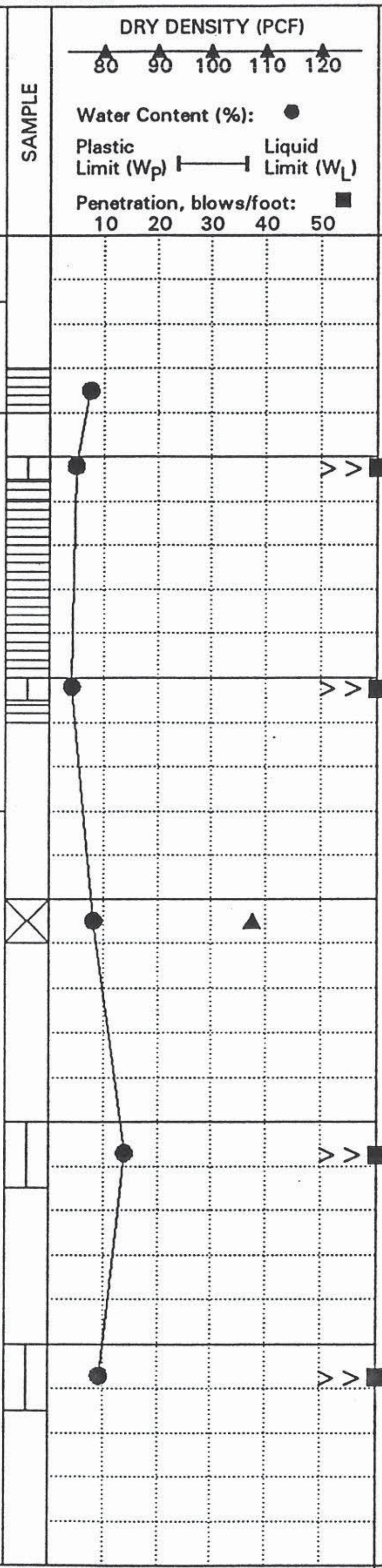
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-4
	DEPTH TO WATER	SURFACE ELEV. ± 400 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 11/24/93	FIGURE NO. B-6

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	SAND (Possible fill) brown/dark brown, clayey, little gravel, trace cobb les. moist.	SC	
	CLAY (Lindavista Formation) reddish brown, trace sand, damp.	CL	
	GRAVEL (Lindavista Formation) tan, sand matrix, silty, cobble and boulder sizes, very dense, dry to damp. (Conglomerate).	GM	
5			50 blows for 4 inches
	... cobbles.		Bulk sample from 5 to 10 feet
10	... very gravelly/cobbley.		70 blows for 6 inches Corrosivity tests
	SANDSTONE (Lindavista Formation) greyish brown, fine to medium grained, little clay, massive, moist.	Ss	
15	... fine grained, trace clay, cemented, damp.		DS, C
20	... light brown, fine grained, trace silt, cemented, slightly laminated, damp, very dense.		N=78
25			N=85
30			

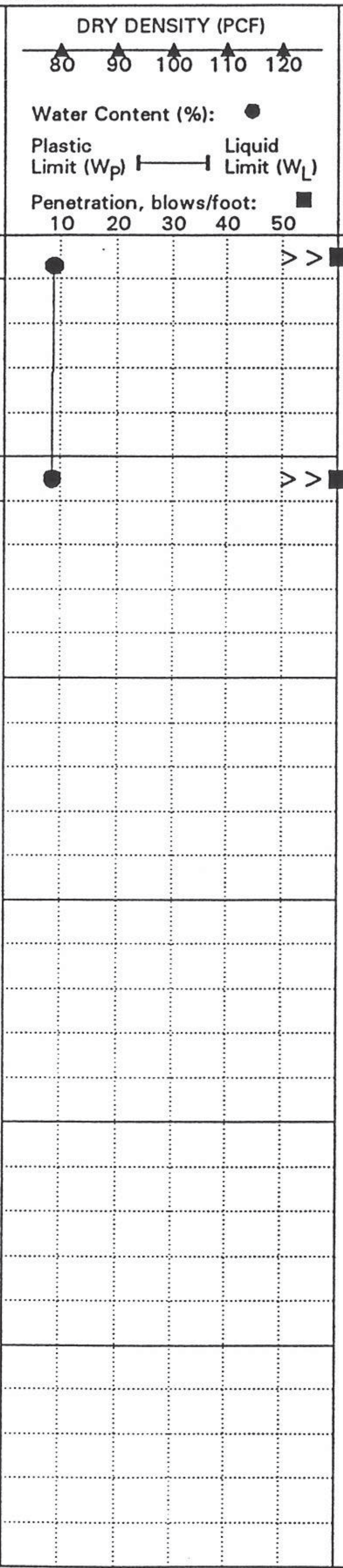


GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-5
	DEPTH TO WATER	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 03/10/94	FIGURE NO. B-7

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ▤ DISTURBED ⧄ NO RECOVERY ⊞ CORE



DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
		<u>SANDSTONE</u> (Lindavista Formation) light brown, fine grained, trace silt, cemented, slightly laminated, damp, very dense.	Ss		50 blows for 5 inches
35		... dry.			50 blows for 5 inches
		* End of Boring at 36.0 feet. * Boring dry at completion of drilling. * Surface elevation of Boring is approximate 5 feet above Miramar Road.			
40					
45					
50					
55					
60					

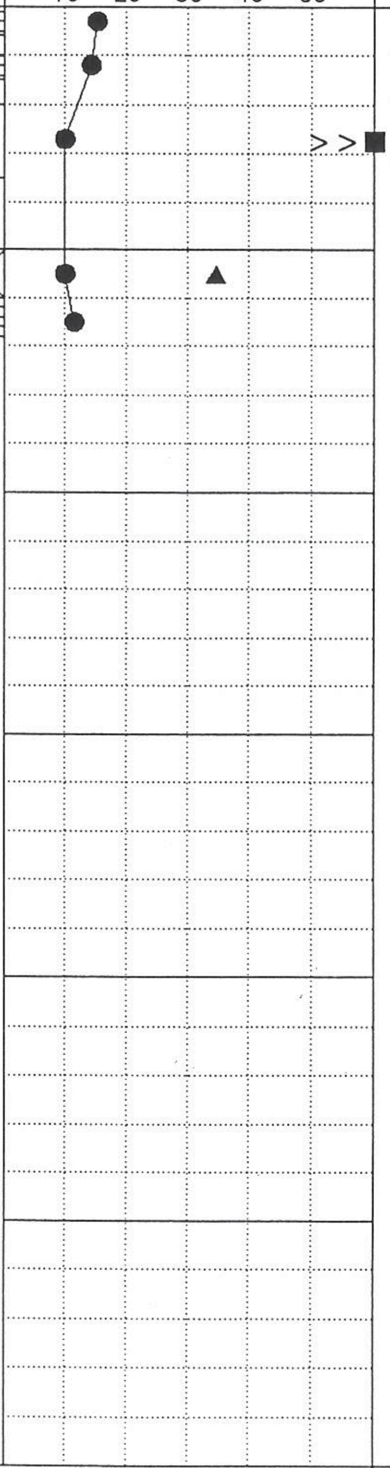
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-5
	DEPTH TO WATER ▽	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ⊼	DRILL CME-75	DATE LOGGED 03/10/94	FIGURE NO. B-7

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ▦ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) ▲ 80 90 100 110 120 Water Content (%): ● Plastic Limit (W_P) ├──┤ Liquid Limit (W_L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		<u>SAND</u> (Talus/Fill) brown, fine to medium grained, trace clay, moist to very moist. ... light brown, trace rounded gravel.	SP/ SC	☰		Bulk sample from 2 to 5 feet
		<u>SANDSTONE</u> (Lindavista Formation) tan, fine to medium grained, silty, very weathered, slightly laminated, very slightly cemented, very dense, damp to moist.	Ss	◫		N=67
5		... slightly cemented.		☒ ☰		C
		* End of boring at 7.0 feet. * Auger refusal on cobble/boulder at 7.0 feet. * Boring dry at completion of drilling.				
10						
15						
20						
25						
30						



GEOBASE, INC.

PROJECT Penasquitos Trunk Sewer Relief Project
San Diego, California

DEPTH TO WATER ▽ SURFACE ELEV. ± 369 feet LOGGED BY WYY

DEPTH TO SLOUGH ▲ DRILL Beaver Tripod DATE LOGGED 03/01/94

BORING NO. B-6

PROJECT NO. P.155.04

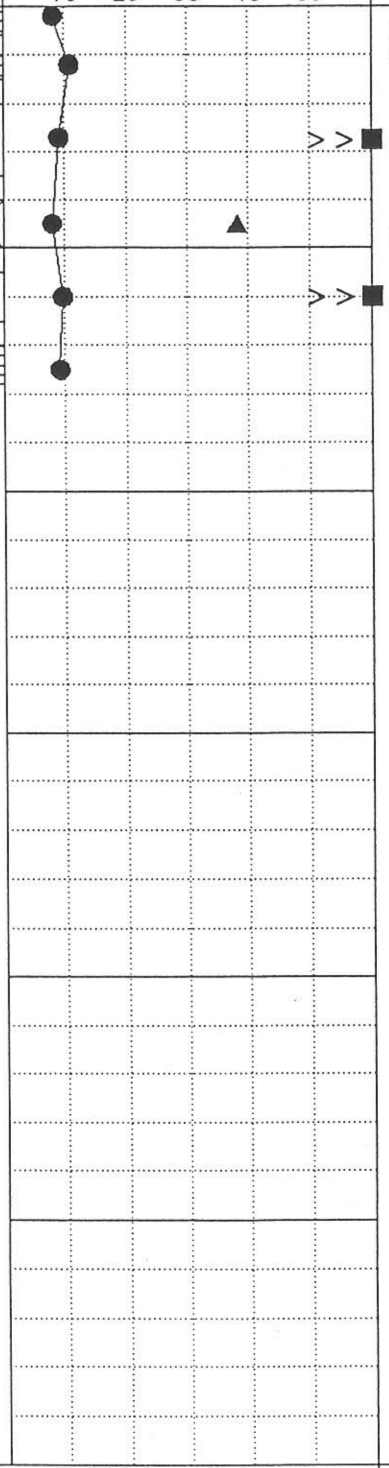
FIGURE NO. B-8

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ⊞ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) ▲ 80 90 100 110 120 Water Content (%): ● Plastic Limit (W_P) ⊢⊣ Liquid Limit (W_L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
	☒	SAND (Talus/Fill) brown, fine to medium grained, trace silt, trace rounded gravel, moist to very moist.	SP	☰		Bulk sample from 2 to 4 feet
	▨	SANDSTONE (Lindavista Formation) tan, fine to medium grained, very slightly laminated, slightly cemented, very dense, moist.	Ss	◫	> > ■	N=85
		... light brown, slightly laminated, damp.		☒	▲	DS
5		... light grey, slitstone lenses.		◫	> > ■	50 blows for 5 inches
		... trace rounded gravel.		☰		
10		* End of Boring at 8.5 feet. * Auger refusal on cobble/boulder at 8.5 feet. * Boring dry at completion of drilling.				
15						
20						
25						
30						



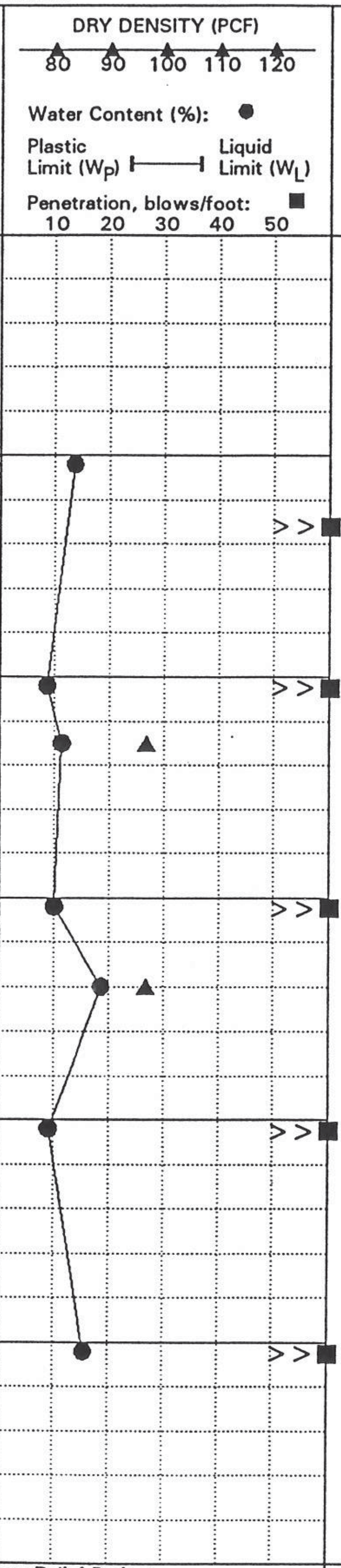
GEOBASE, INC.	PROJECT	**Penasquitos Trunk Sewer Relief Project** **San Diego, California**		BORING NO. **B-7**
	DEPTH TO WATER ▼	SURFACE ELEV. **± 369 feet**	LOGGED BY **WYY**	PROJECT NO. **P.155.04**
	DEPTH TO SLOUGH ▲	DRILL **Beaver Tripod**	DATE LOGGED **03/01/94**	FIGURE NO. **B-9**

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ▦ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (15.0").		
	AGGREGATE BASE (Fill) tan, sand, little gravel, damp.	SM/ GP	
	CLAY (Fill) brown, medium plastic, damp to moist.	CL	
	... cobbles.		
5			
	GRAVEL (Lindavista Formation) tan/light brown, sand matrix, cobbles and boulders sizes, very dense, damp. (Conglomerate).	GM	50 blows for 2 inches
	... silty, very dense, dry.		
10			50 blows for 6 inches
	... moist.		
15			50 blows for i inch
	SANDSTONE (LIndavista Formation) tan, fine grained, silty, moist.	Ss	
	... trace clay, slightly laminated, slightly cemented.		
20	... orange/light brown, trace gravel, damp to moist.		50 blows for 5 inches
	... brown clayey sand lenses.		
	... gravelly, little cobbles.		
25	... moist.		50 blows for 5 inches
	* End of Boring at 25.5 feet. * Boring dry at completion of drilling.		
30			



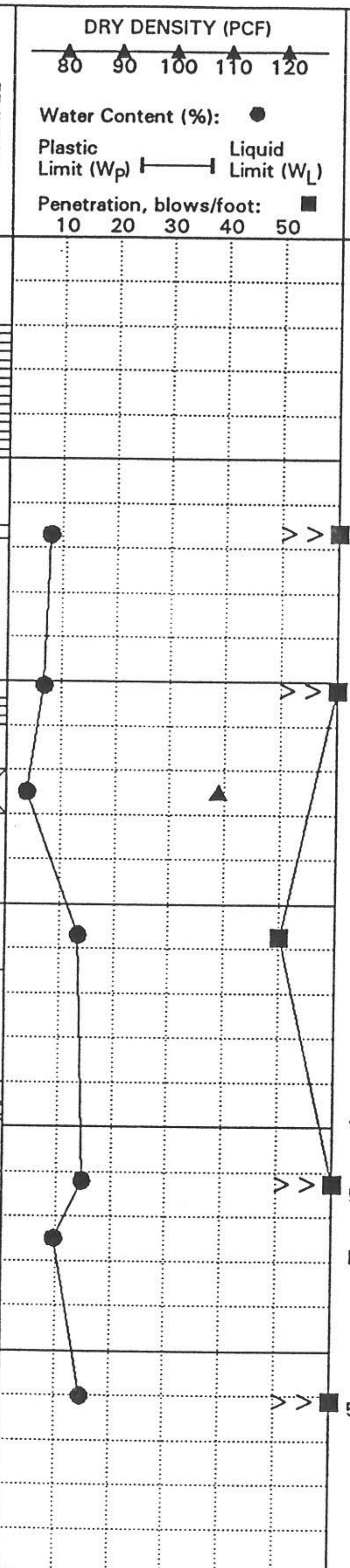
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California	BORING NO. B-8
	DEPTH TO WATER ⊻	SURFACE ELEV. ± 416 feet | LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ⊼	DRILL CME-75 | DATE LOGGED 11/30/93	FIGURE NO. B-10

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALT CONCRETE (12")		
	AGGREGATE BASE (Fill) light brown, medium grained, little gravel, damp to moist.	GP	
	SAND (Fill) dark brown, medium grained, some gravel, little clay, moist.	SC	Bulk sample from 2 to 5 feet
5	GRAVEL (Lindavista Formation) tan to light brown, sand matrix, cobbles and boulder sizes, very dense, damp. (Conglomerate).	GP	
	... gravelly and some cobbles.		50 blows for 3 inches
	... gravelly.		
10			50 blows for 4 inches Jar sample
	... brown, gravelly.		
			DS
15			N=50
	... cobbles/boulders.		
	... cobbles/boulders.		
20			Jar sample
	SANDSTONE (Lindavista Formation) tan, fine to medium grained, little silt, slightly cemented, slightly laminated, very dense, damp to moist.	Ss	50 blows for 6 inches
	... trace clay.		Poor samples
	... some gravel.		
25			
	... tan to brown, trace clay, dry. * End of Boring at 26.5 feet. * Boring dry at completion of drilling.		50 blows for 2 inches
30			

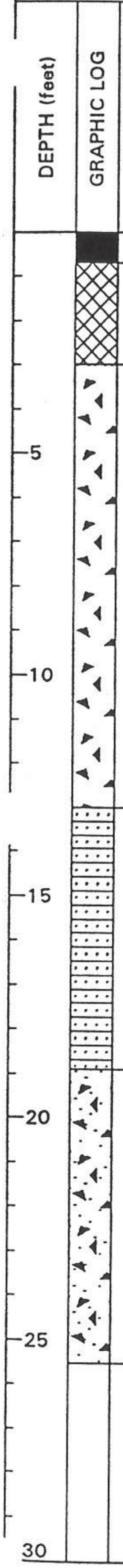


GEOBASE, INC.	PROJECT: Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-9
	DEPTH TO WATER	SURFACE ELEV. ± 425 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 11/30/93	FIGURE NO. B-11

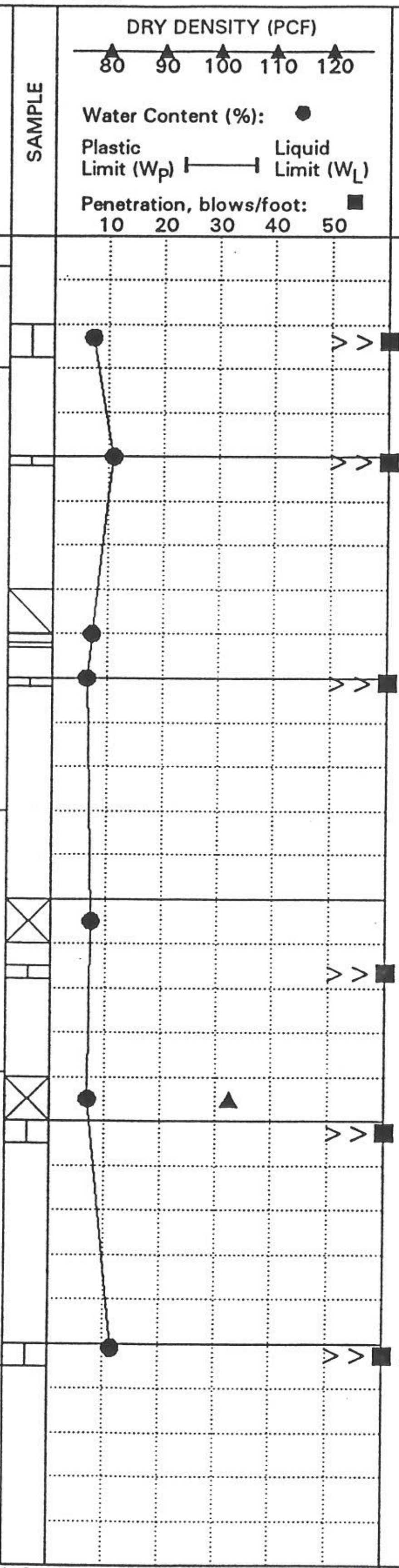
Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ⊞ CORE



SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
ASPHALTIC CONCRETE (8.0").		
SAND (Fill) brown, clayey, trace sandstone/siltstone fragment, trace gravel, very dense, moist.	SC	50 blows for 3 inches
GRAVEL (Lindavista Formation) light brown, sand matrix, silty, cobbles and boulder sizes, very dense, dry. (Conglomerate). ... some gravel/cobbles. ... trace clay.	GM	50 blows for 2 inches 60 blows for 3 inches
SANDSTONE (Lindavista Formation) tan to light brown, silty, trace rounded gravel, very dense, damp. ...claystone lenses. ... trace clay. ... some gravel.	Ss	Corrosivity tests Poor sample 70 blows for 3 inches No recovery
GRAVEL (Stadium Conglomerate) orange/light brown, sand matrix, trace clay, very dense, damp. ... cobbles. ... cobbles.	GP	50 blows for 5 inches 75 blows for 5 inches
* End of Boring at 25.5 feet. * Boring dry at completion of drilling.		



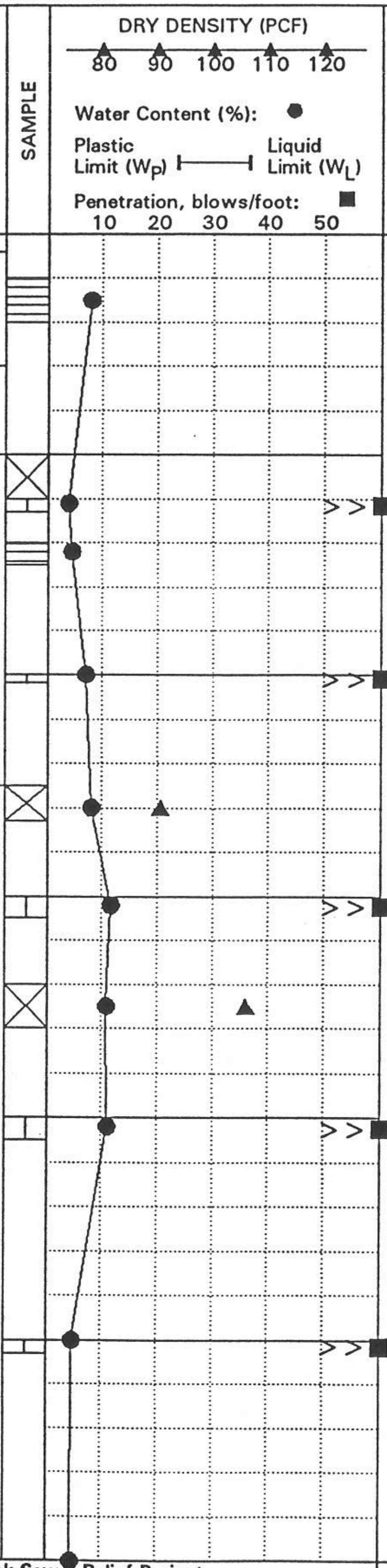
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California	BORING NO. B-10
	DEPTH TO WATER ▼	SURFACE ELEV. ± 421 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ▲	DRILL CME-75	DATE LOGGED 12/07/93	FIGURE NO. B-12

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (5.0").		
	AGGREGATE BASE (Fill) brown, sand, trace silt, trace gravel, moist.	SP/ SM	
	SAND (Fill) dark brown, little clay, moist.	SC	
5	GRAVEL (Lindavista Formation) greyish brown, sand matrix, trace silt, cobbles and boulder sizes, very dense, dry to damp. (Conglomerate).	GP	Poor sample 50 blows for 4 inches
	... tan.		
10			50 blows for 2 inches
	SANDSTONE (Lindavista Formation), light brown, little silt, slightly laminated, slightly cemented, very dense, dry.	Ss	
15			60 blows for 6 inches
20			75 blows for 6 inches
	... some gravel/cobbles.		
25	... little gravel.		100 blows for 4 inches
	... gravelly.		
30			



GEOBASE, INC.

PROJECT: Penasquitos Trunk Sewer Relief Project
San Diego, California

BORING NO. B-11

DEPTH TO WATER

SURFACE ELEV. ± 429 feet

LOGGED BY WYY

PROJECT NO. P.155.04

DEPTH TO SLOUGH

DRILL CME-75

DATE LOGGED 12/01/93

FIGURE NO. B-13

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ▦ CORE

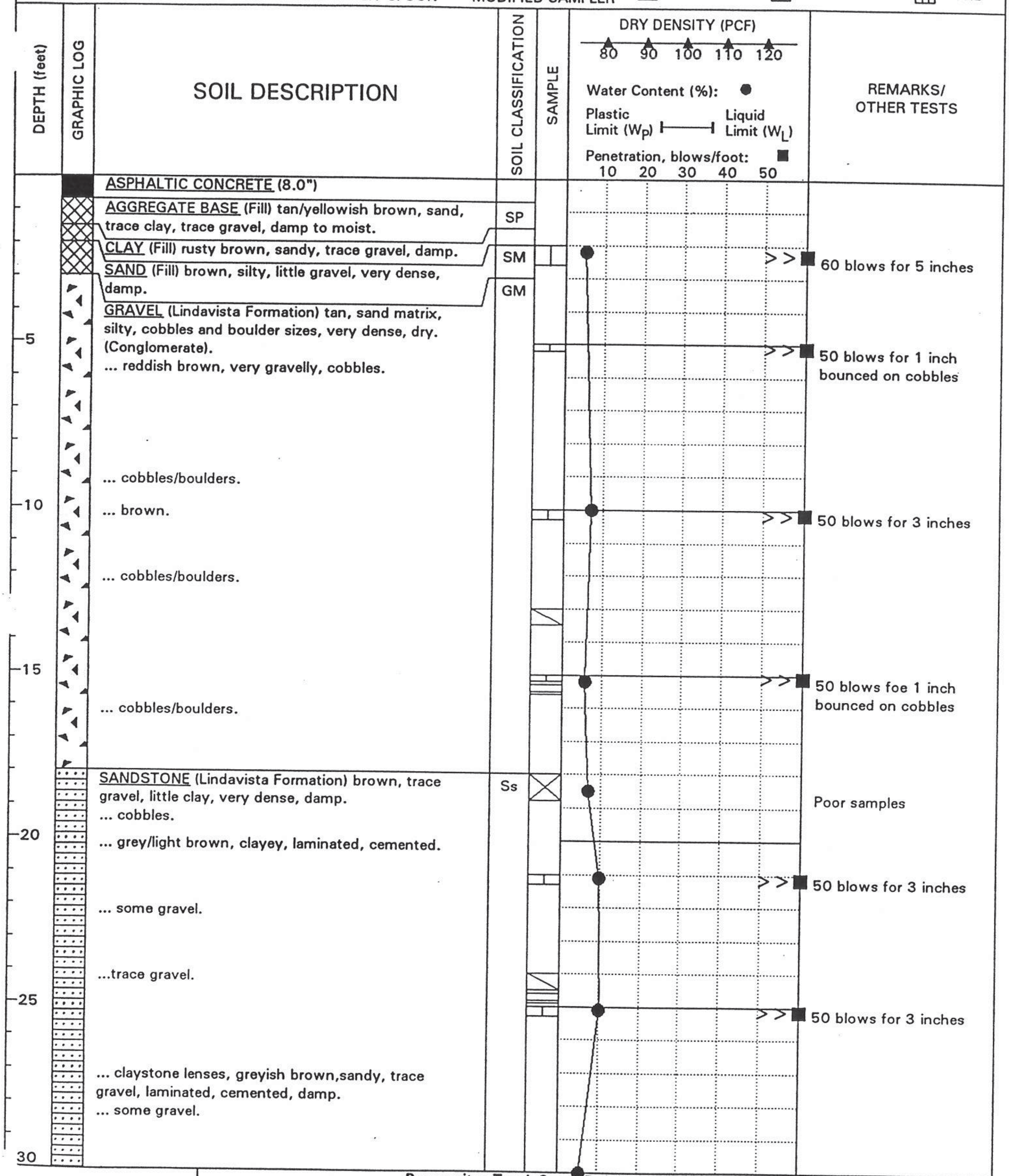
DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	DRY DENSITY (PCF) 80 90 100 110 120 Water Content (%): ● Plastic Limit (W_P) ├──┤ Liquid Limit (W_L) Penetration, blows/foot: ■ 10 20 30 40 50	REMARKS/ OTHER TESTS
		... gravelly.		◫	● >> ■	50 blows for 3 inches
		* End of Boring at 30.5 feet. * Boring dry at completion of drilling.				
35						
40						
45						
50						
55						
60						

GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-11
	DEPTH TO WATER ▽	SURFACE ELEV. ± 429 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/01/93	FIGURE NO. B-13

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE



ASPHALTIC CONCRETE (8.0")

AGGREGATE BASE (Fill) tan/yellowish brown, sand, trace clay, trace gravel, damp to moist. — SP

CLAY (Fill) rusty brown, sandy, trace gravel, damp. — SM

SAND (Fill) brown, silty, little gravel, very dense, damp. — GM

GRAVEL (Lindavista Formation) tan, sand matrix, silty, cobbles and boulder sizes, very dense, dry. (Conglomerate).
... reddish brown, very gravelly, cobbles.

... cobbles/boulders.

... brown.

... cobbles/boulders.

... cobbles/boulders.

SANDSTONE (Lindavista Formation) brown, trace gravel, little clay, very dense, damp. — Ss
... cobbles.

... grey/light brown, clayey, laminated, cemented.

... some gravel.

...trace gravel.

... claystone lenses, greyish brown,sandy, trace gravel, laminated, cemented, damp.
... some gravel.

REMARKS/OTHER TESTS:

60 blows for 5 inches

50 blows for 1 inch bounced on cobbles

50 blows for 3 inches

50 blows foe 1 inch bounced on cobbles

Poor samples

50 blows for 3 inches

50 blows for 3 inches

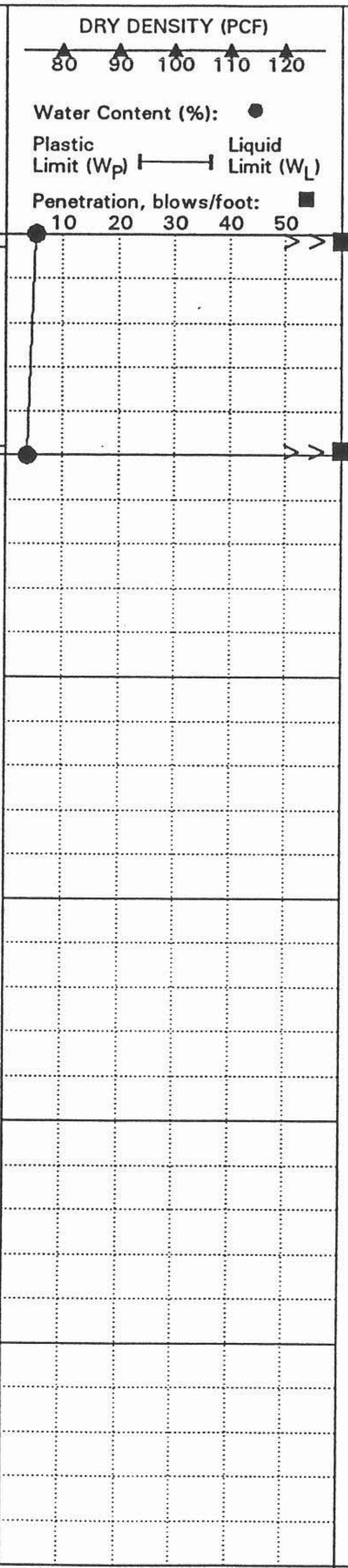
GEOBASE, INC.	PROJECT Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-12
	DEPTH TO WATER ⊻	SURFACE ELEV. ± 440 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ⊼	DRILL CME-75	DATE LOGGED 12/07/93	FIGURE NO. B-14

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
		SANDSTONE (as above). ... very gravelly. ... gravel/cobbles.	Ss	◫	70 blows for 3 inches
35		* End of Boring at 35.0 feet. * Boring dry at completion of drilling.		◫	50 blows for 2 inches
40					
45					
50					
55					
60					



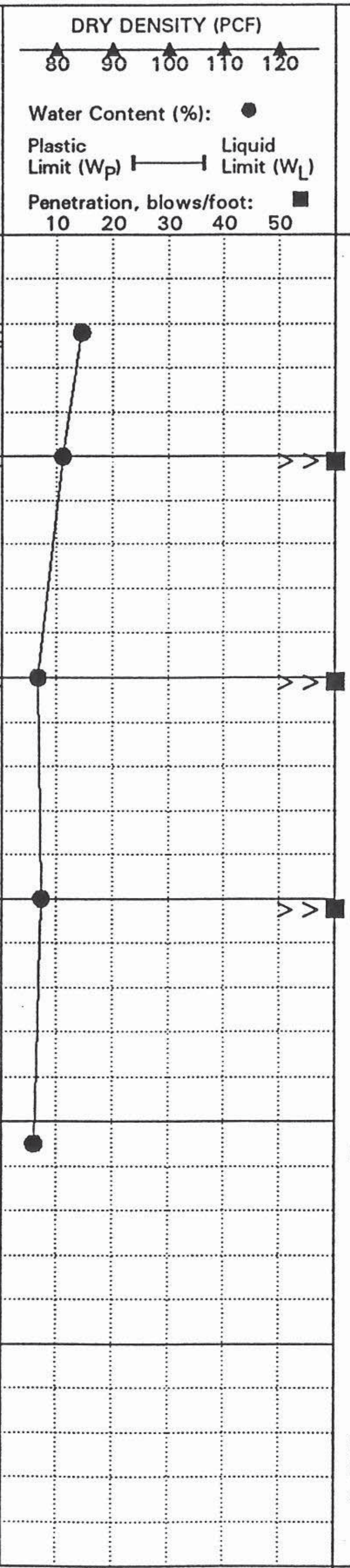
GEOBASE, INC.	PROJECT Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-12
	DEPTH TO WATER	SURFACE ELEV. ± 440 feet / LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75 / DATE LOGGED 12/07/93	FIGURE NO. B-14

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ⊞ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (5.0")		
	AGGREGATE BASE (Fill) brown, sand, trace gravel, little clay, moist.	SC	
	GRAVEL (Lindavista Formation) brown, sand matrix, cobbles and boulder sizes, very dense, damp. (Conglomerate).	GP	
	... very gravelly, cobbles.		
5	... cobbles.		50 blows for 2 inches
	... orange/brown.		
	... cobbles/boulders.		
10	... tan/brown, trace rock fragments.		50 blows fpr 2 inches
	... cobbles/boulders.		
15	... brown/dark brown, trace silt.		60 blows for 6 inches
	... cobbles/boulders.		
20			Poor sample
	... cobbles.		
25	... cobbles.		
	* End of Boring at 27.2 feet. * Boring dry at completion of drilling.		50 blows for 1 inch Bounced on cobbles, no recovery
30			



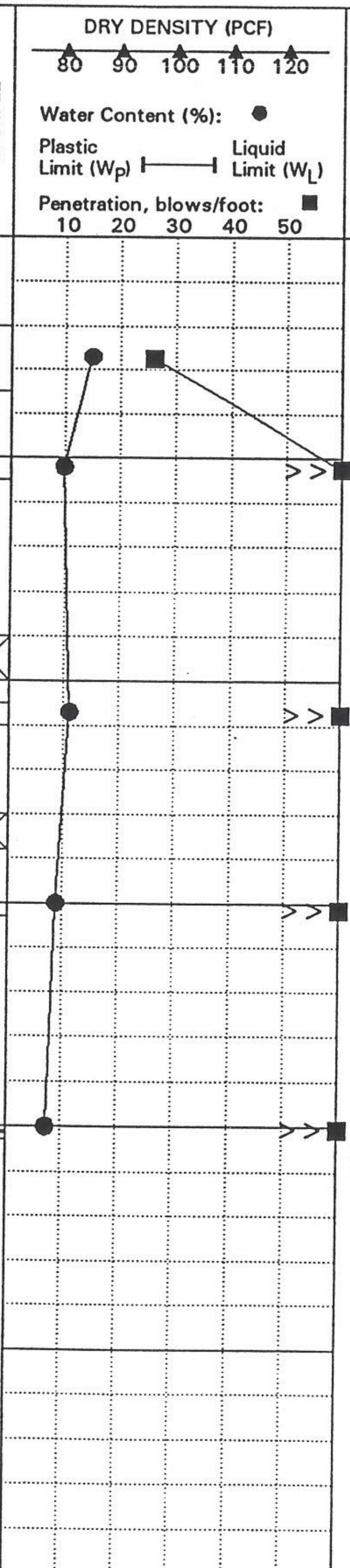
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California	BORING NO. B-13
	DEPTH TO WATER	SURFACE ELEV. ± 437 feet / LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75 / DATE LOGGED 12/01/93	FIGURE NO. B-15

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (5.0").		
	AGGREGATE BASE (Fill) tan/ yellowish brown, sand, trace silt, trace gravel, dry to damp.	SP	
	CLAY (Lindavista Formation) dark brown, little sand, medium plastic, very stiff, moist.	CL	PP=0.5 to 1.0 tsf
5	GRAVEL (Lindavista Formation) light brown, sand matrix, trace silt, very dense, damp. (Conglomerate).	GP	50 blows for 6 inches
	... very gravelly.		
10	SANDSTONE (Lindavista Formation) tan, trace silt, slightly laminated, sl ightly cemented, very dense, damp.	Ss	Corrosivity tests
	... siltstone lenses, yellowish brown, trace gravel, laminated, cemented, very dense, damp.		55 blows for 6 inches
	... claystone layer, yellowish brown, trace sand, trace rounded gravel,slightly plastic, hard, damp.		
15	... claystone layer, little gravel.		60 blows for 3 inches
	... cobbles.		
	... some gravel.		
20	* End of Boring at 20.3 feet. * Boring dry at completion of drilling.		60 blows for 2 inches No recovery
25			
30			

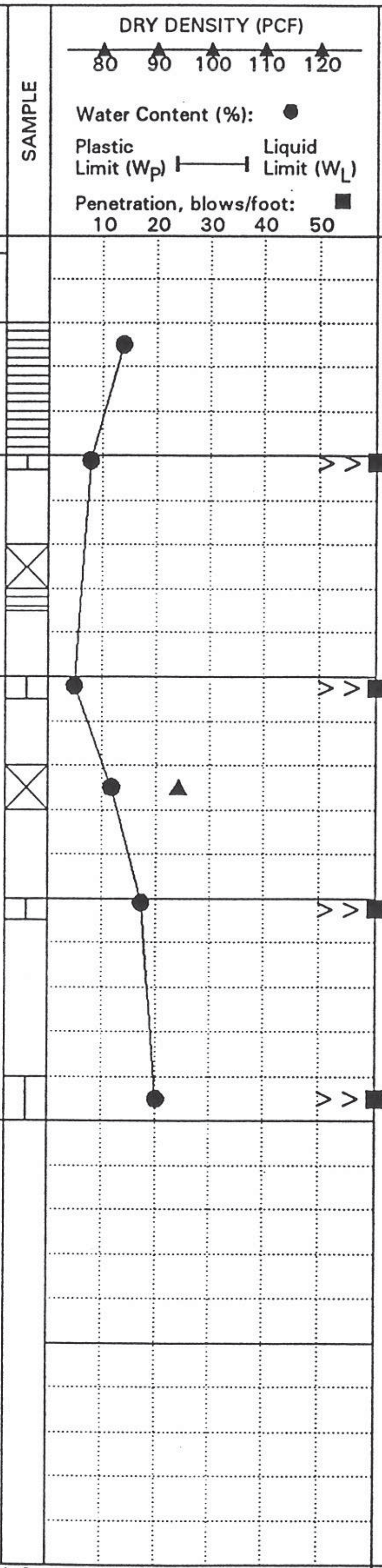


GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California	BORING NO. B-14
	DEPTH TO WATER	SURFACE ELEV. ± 443 feet / LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75 / DATE LOGGED 12/08/93	FIGURE NO. B-16

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ▤ DISTURBED ⧅ NO RECOVERY ⊞ CORE



DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (5.0").		
	AGGREGATE BASE (Fill) tan/yellowish brown, sand, trace clay, trace gravel, damp to moist.	SP	
	SAND (Fill) brown, clayey, trace gravel, moist.	SC	Bulk sample from 2 to 5 feet
	... trace clay, some gravel.		
5	GRAVEL (Lindavista Formation) tan, sand matrix, trace silt, cobbles and boulder sizes, very dense, dry. (Conglomerate).	GP	Jar sample 50 blows for 3 inches
			Corrosivity tests
			Jar sample
10	SANDSTONE (Lindavista Formation) brown, little clay, very dense, dry.	Ss	50 blows for 6 inches
	... trace clay.		C
15			50 blows for 6 inches
	... siltstone layer, brown, trace sand, very dense, moist.		50 blows for 6 inches
20	* End of Boring at 20.0 feet. * Boring dry at completion of drilling.		
25			
30			

GEOBASE, INC.

PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-15
DEPTH TO WATER ▼	SURFACE ELEV. ± 472 feet	LOGGED BY WYY	PROJECT NO. P.155.04
DEPTH TO SLOUGH ▲	DRILL CME-75	DATE LOGGED 12/01/93	FIGURE NO. B-17

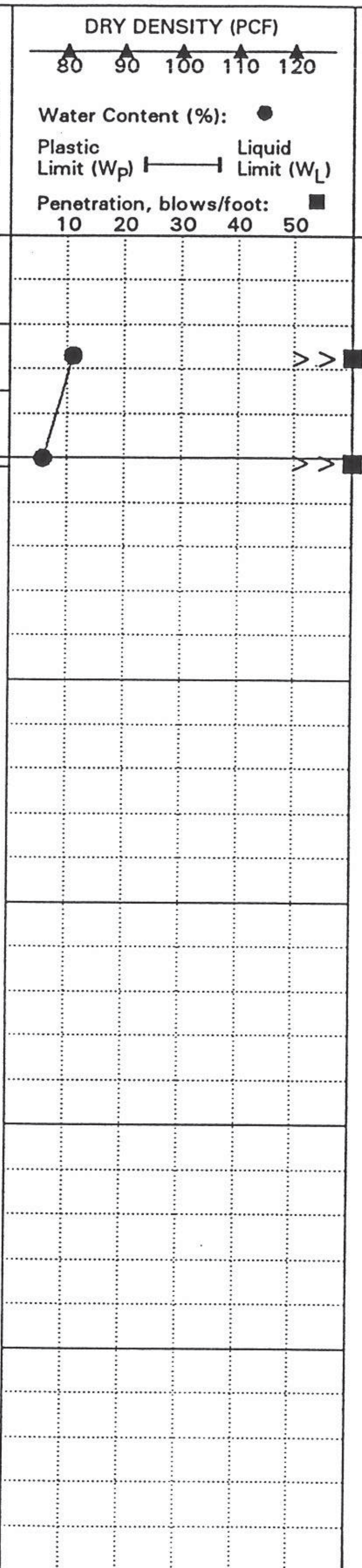
Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧅ NO RECOVERY ▦ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	REMARKS/ OTHER TESTS
	ASPHALTIC CONCRETE (5.0").		
	AGGREGATE BASE (Fill) tan/yellowish brown, sand, trace clay, trace gravel, moist.	SP	
	CLAY (Lindavista Formation) dark brown, sandy, trace gravel, very dense, moist.	CL	
	GRAVEL (Lindavista Formation) tan, sand matrix, trace silt, cobbles and boulder sizes, very dense, dry. (Conglomerate).	GP	N=80
5	... cobbles/boulders.		50 blows for 2 inches
10			

- End of Boring at 11.0 feet.
- Auger refusal on boulder at 11.0 feet.
- Boring dry at completion of drilling.



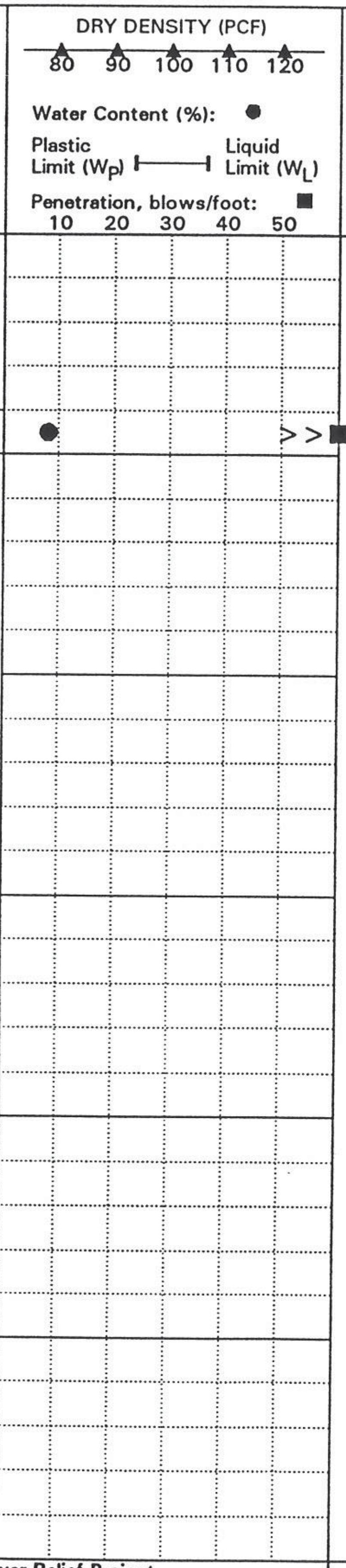
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-16
	DEPTH TO WATER	SURFACE ELEV. ± 491 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH	DRILL CME-75	DATE LOGGED 12/08/93	FIGURE NO. B-18

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ⊟ DISTURBED ⧅ NO RECOVERY ⊞ CORE

DEPTH (feet)	GRAPHIC LOG	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	■	ASPHALTIC CONCRETE (18.0" - 6.5" new AC over 12.0" old AC).			
	▨	AGGREGATE BASE (Fill) yellowish brown, sand, silty, trace gravel, damp.	GP		
5	▨	SAND (Fill) reddish brown, trace silt, little gravel, very dense, damp.	SP	◫	80 blows for 9 inches
		• End of Boring at 5.5 feet. • Pipe was encountered at approximately 5.5 feet. • Boring dry at completion of drilling.			
10					
15					
20					
25					
30					



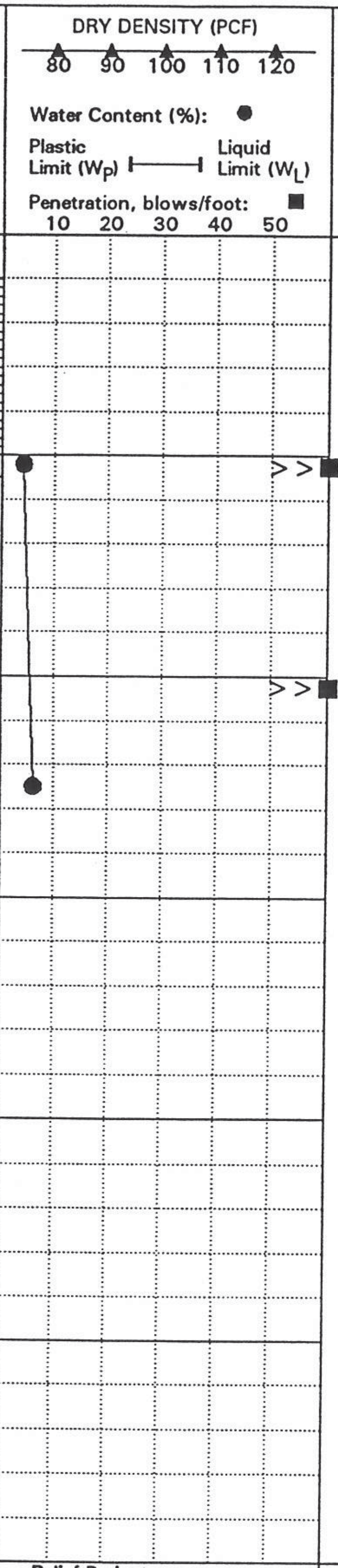
GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-40
	DEPTH TO WATER ▼	SURFACE ELEV. ± 407 feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ▲	DRILL CME-75	DATE LOGGED 11/30/93	FIGURE NO. B-39

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

LOG OF BORING

SAMPLE TYPE: ■ THIN WALLED TUBE ◫ SPT SPLIT SPOON ☒ CALIFORNIA MODIFIED SAMPLER ☰ DISTURBED ⧄ NO RECOVERY ▦ CORE

DEPTH (feet)	SOIL DESCRIPTION	SOIL CLASSIFICATION	SAMPLE	REMARKS/ OTHER TESTS
	<u>ASPHALTIC CONCRETE</u> (9.0")			
	<u>AGGREGATE BASE</u> (Fill) light brown/tan, sand ,trace clay, little gravel, moist.	GP	☰	Bulk sample from 1 to 5 feet
	<u>CLAY</u> (Lindavista Formation) reddish brown, sandy, trace gravel, moist.	SC	☰	
	<u>GRAVEL</u> (Lindavista Formation) tan/light brown, sand matrix, fine to medium grained, silty, cobble and boulder sizes, very dense, dry to damp. (Conglomerate).	GP	☰	
5	... gravelly/cobbles.		◫	Jar sample 50 blows for 5 inches
	... brown, gravelly/cobbles.			
	... cobbles.			
			☰	
10			◫	Jar sample 50 blows for 4 inches
	<u>SANDSTONE</u> (Lindavista Formation) brown, medium grained, trace clay, moist.	Ss	☰	
15			☒	
	* End of Boring at 16.5 feet. * Boring dry at completion of drilling.			
20				
25				
30				



GEOBASE, INC.	PROJECT	Penasquitos Trunk Sewer Relief Project San Diego, California		BORING NO. B-41
	DEPTH TO WATER ⊻	SURFACE ELEV. ± feet	LOGGED BY WYY	PROJECT NO. P.155.04
	DEPTH TO SLOUGH ⊼	DRILL CME-75	DATE LOGGED 03/10/94	FIGURE NO. B-40

Note: This log of BORING should be evaluated in conjunction with the complete geotechnical report. This log of BORING represents conditions observed at the specific BORING location and at the date indicated.

APPENDIX B

GEOPHYSICAL SURVEY RESULTS

SEISMIC SURVEY
by Southwest Geophysics, Inc.
dated October 31, 2016

TerraCosta
Consulting Group

SEISMIC SURVEY
NCCS MIRAMAR PIPELINE
SAN DIEGO, CALIFORNIA

PREPARED FOR:
TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 200
San Diego, CA 92123

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

October 31, 2016
Project No. 116500

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

October 31, 2016
Project No. 116500

Mr. Bob Smillie
TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 1200
San Diego, CA 92123

Subject: Seismic Survey
NCCS Miramar Pipeline
San Diego, California

Dear Mr. Smillie:

In accordance with your authorization, we have performed a seismic survey for the proposed NCCS Miramar Pipeline project located in San Diego, California. Specifically, our survey consisted of performing six P-wave refraction traverses and one refraction microtremor (ReMi) profile at the project site. The purpose of our study was to develop subsurface velocity profiles in the study area. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Hans van de Vrugt

Aaron T. Puente
Project Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

ATP/HV/hv

Distribution: (1) Addressee (electronic)

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/18
STATE OF CALIFORNIA

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic survey for the proposed NCCS Miramar Pipeline project located in San Diego, California (Figure 1). Specifically, our survey consisted of performing six P-wave refraction traverses and one refraction microtremor (ReMi) profile at the project site. The purpose of our study was to develop subsurface velocity profiles in the study area. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of six seismic P-wave refraction lines, SL-1 through SL-6.
- Performance of one ReMi profile, RL-1 in the same location as SL-6.
- Compilation and analysis of the data collected.
- Preparation of this illustrated data report presenting our results.

3. SITE DESCRIPTION AND PROJECT DESCRIPTION

The project site generally includes two areas located along Scripps Lake Drive between Red Cedar Drive and Scripps Ranch Boulevard in the Miramar area of San Diego (Figure 1). One area is located along the south side of Scripps Lake Drive adjacent to Evans Pond. The second area is located along the north side of Scripps Lake Drive adjacent to the entrance road to Lake Miramar. Figures 2 and 3 depict the general site conditions in the study areas and along the seismic lines.

Based on our discussions with you, it is our understanding your office is conducting a geotechnical evaluation of the site for the proposed pipeline project. The results of our survey will be used in the formulation of design and construction parameters for the project.

4. SURVEY METHODOLOGY AND ANALYSIS

As previously indicated, the primary purpose of our services was to characterize the subsurface conditions at pre-selected locations through the collection of seismic data. The following sections provide an overview of the methodologies used during our study.

4.1. P-wave Refraction Survey

The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves (compression waves) generated at the surface are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones, and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse. The refraction method requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by buried boulders, fractures, dikes, etc. can result in the misinterpretation of the subsurface conditions.

Six seismic P-wave traverses, SL-1 through SL-6, were conducted at the site. The location of the profiles, which were selected by your office, and the line lengths are depicted on Figure 2. Multiple shot points (signal generator locations) were conducted at the ends, midpoint, and intermediate points along the lines. The P-wave signal (shot) was generated using a 16-pound hammer and an aluminum plate.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2011) as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability.

The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

4.2. ReMi Survey

The refraction microtremor technique uses recorded surface waves (specifically Rayleigh waves) that are contained in the background noise to develop a shear wave velocity profile of the site. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one dimensional sounding which represents the average condition across the length of the line. Unlike the refraction method, described above, the ReMi method does not require an increase of material velocity with depth. Therefore, low velocity zones (velocity inversions) are detectable with ReMi.

One 230-foot long ReMi line, RL-1, was performed at the site in the same location as SL-6. Fifteen records, 30 seconds long were collected with a 24-channel Geometrics Geode seismograph and 4.5-Hz vertical component geophones.

Collected ReMi data were processed using SeisOpt® ReMi™ software (© Optim LLC, 2005), which uses the refraction microtremor method (Louie, 2001). The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determines the best fitting model. The result is a one-dimensional shear-wave velocity model of the site with roughly 5 to 15 percent accuracy.

5. RESULTS

Figures 4a through 4f present the results from the P-wave refraction survey and Figure 5 presents the ReMi results. Based on the velocity models generated from our P-wave analysis it appears that the study areas are underlain by low velocity materials (e.g., colluvium and topsoil) in the very near surface, and bedrock with varying degrees of weathering. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the study areas. In addition, remnant boulders appear to be present in the subsurface.

The results from the ReMi line are generally consistent with the P-wave results for SL-6 with some slight variations. The variations are likely due to the averaging effect of the ReMi method.

6. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

7. SELECTED REFERENCES

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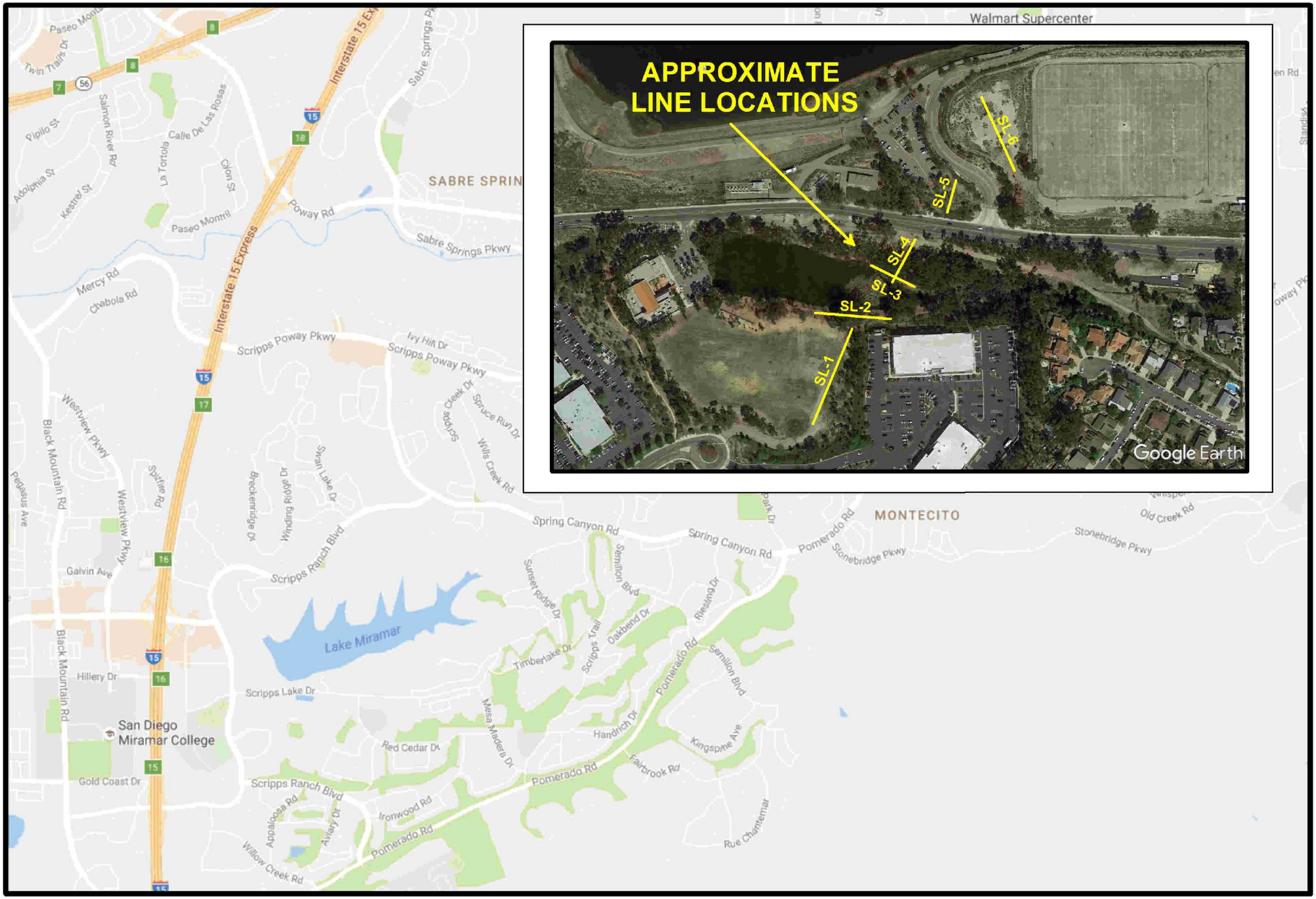


SITE LOCATION MAP

N

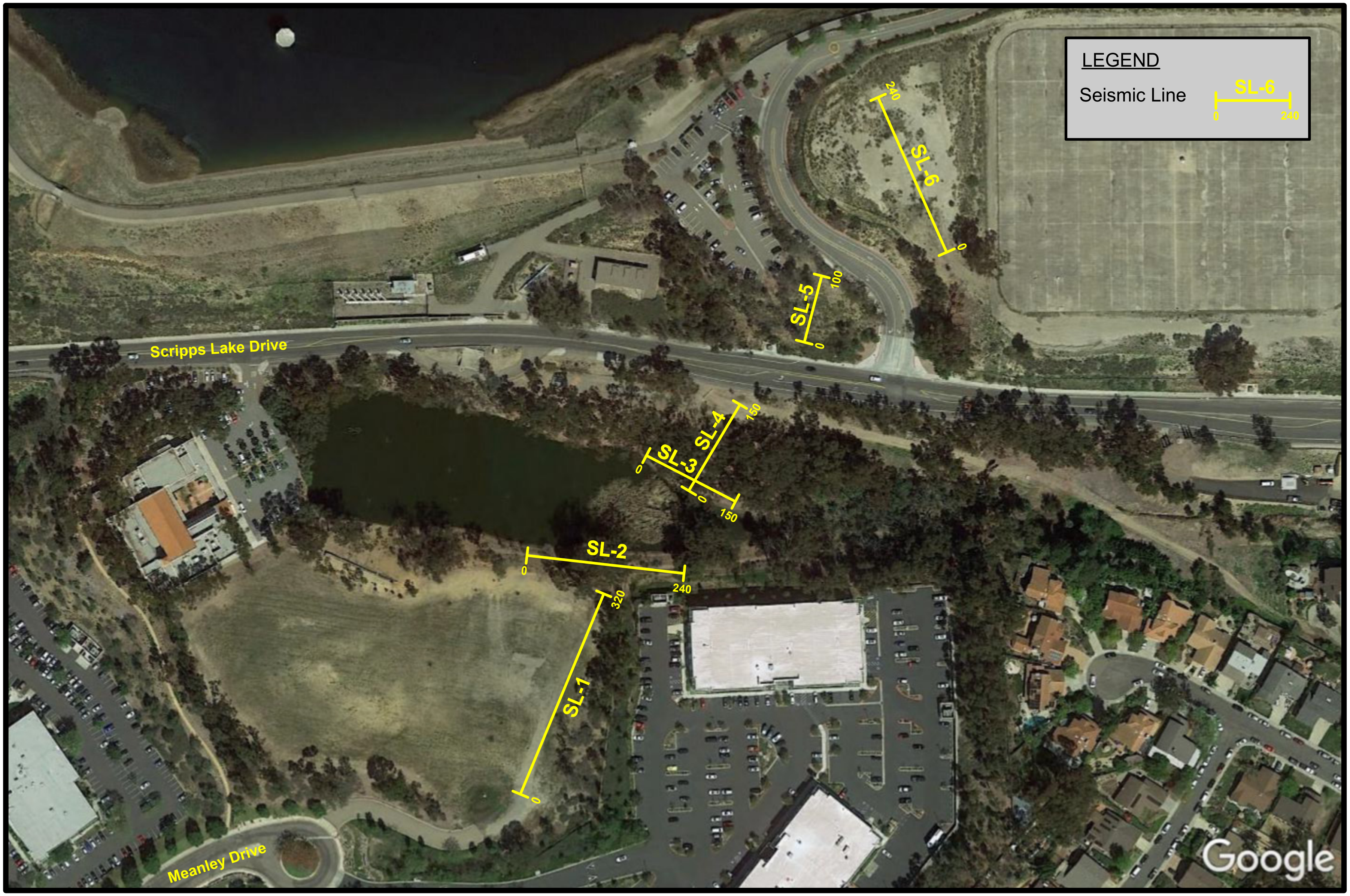
NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

Date: 10/16

SOUTHWEST
GEOPHYSICS INC.

Figure 1



SEISMIC LINE LOCATION MAP

N

NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

Date: 10/16

SOUTHWEST GEOPHYSICS INC.

Figure 2

0 200 400

approximate scale in feet

SITE PHOTOGRAPHS

NCCS Miramar Pipeline
San Diego, California

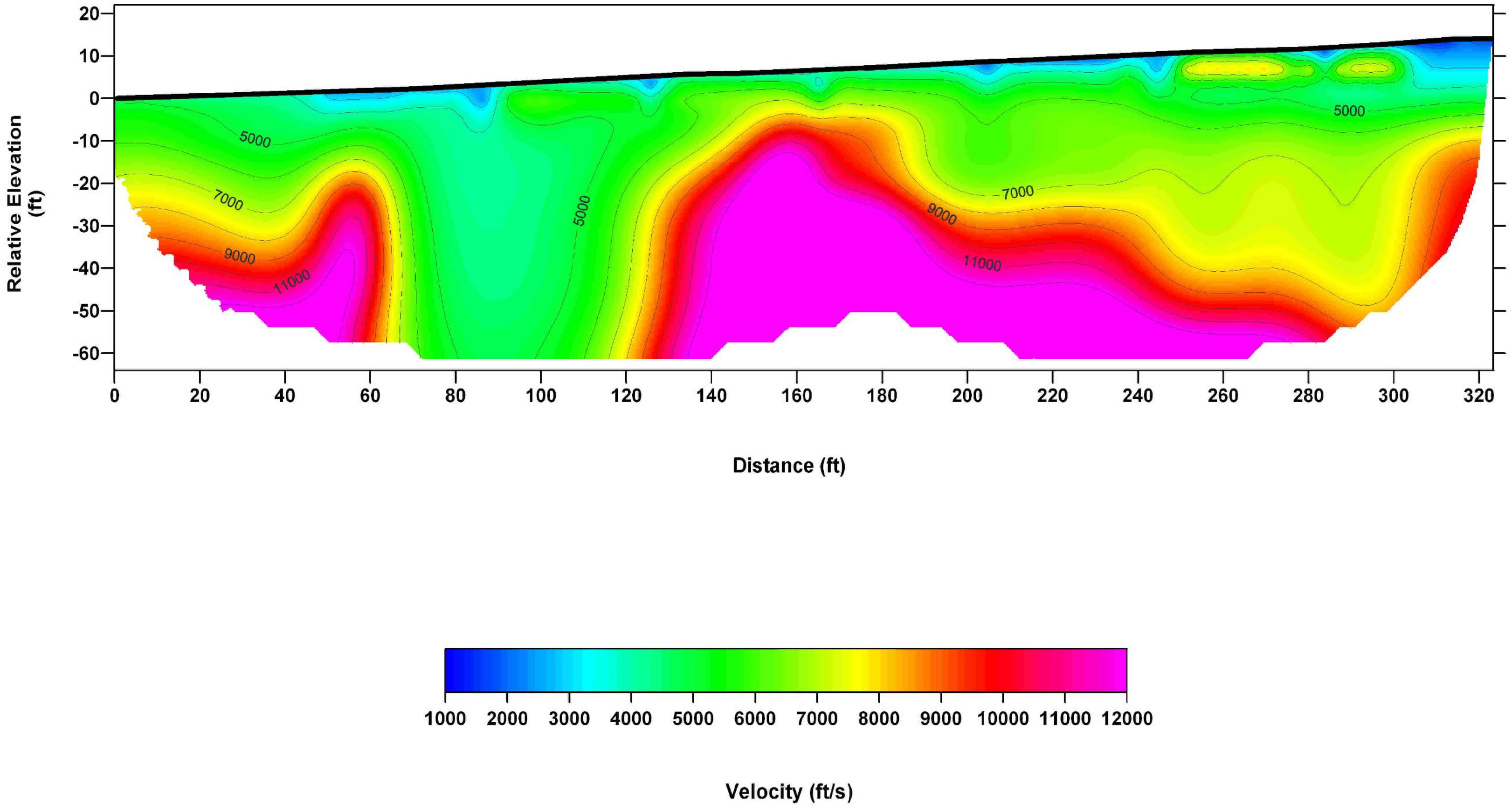
Project No.: 116500

Date: 10/16

SOUTHWEST GEOPHYSICS INC.

Figure 3

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-1

NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

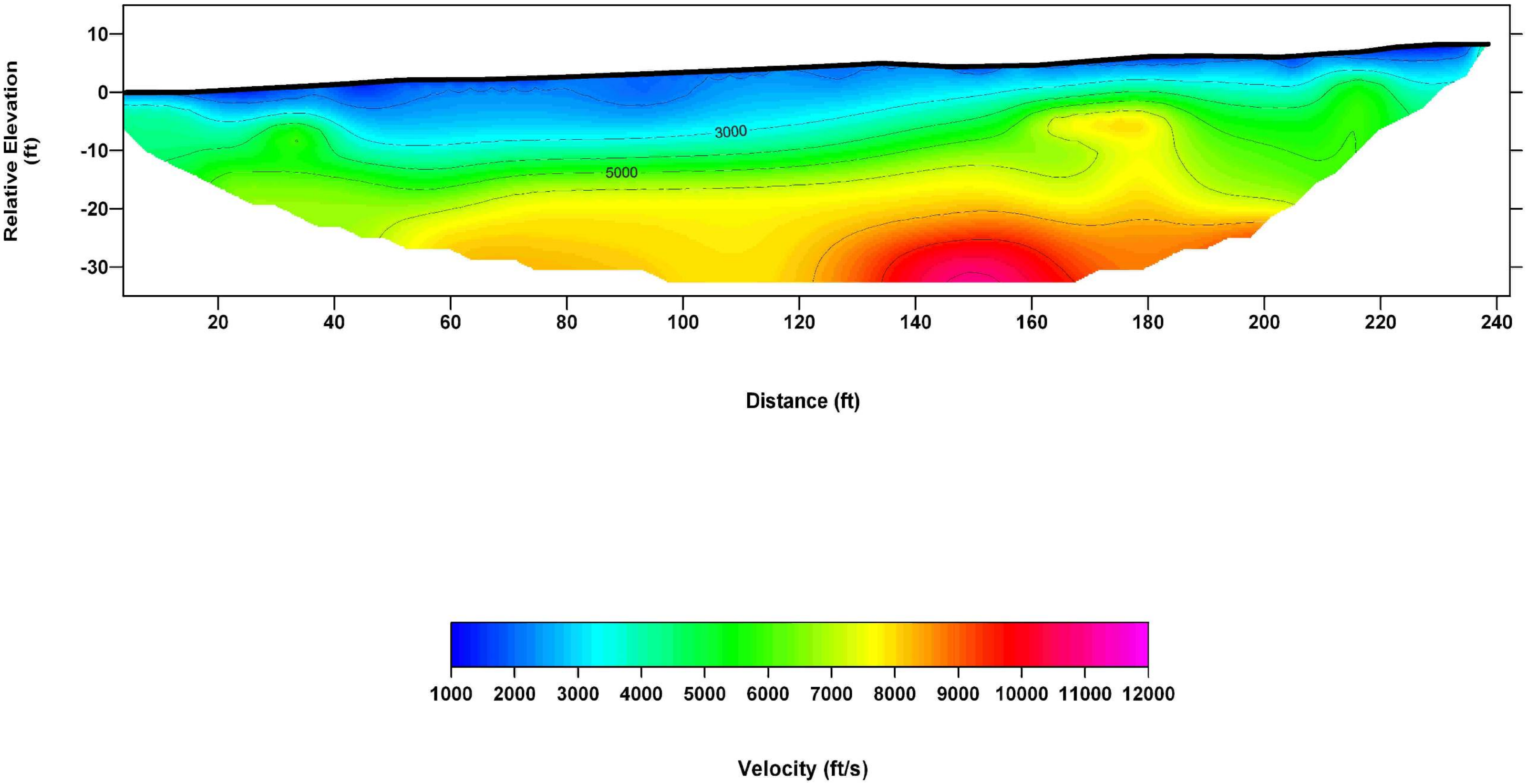
Date: 10/16

SOUTHWEST
GEOPHYSICS INC.

Figure 4a

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-2

NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

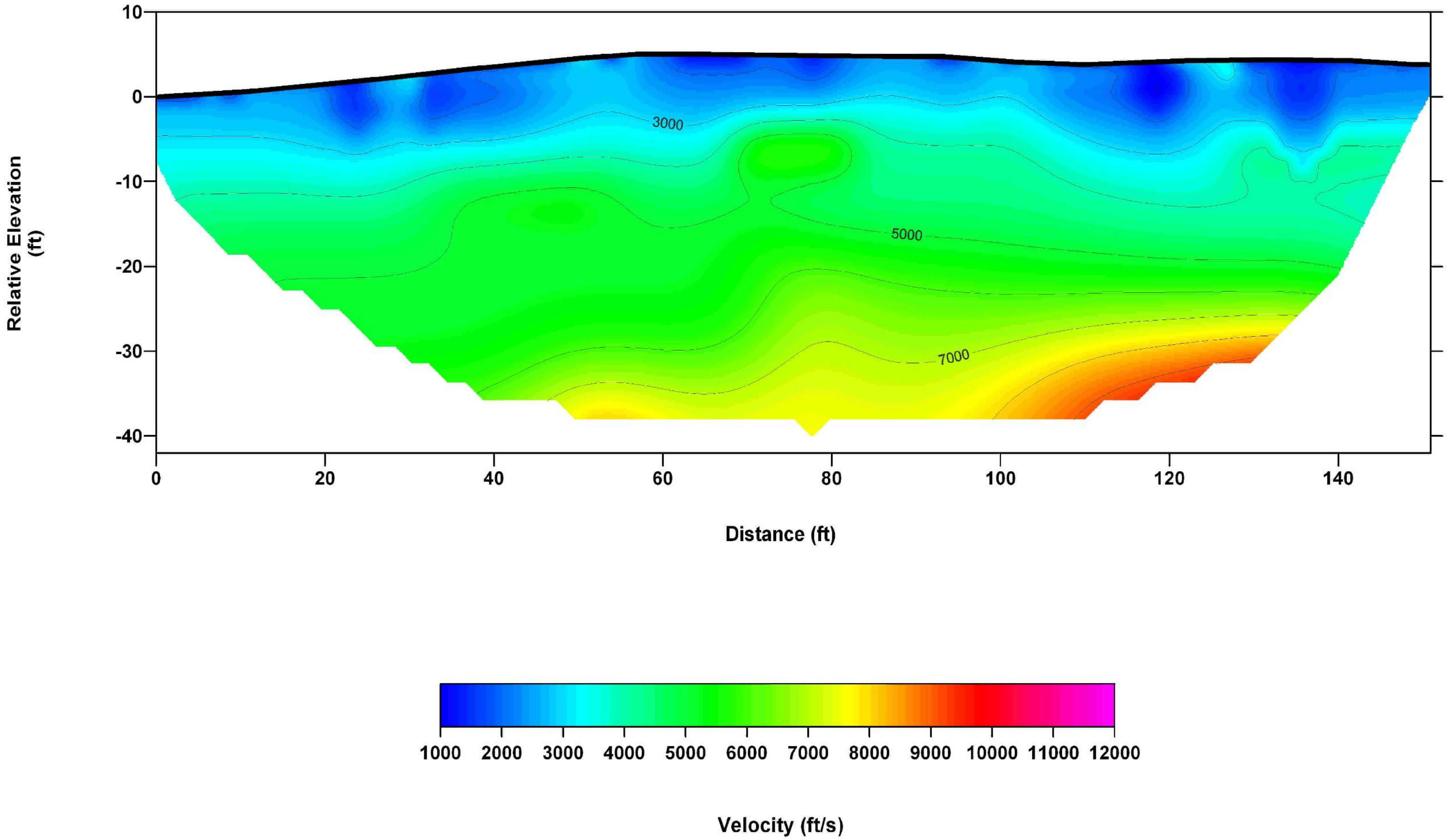
Date: 10/16

SOUTHWEST
GEOPHYSICS INC.

Figure 4b

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-3

NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

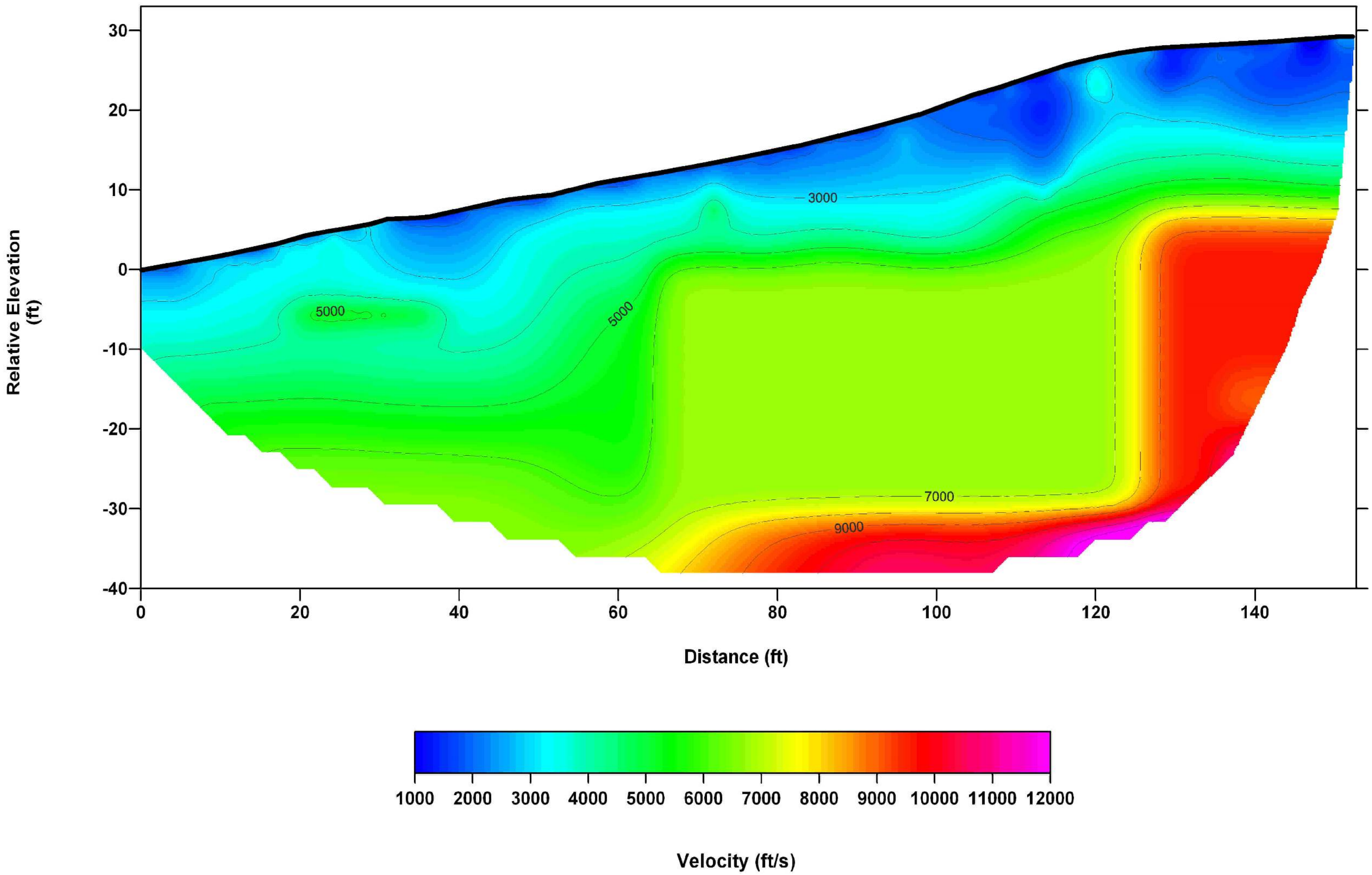
Date: 10/16

SOUTHWEST
GEOPHYSICS INC.

Figure 4c

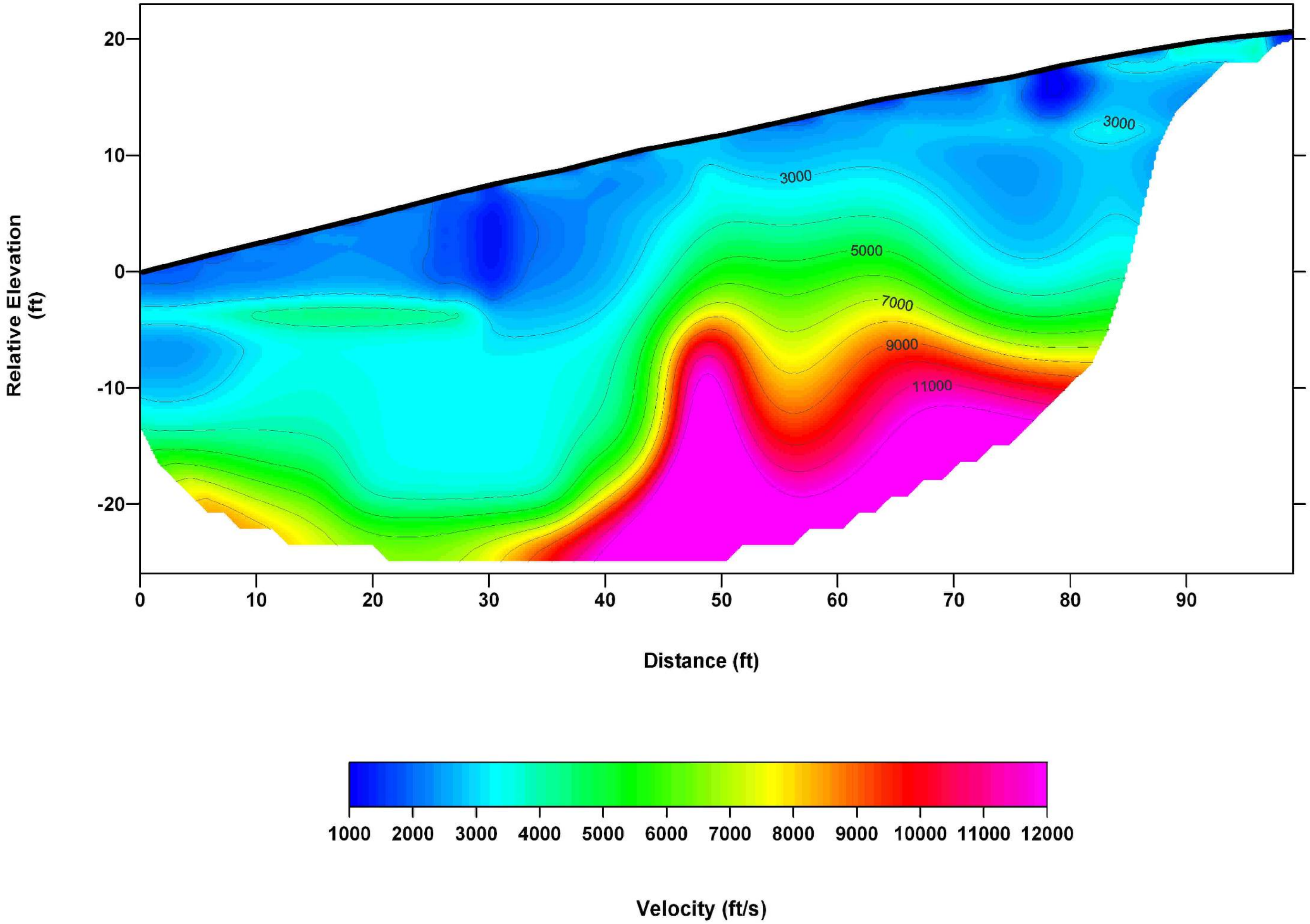
Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE **SL-4**	NCCS Miramar Pipeline San Diego, California	SOUTHWEST GEOPHYSICS INC.	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 116500 / Date: 10/16	Figure 4d	

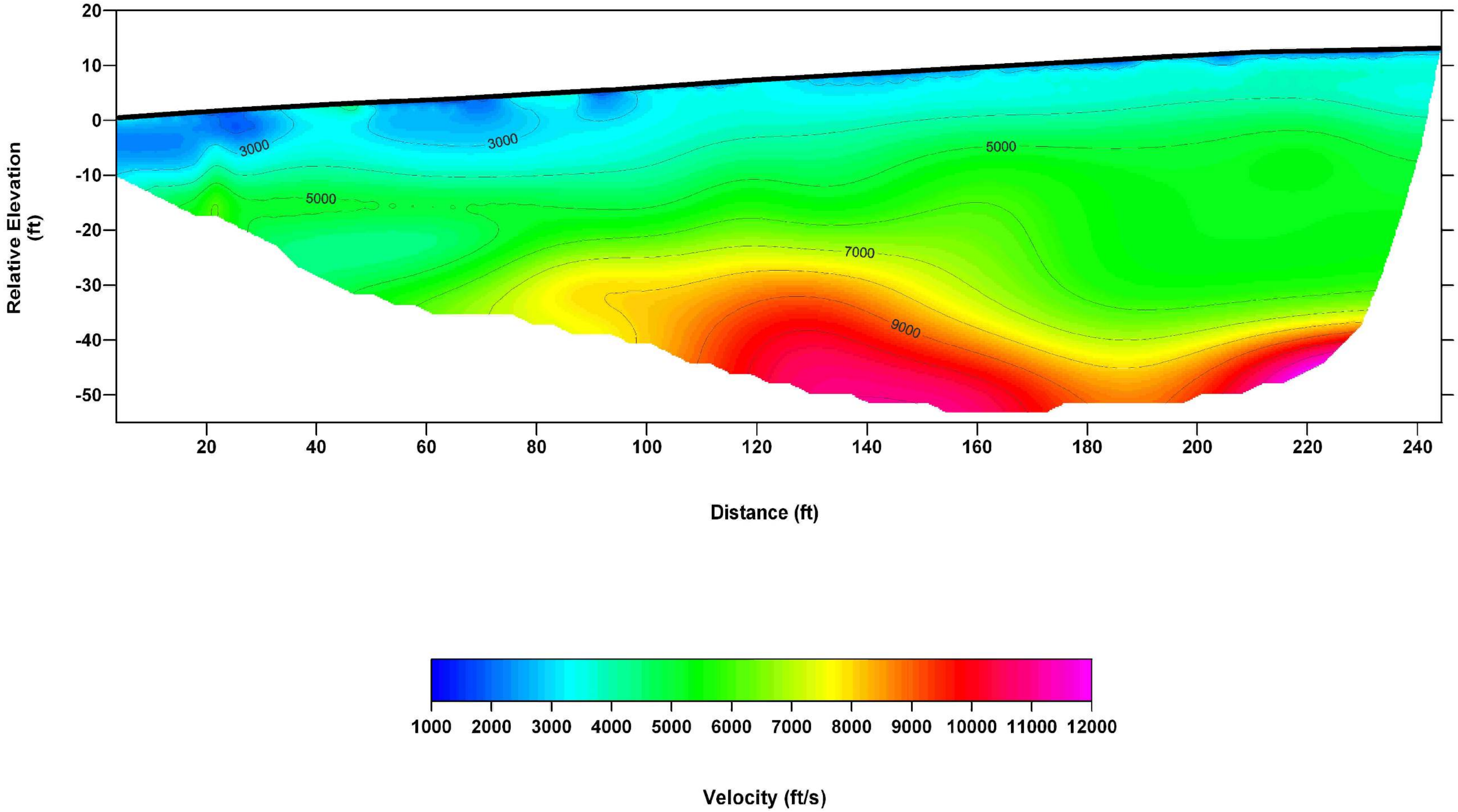
TOMOGRAPHY MODEL



SEISMIC PROFILE SL-5	NCCS Miramar Pipeline San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 4e
	Project No.: 116500 · Date: 10/16	

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-6

NCCS Miramar Pipeline
San Diego, California

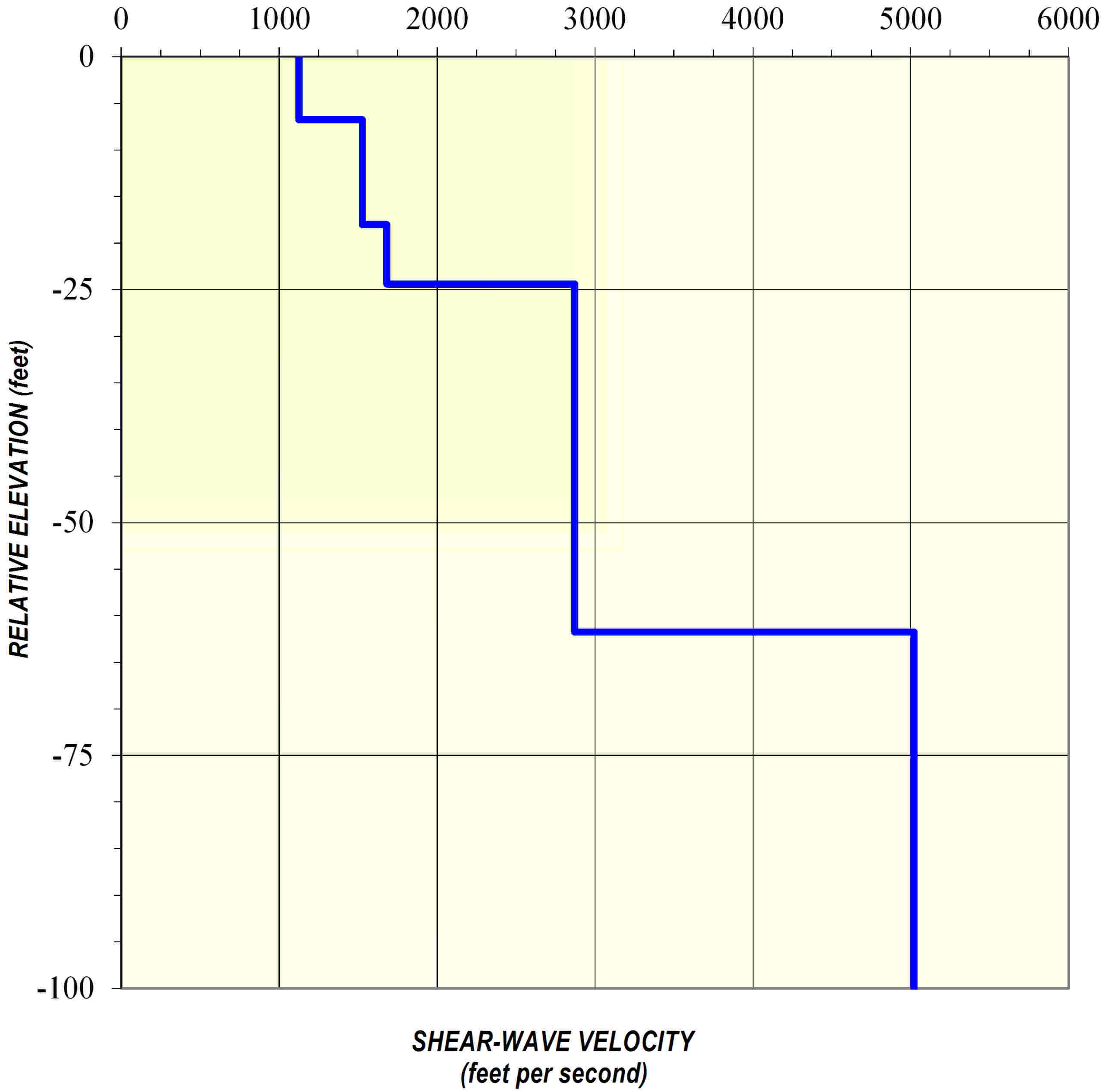
Project No.: 116500

Date: 10/16

SOUTHWEST
GEOPHYSICS INC.

Figure 4f

Note: Contour Interval = 1,000 feet per second



ReMi RESULTS
RL-1

NCCS Miramar Pipeline
San Diego, California

Project No.: 116500

Date: 10/16

SOUTHWEST GEOPHYSICS INC.

Figure 5

SEISMIC SURVEY
by Southwest Geophysics, Inc.
dated February 28, 2017

SEISMIC SURVEY
NORTH CITY CONVEYANCE SYSTEM
SAN DIEGO, CALIFORNIA

PREPARED FOR:
TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 200
San Diego, CA 92123

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

February 28, 2017
Project No. 117056

SOUTHWEST
GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

February 28, 2017
Project No. 117056

Mr. Bob Smillie
TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 200
San Diego, CA 92123

Subject: Seismic Survey
North City Conveyance System
San Diego, California

Dear Mr. Smillie:

In accordance with your authorization, we have performed a seismic survey for the proposed North City Conveyance System project located in San Diego, California. Specifically, our survey consisted of performing seven P-wave refraction traverses and three refraction microtremor (ReMi) profiles at the project site. The purpose of our study was to develop subsurface velocity profiles in the study area. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Hans van de Vrugt

Aaron T. Puente
Project Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

ATP/HV/hv

Distribution: (1) Addressee (electronic)

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/18
STATE OF CALIFORNIA

8057 Raytheon Road, Suite 9 • San Diego • California 92111 • Telephone 858-527-0849 • Fax 858-225-0114

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic survey for the proposed North City Conveyance System project located in San Diego, California (Figure 1). Specifically, our survey consisted of performing seven P-wave refraction traverses and three refraction micro-tremor (ReMi) profiles at the project site. The purpose of our study was to develop subsurface velocity profiles in the study area. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of seven seismic P-wave refraction lines, SL-1 through SL-6, and SL-9.
- Performance of three ReMi profiles, SL-7 through SL-9.
- Compilation and analysis of the data collected.
- Preparation of this illustrated data report presenting our results.

3. SITE DESCRIPTION AND PROJECT DESCRIPTION

The project site generally includes two areas located along Interstate 15 (I-15), just north of Miramar/Pomerado Road in the Miramar area of San Diego (Figure 1). One area is located along the west side of I-15 near the intersec5tion of Via Pasar and Candida Street. The second area is located along the east side of I-15 just west of the southern terminus of Business Park Avenue. Figures 2a, 2b, 3a and 3b depict the general site conditions in the study areas and along the seismic lines.

Based on our discussions with you, it is our understanding your office is conducting a geotechnical evaluation of the site for the proposed pipeline project. The results of our survey will be used in the formulation of design and construction parameters for the project.

4. SURVEY METHODOLOGY AND ANALYSIS

As previously indicated, the primary purpose of our services was to characterize the subsurface conditions at pre-selected locations through the collection of seismic data. The following sections provide an overview of the methodologies used during our study.

4.1. P-wave Refraction Survey

The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves (compression waves) generated at the surface are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component 14-Hz geophones, and recorded with a 24-channel Geometrics Geode seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse. The refraction method requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by buried boulders, fractures, dikes, etc. can result in the misinterpretation of the subsurface conditions.

Seven seismic P-wave traverses, SL-1 through SL-6 and SL-9, were conducted at the site (It should be noted that seismic P-wave refraction traverses were attempted at locations SL-7 and SL-8; however, due to excessive noise from the I-15 freeway the data were not useable). The location of the profiles, which were selected by your office, and the line lengths are depicted on Figures 2a and 2b. Multiple shot points (signal generator locations) were conducted at the ends, midpoint, and intermediate points along the lines. The P-wave signal (shot) was generated using a 16-pound hammer and an aluminum plate.

In general, the seismic P-wave velocity of a material can be correlated to rippability (see Table 1 below), or to some degree "hardness." Table 1 is based on published information from the Caterpillar Performance Handbook (Caterpillar, 2011) as well as our experience with similar materials, and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock quality or rippability.

The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using SeisOpt Pro (Optim, 2008). SeisOpt Pro uses first arrival picks and elevation data to produce subsurface velocity models through a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Table 1 – Rippability Classification

Seismic P-wave Velocity	Rippability
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

4.2. ReMi Survey

The refraction microtremor technique uses recorded surface waves (specifically Rayleigh waves) that are contained in the background noise to develop a shear wave velocity profile of the site. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one dimensional sounding which represents the average condition across the length of the line. Unlike the refraction method, described above, the ReMi method does not require an increase of material velocity with depth. Therefore, low velocity zones (velocity inversions) are detectable with ReMi.

Three 230-foot long ReMi lines, SL-7 through SL-9, were performed at the project. Fifteen records, 30 seconds long were collected with a 24-channel Geometrics Geode seismograph and 4.5-Hz vertical component geophones.

Collected ReMi data were processed using SeisOpt® ReMi™ software (© Optim LLC, 2005), which uses the refraction microtremor method (Louie, 2001). The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional shear-wave velocity model of the site with roughly 85 to 95 percent accuracy.

5. RESULTS

Figures 4a through 4g present the results from the P-wave refraction survey and Figures 5a through 5c present the ReMi results. Based on the velocity models generated from our P-wave analysis it appears that the study areas are underlain by low velocity materials (e.g., colluvium and topsoil) in the very near surface, and bedrock with varying degrees of weathering at depth. Distinct vertical and lateral velocity variations are evident in the models. Moreover, the degree of bedrock weathering and the depth to bedrock appears to be highly variable across the study areas. In addition, remnant boulders appear to be present in the subsurface.

The ReMi and P-wave results from line SL-9 are generally consistent with some slight variations. The variations are likely due to the averaging effect of the ReMi method.

6. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

7. SELECTED REFERENCES

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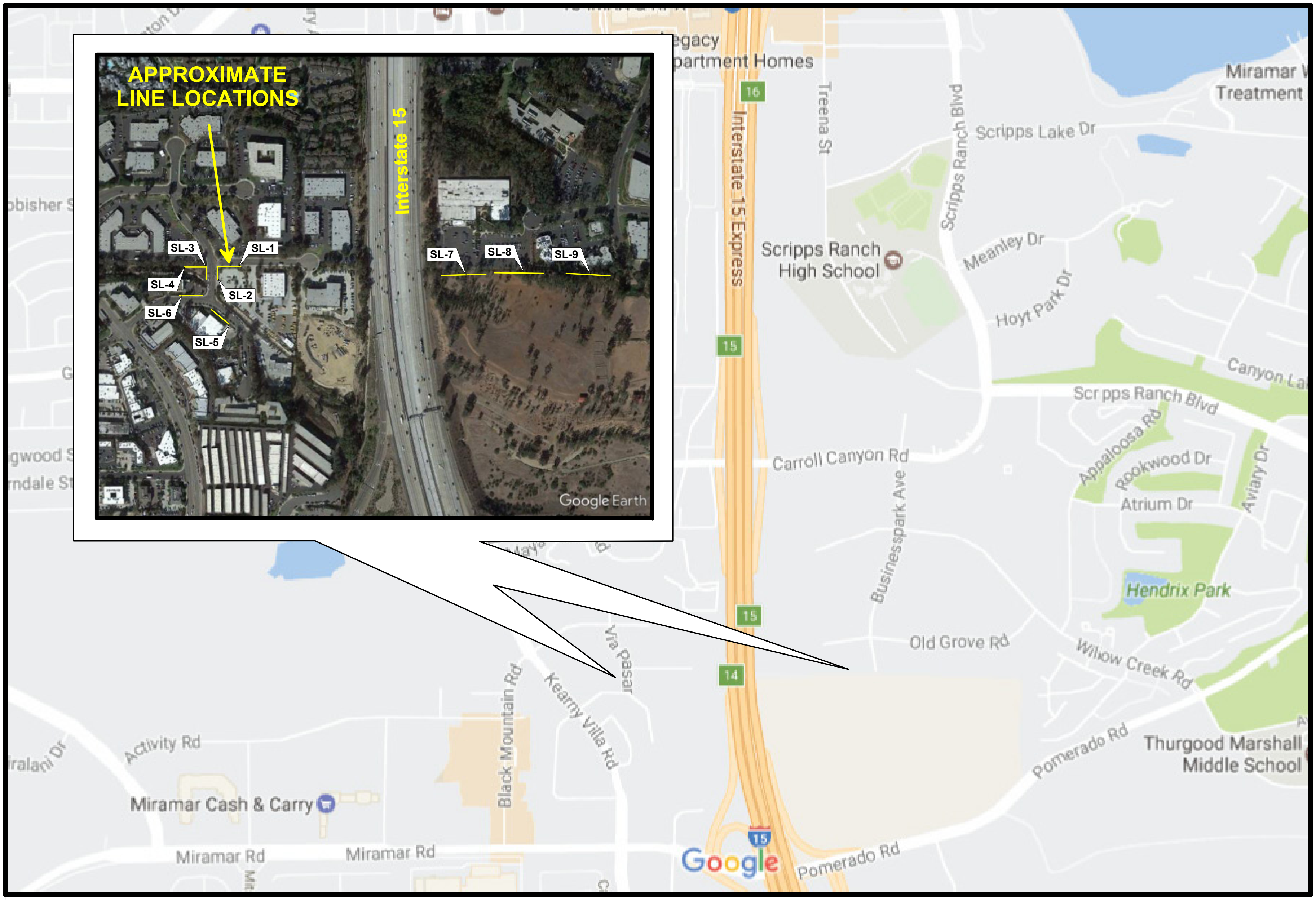


SITE LOCATION MAP

N

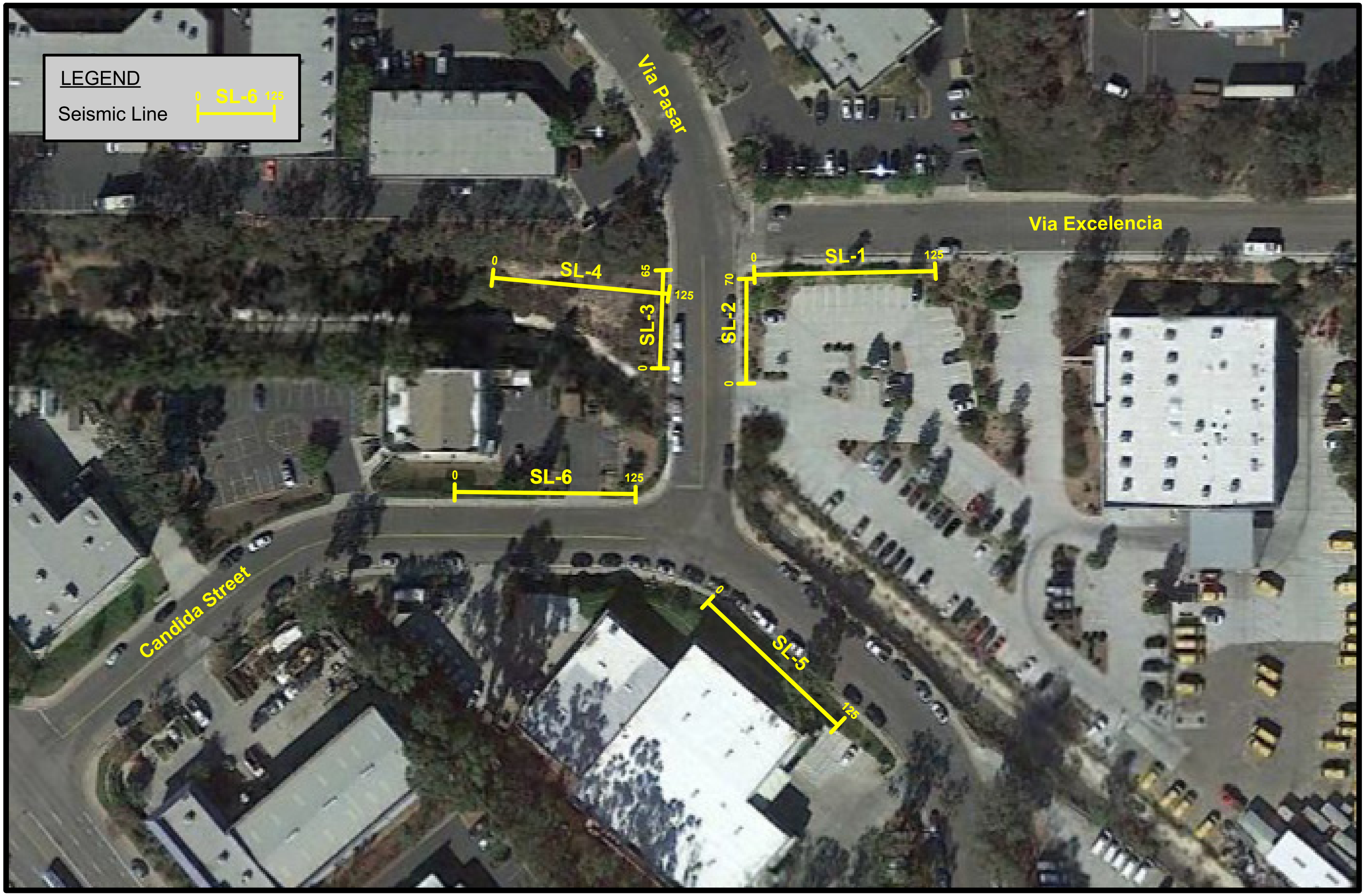
North City Conveyance System
San Diego, California

Project No.: 117056

Date: 02/17

SOUTHWEST GEOPHYSICS INC.

Figure 1



LINE LOCATION MAP

(SL-1 through SL-6)

N

North City Conveyance System
San Diego, California

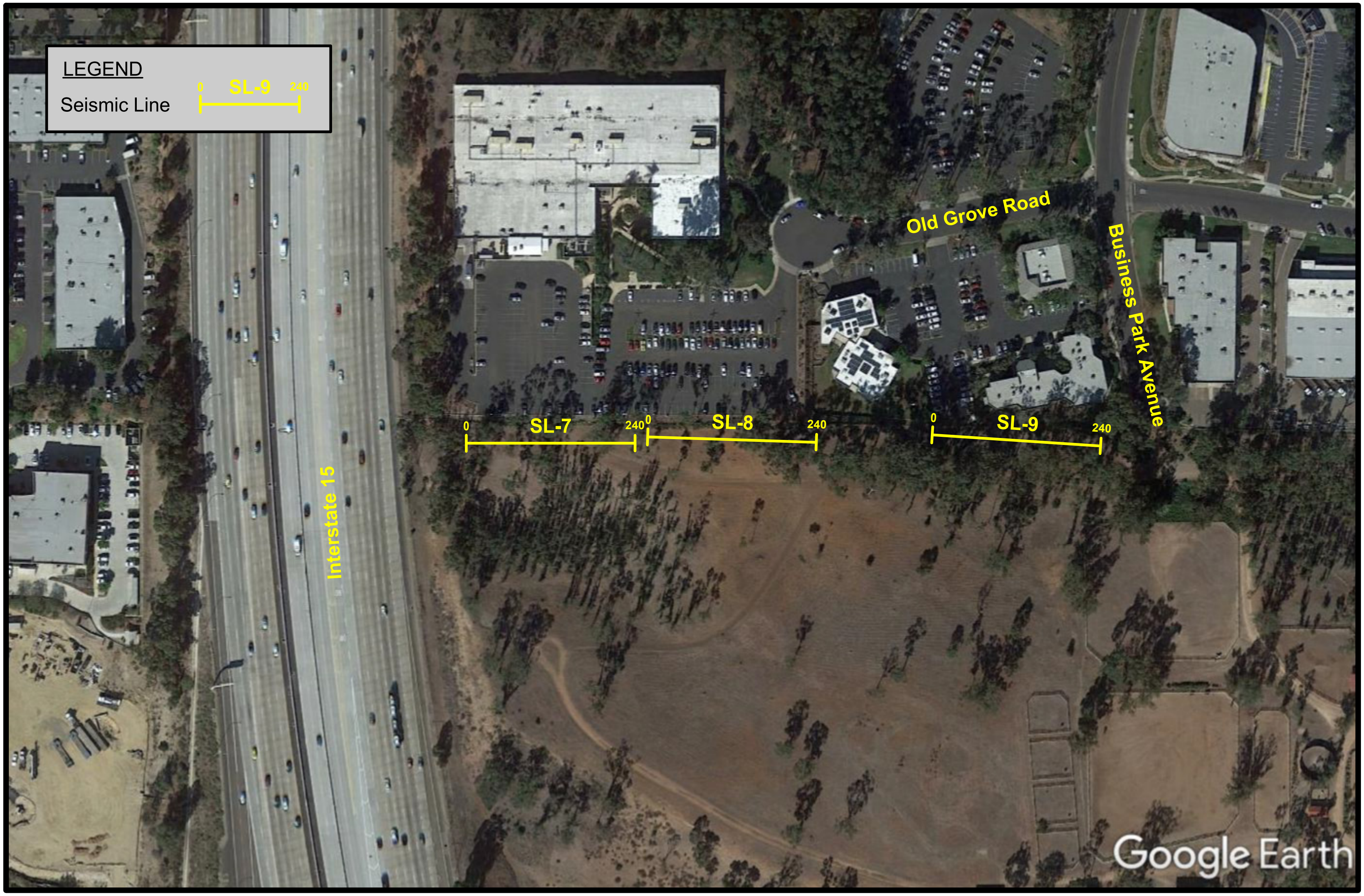
Project No.: 117056

Date: 02/17

SOUTHWEST GEOPHYSICS INC.

Figure 2a

0 200 400
approximate scale in feet



LINE LOCATION MAP
(SL-7 through SL-9)

N

North City Conveyance System
San Diego, California

Project No.: 117056
Date: 02/17

SOUTHWEST GEOPHYSICS INC.

Figure 2b

0 200 400
approximate scale in feet

SITE PHOTOGRAPHS
(SL-1 through SL-6)

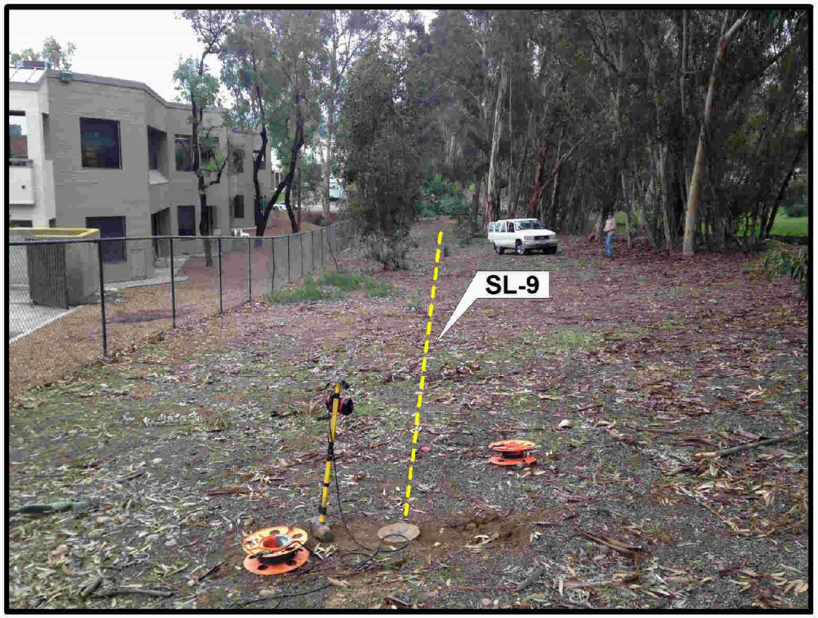
North City Conveyance System
San Diego, California

Project No.: 117056

Date: 02/17

SOUTHWEST
GEOPHYSICS INC.

Figure 3a



SITE PHOTOGRAPHS (SL-7 through SL-9)

North City Conveyance System
San Diego, California

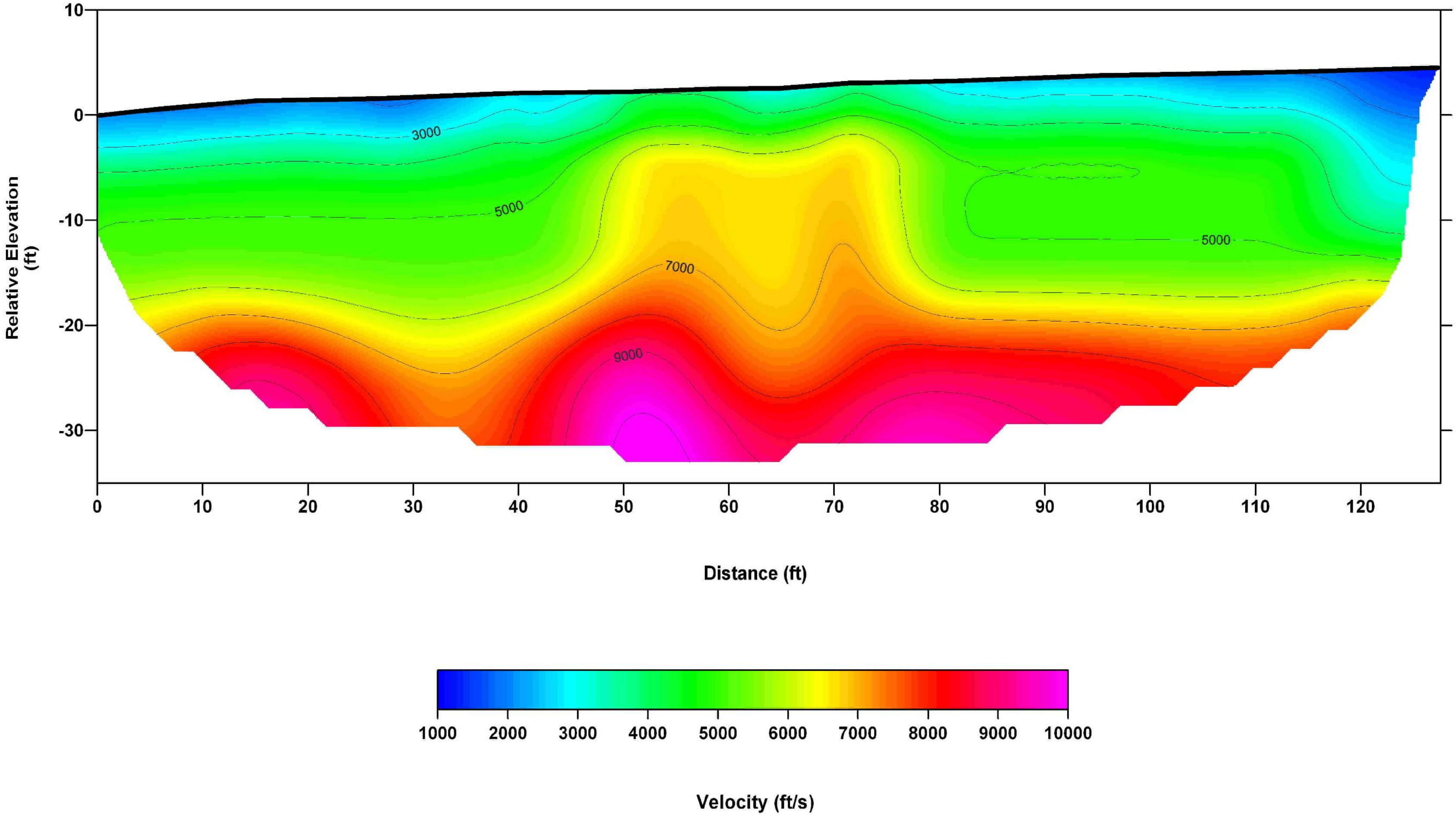
Project No.: 117056

Date: 02/17

SOUTHWEST GEOPHYSICS INC.

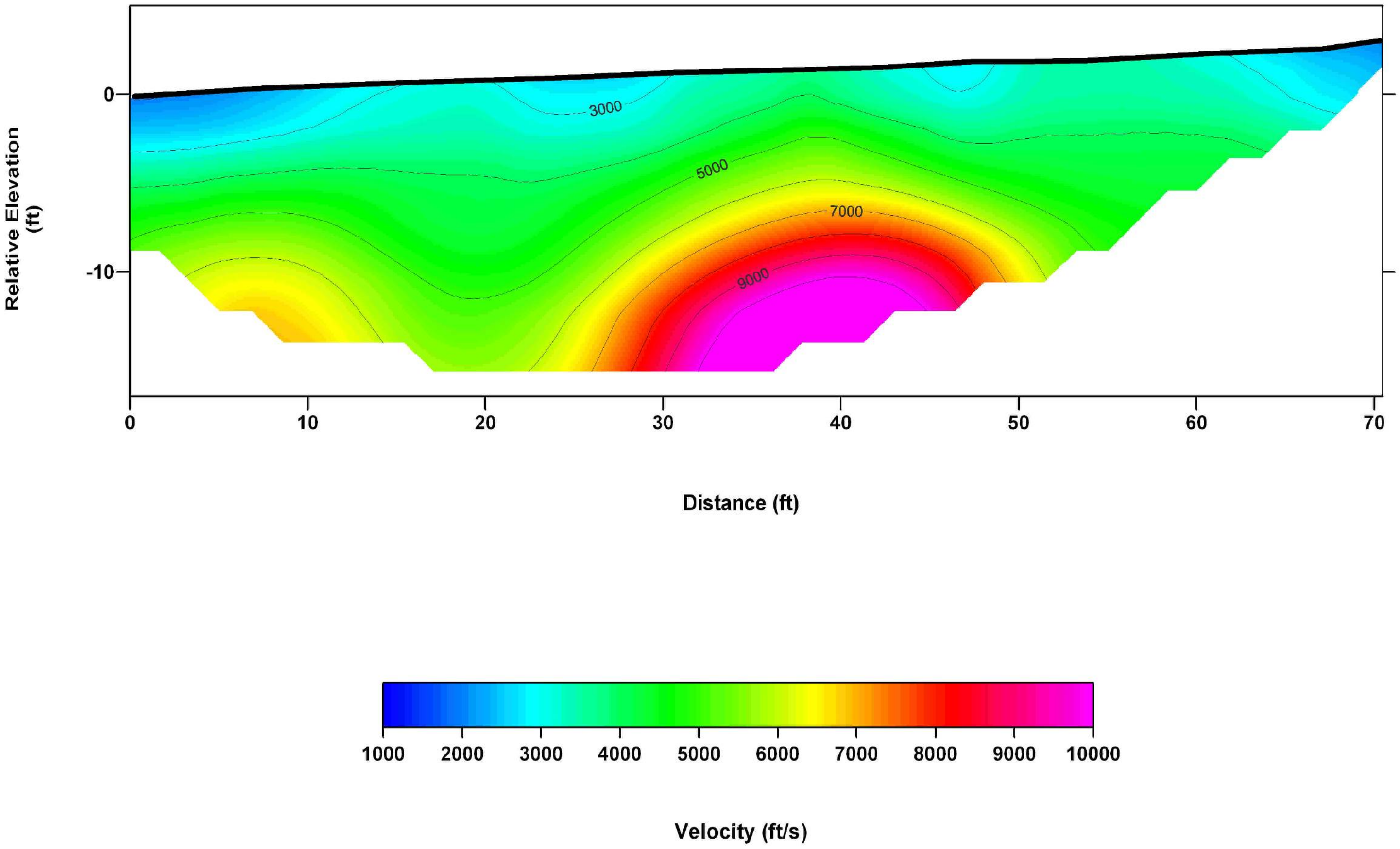
Figure 3b

TOMOGRAPHY MODEL



SEISMIC PROFILE **SL-1**	North City Conveyance System San Diego, California	SOUTHWEST GEOPHYSICS INC. Figure 4a	**Note: Contour Interval = 1,000 feet per second**
	Project No.: 117056	Date: 02/17	

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-2

North City Conveyance System
San Diego, California

Project No.: 117056

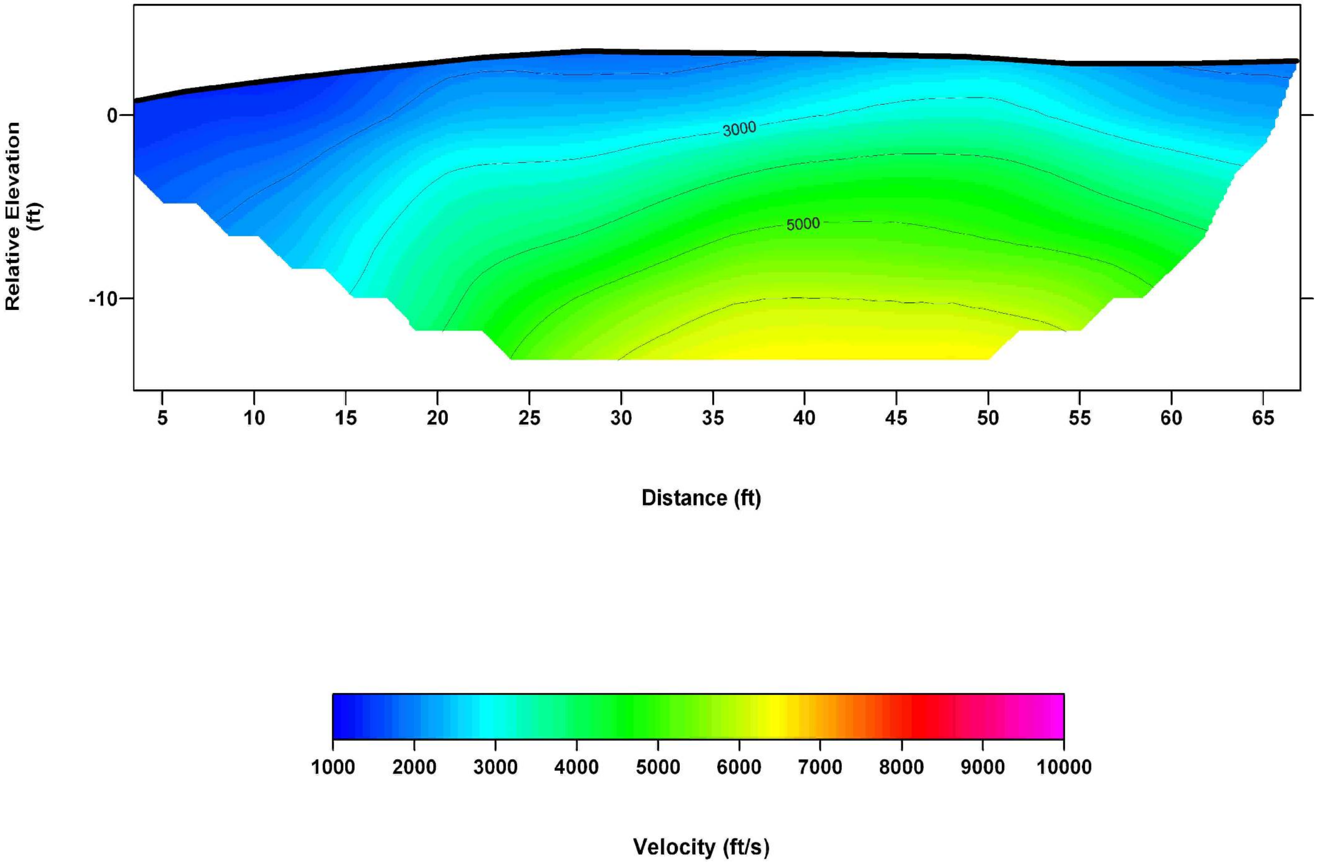
Date: 02/17

SOUTHWEST
GEOPHYSICS INC.

Figure 4b

Note: Contour Interval = 1,000 feet per second

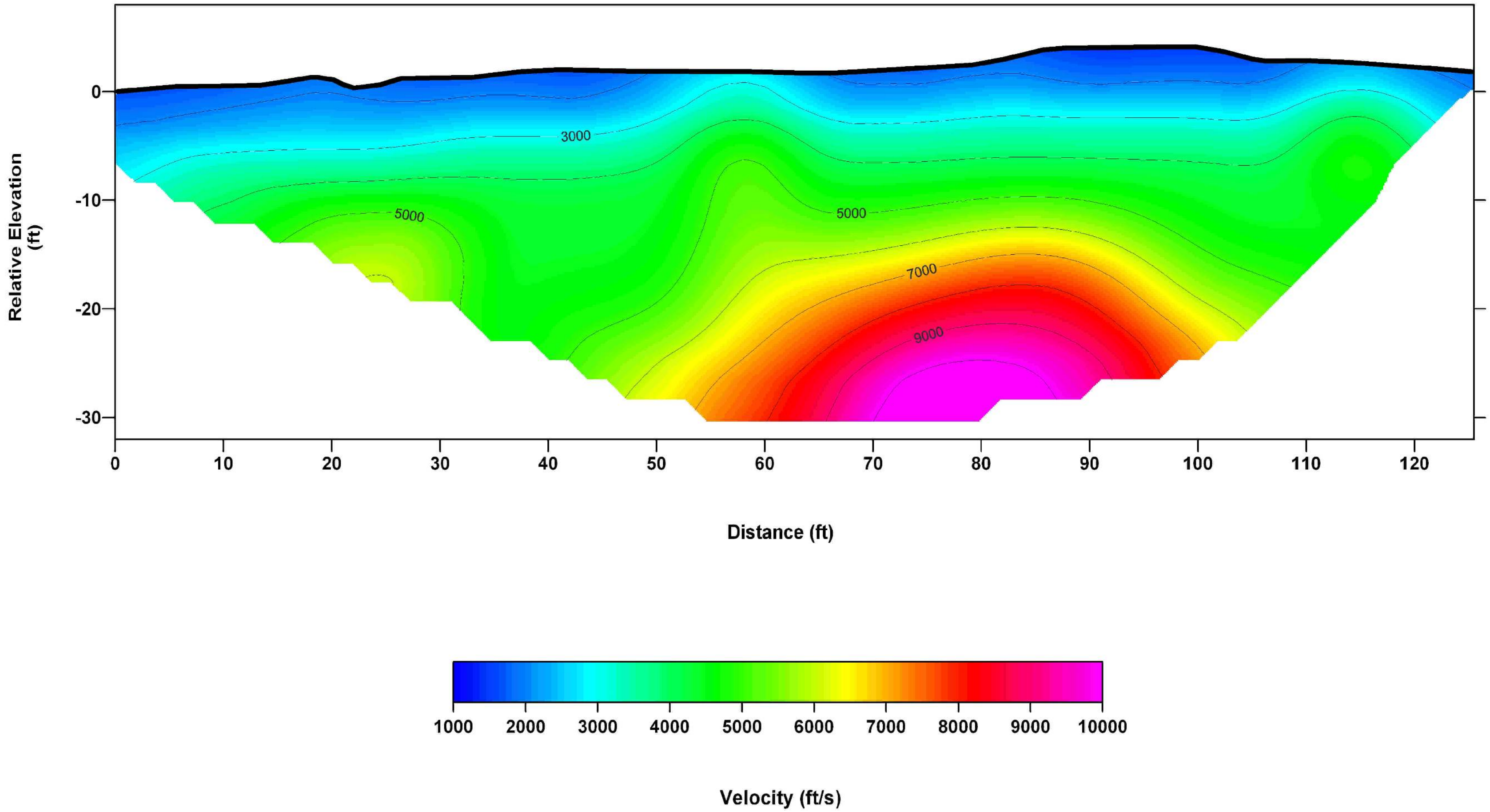
TOMOGRAPHY MODEL



SEISMIC PROFILE **SL-3**	North City Conveyance System San Diego, California Project No.: 117056 Date: 02/17	SOUTHWEST GEOPHYSICS INC. Figure 4c

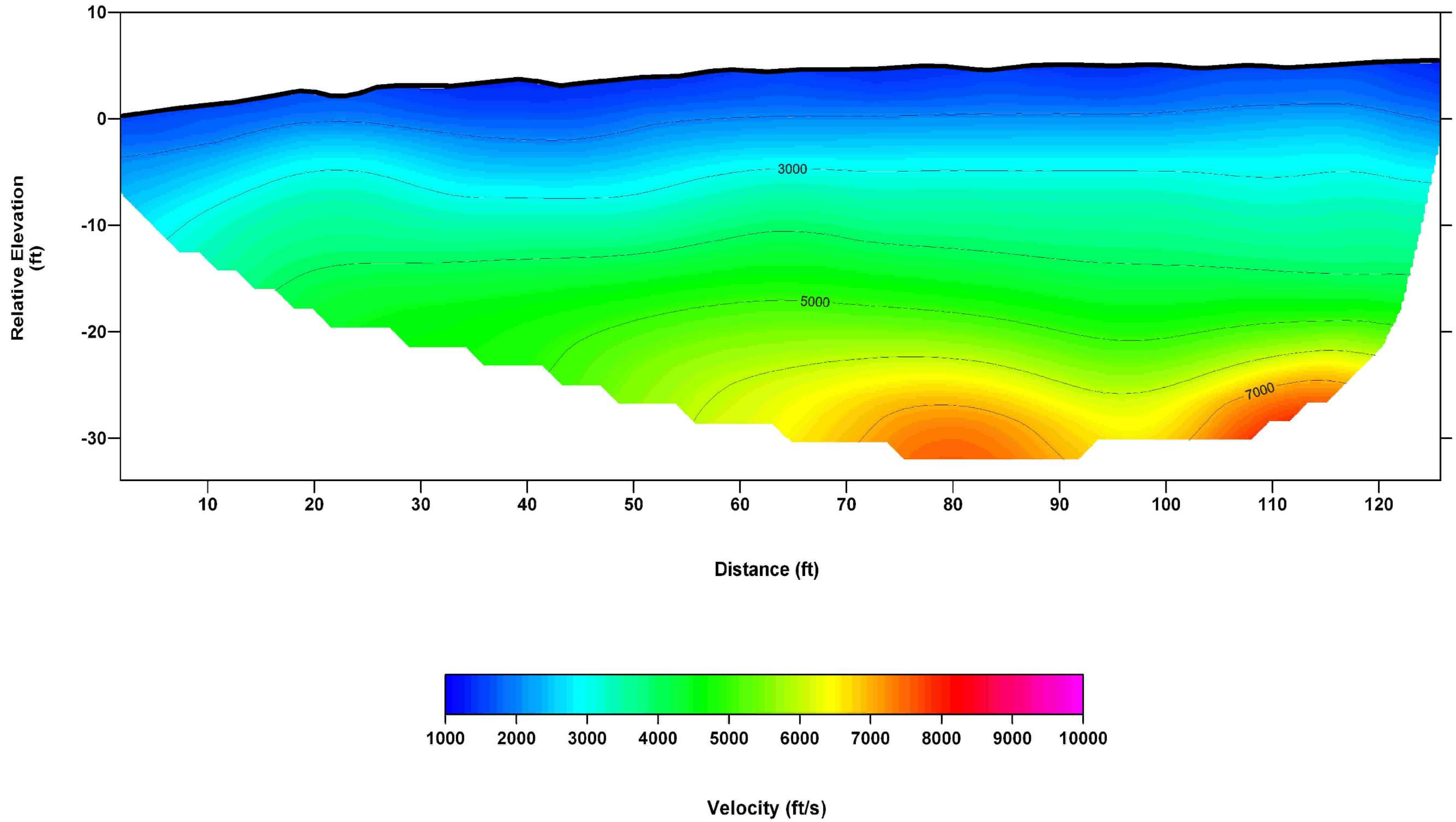
Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE **SL-4**	North City Conveyance System San Diego, California Project No.: 117056 Date: 02/17	SOUTHWEST GEOPHYSICS INC. Figure 4d	**Note: Contour Interval = 1,000 feet per second**

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-5

North City Conveyance System
San Diego, California

Project No.: 117056

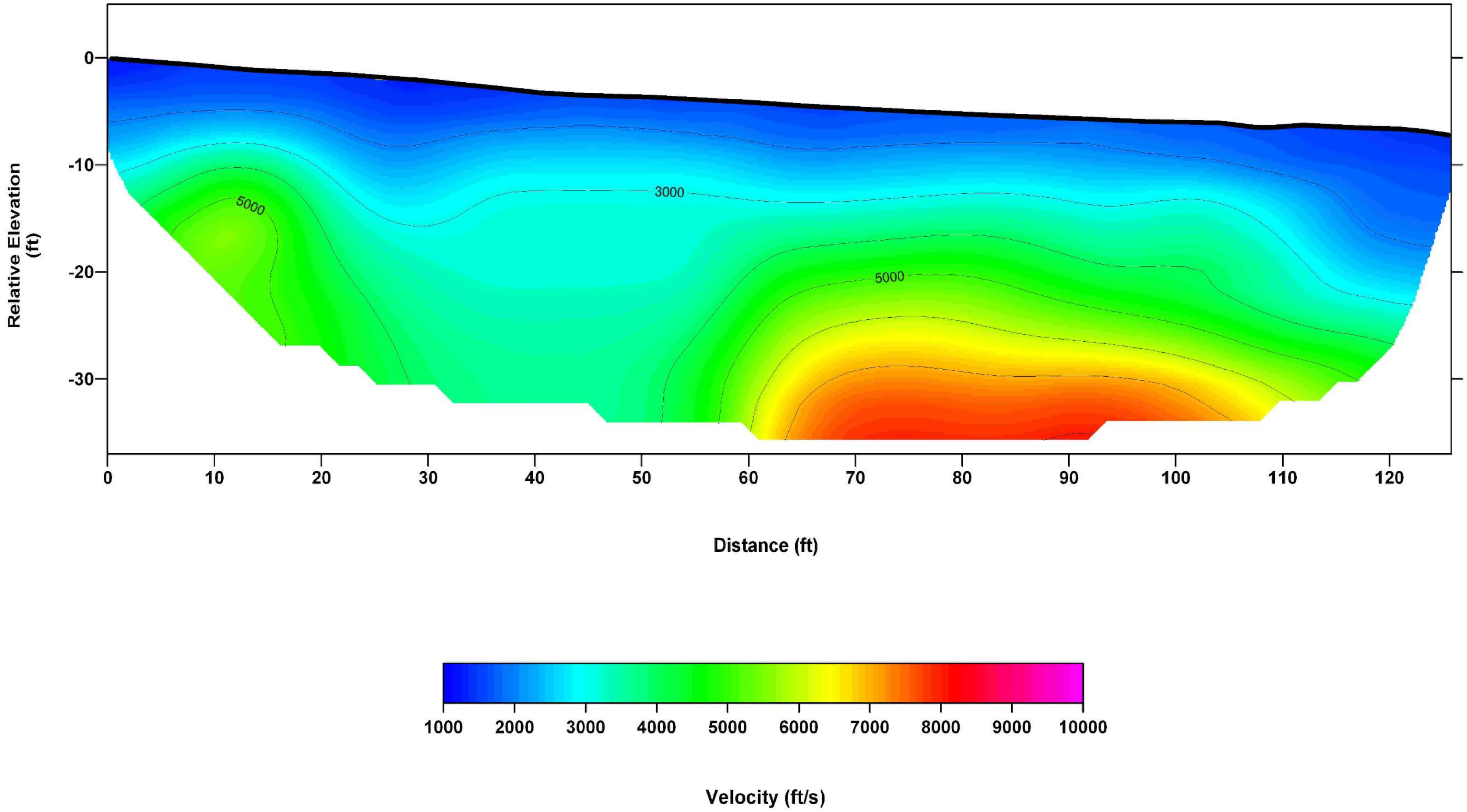
Date: 02/17

SOUTHWEST GEOPHYSICS INC.

Figure 4e

Note: Contour Interval = 1,000 feet per second

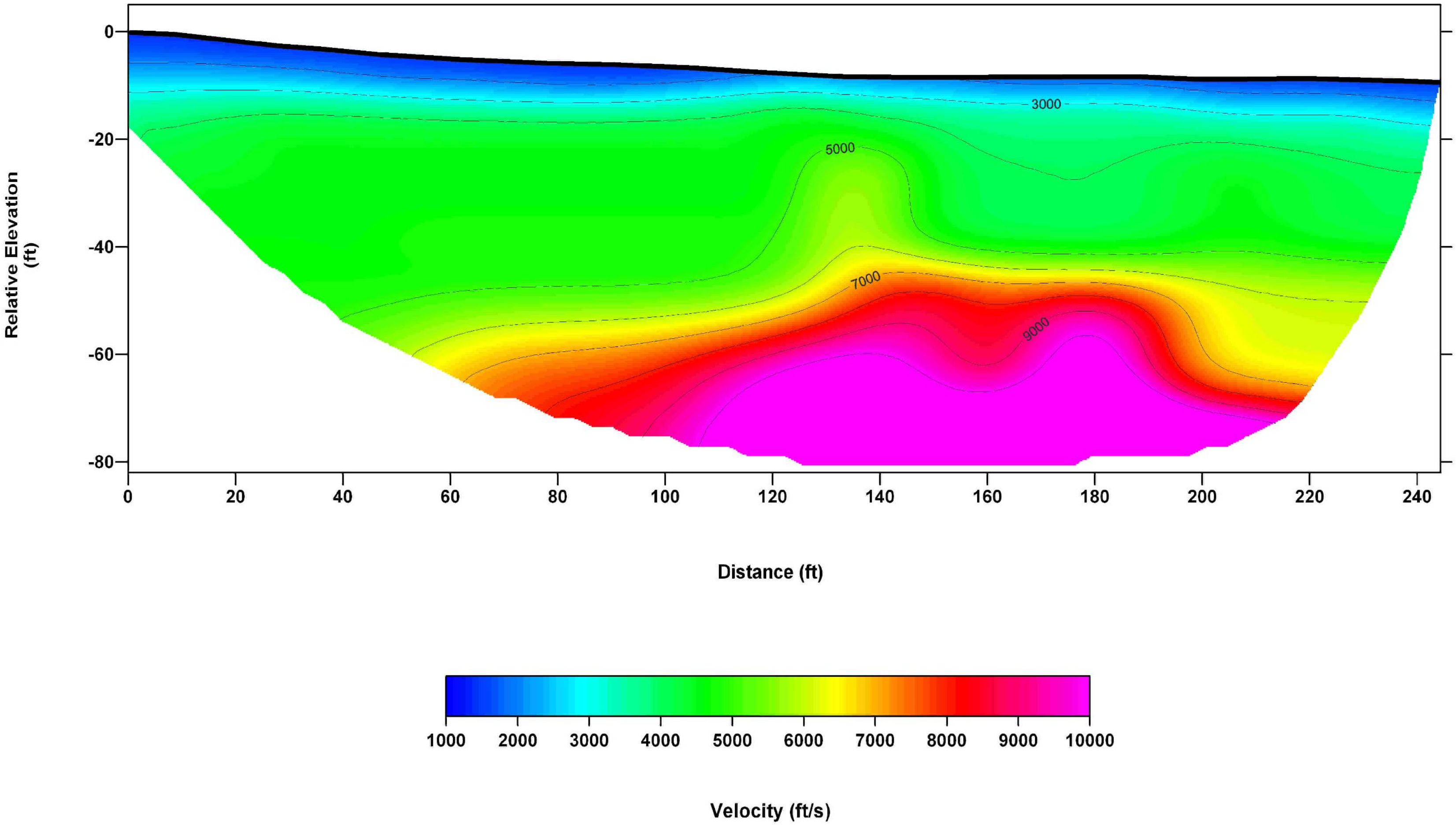
TOMOGRAPHY MODEL



SEISMIC PROFILE **SL-6**	North City Conveyance System San Diego, California Project No.: 117056 Date: 02/17	SOUTHWEST GEOPHYSICS INC. Figure 4f

Note: Contour Interval = 1,000 feet per second

TOMOGRAPHY MODEL



SEISMIC PROFILE
SL-9

North City Conveyance System
San Diego, California

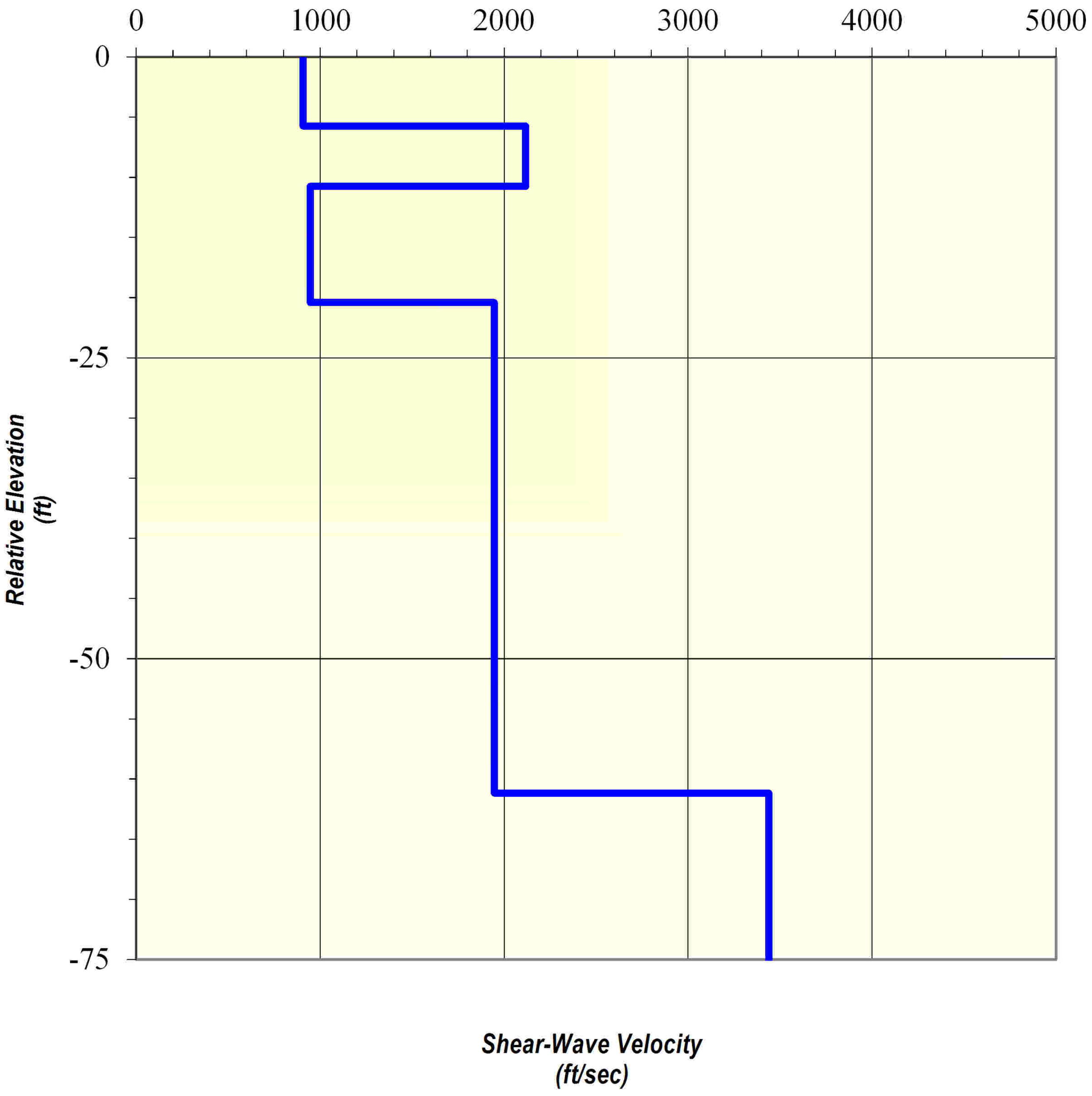
Project No.: 117056

Date: 02/17

SOUTHWEST
GEOPHYSICS INC.

Figure 4g

Note: Contour Interval = 1,000 feet per second



ReMi RESULTS
SL-7

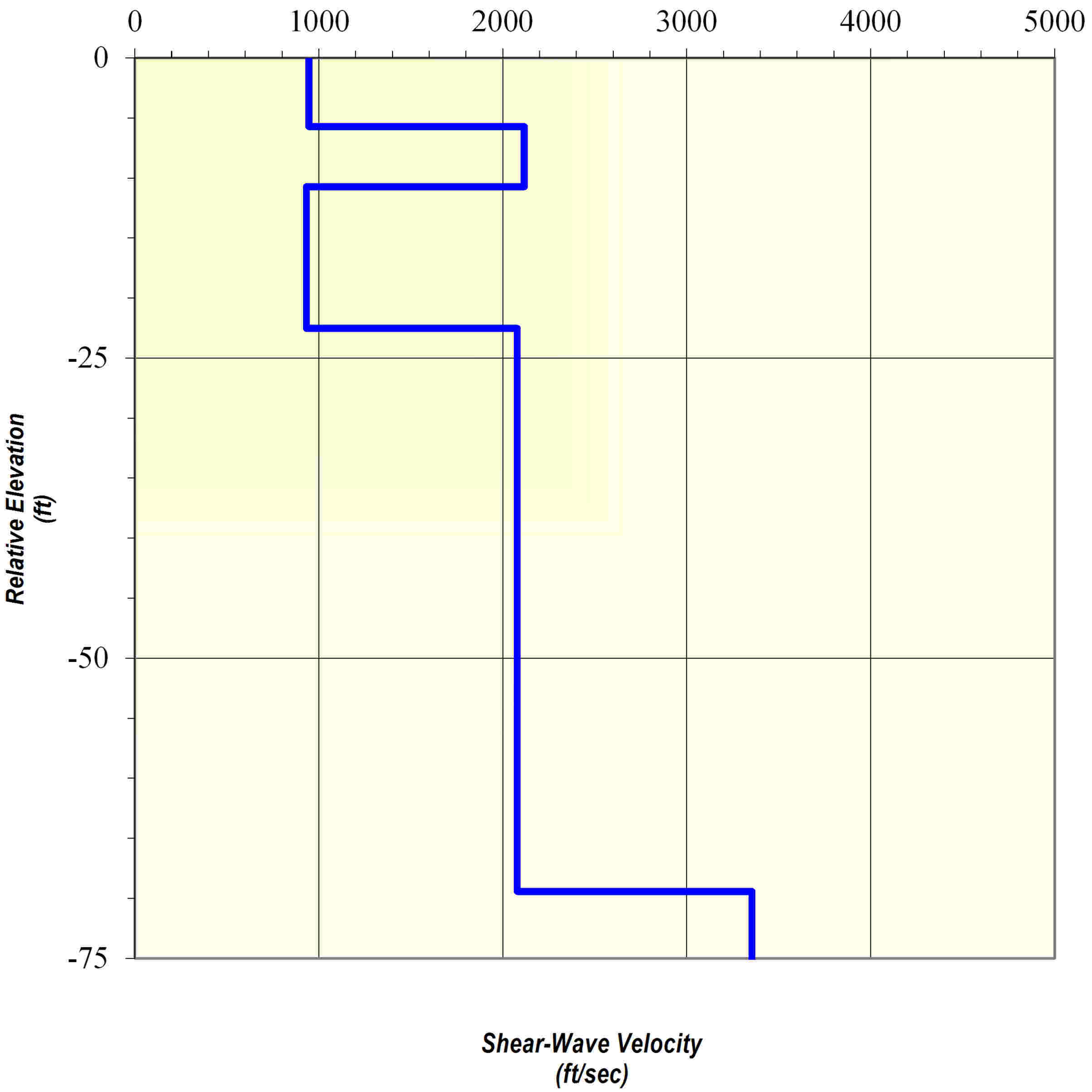
North City Conveyance System
San Diego, California

Project No.: 117056

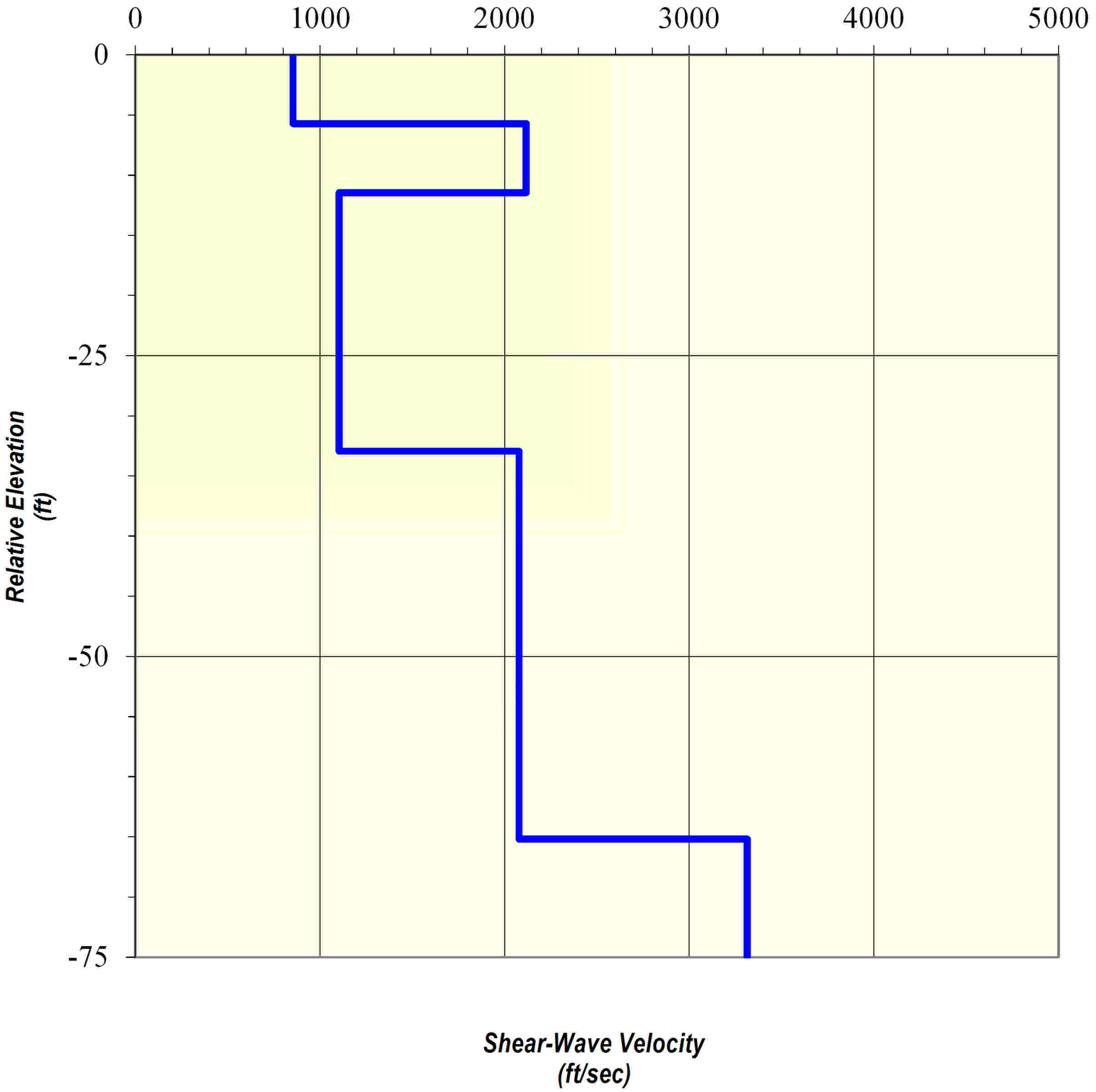
Date: 02/17

SOUTHWEST GEOPHYSICS INC.

Figure 5a



ReMi RESULTS **SL-8**	North City Conveyance System San Diego, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 117056	Date: 02/17
		Figure 5b



ReMi RESULTS
SL-9

North City Conveyance System
San Diego, California

Project No.: 117056

Date: 02/17

SOUTHWEST
GEOPHYSICS INC.

Figure 5c

APPENDIX C

LABORATORY TEST RESULTS

Table 1 - Laboratory Tests on Soil Samples

TerraCosta Consulting Group
North City Conveyance System
HDR Lab #17-0038LAB
20-Feb-17

Sample ID			B1-5 @ 18-19.5'	B1-2 @ 8-9'	TB-1a-5 @ 20-21'	B3-6 @ 23'	B4-3 @ 15'
Resistivity		**Units**					
as-received		ohm-cm	2,960	1,520	5,600	3,840	4,800
saturated		ohm-cm	600	440	3,440	880	2,240
pH			6.3	6.0	3.9	3.3	3.6
Electrical Conductivity		mS/cm	0.58	1.25	0.04	0.34	0.06
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	21	37	17	12	16
magnesium	Mg^{2+}	mg/kg	13	42	11	6.9	12
sodium	Na^{1+}	mg/kg	581	1,230	53	292	68
potassium	K^{1+}	mg/kg	17	24	4.0	24	5.4
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	67	24	55	18	27
fluoride	F^{1-}	mg/kg	2.9	13	4.7	ND	1.4
chloride	Cl^{1-}	mg/kg	735	1720	15	373	24
sulfate	SO_4^{2-}	mg/kg	152	293	17	96	48
phosphate	PO_4^{3-}	mg/kg	ND	ND	1.0	ND	ND
Other Tests							
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	ND	ND	ND	ND	ND
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na
% moisture	H_2O	%	19.1%	14.0%	12.6%	9.3%	12.1%

Resistivity per ASTM G-187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per AWWA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

Table 1 - Laboratory Tests on Soil Samples

TerraCosta Consulting Group
North City Conveyance System
HDR Lab #17-0038LAB
20-Feb-17

Sample ID			B5-3 @ 12-14'	B6-2 @ 10-12'	B21-3 @ 10'	B7-5 @ 16-18'	B8-2 @ 10-12'
Resistivity		**Units**					
as-received		ohm-cm	44,000	3,720	15,200	24,800	13,200
saturated		ohm-cm	2,520	2,280	2,160	3,400	3,360
pH			5.8	6.0	3.6	5.0	4.6
Electrical Conductivity		mS/cm	0.11	0.18	0.15	0.04	0.04
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	17	32	14	10	12
magnesium	Mg^{2+}	mg/kg	7.2	18	7.3	17	7.2
sodium	Na^{1+}	mg/kg	115	122	143	41	44
potassium	K^{1+}	mg/kg	13	23	9.3	10	4.8
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	107	85	37	55	34
fluoride	F^{1-}	mg/kg	3.6	ND	ND	ND	ND
chloride	Cl^{1-}	mg/kg	18	15	39	8.1	5.7
sulfate	SO_4^{2-}	mg/kg	92	205	203	40	45
phosphate	PO_4^{3-}	mg/kg	ND	1.0	1.4	ND	ND
Other Tests							
ammonium	NH_4^{1+}	mg/kg	ND	9.1	ND	1.6	ND
nitrate	NO_3^{1-}	mg/kg	ND	ND	ND	ND	ND
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na
% moisture	H_2O	%	3.1%	10.6%	5.3%	6.9%	9.8%

Resistivity per ASTM G-187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per AWWA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

Table 1 - Laboratory Tests on Soil Samples

TerraCosta Consulting Group
North City Conveyance System
HDR Lab #17-0038LAB
20-Feb-17

Sample ID			B9-3 @ 11-13'	B10-2 @ 10'	B11-2 @ 8-10'	B12-3 @ 10-11'	B13-2 @ 7-8'
Resistivity		**Units**					
as-received		ohm-cm	1,080	2,400	28,400	1,880	1,320
saturated		ohm-cm	480	1,840	2,000	960	960
pH			3.7	5.5	4.3	4.1	3.1
Electrical Conductivity		mS/cm	0.30	0.21	0.11	0.17	0.06
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	9.2	26	13	18	8.5
magnesium	Mg^{2+}	mg/kg	19	16	10	15	31
sodium	Na^{1+}	mg/kg	299	159	113	167	69
potassium	K^{1+}	mg/kg	11	24	10	12	6.4
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	27	64	34	43	34
fluoride	F^{1-}	mg/kg	ND	ND	ND	1.0	0.7
chloride	Cl^{1-}	mg/kg	297	30	54	81	7.2
sulfate	SO_4^{2-}	mg/kg	165	257	101	181	72
phosphate	PO_4^{3-}	mg/kg	ND	4.5	4.8	4.9	5.2
Other Tests							
ammonium	NH_4^{1+}	mg/kg	3.2	11	2.5	2.6	1.4
nitrate	NO_3^{1-}	mg/kg	ND	0.7	ND	ND	ND
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na
% moisture	H_2O	%	13.5%	16.4%	8.7%	12.2%	17.6%

Resistivity per ASTM G-187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per AWWA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

Table 1 - Laboratory Tests on Soil Samples

TerraCosta Consulting Group
North City Conveyance System
HDR Lab #17-0038LAB
20-Feb-17

Sample ID			B14-3 @ 13-14'	TB-2a-7 @ 32'	B16-4 @ 15-16.5'	B17-2 @ 10-11.5'	B18-2 @ 10-11.5'
Resistivity		**Units**					
as-received		ohm-cm	56,000	116,000	8,400	39,200	34,000
saturated		ohm-cm	4,800	1,520	2,600	3,600	2,680
pH			4.0	5.1	4.5	4.7	5.9
Electrical Conductivity		mS/cm	0.07	0.32	0.08	0.09	0.09
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	12	11	31	12	12
magnesium	Mg^{2+}	mg/kg	7.6	24	34	6.4	5.3
sodium	Na^{1+}	mg/kg	67	309	104	86	91
potassium	K^{1+}	mg/kg	11	32	13	12	4.5
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	46	92	64	49	85
fluoride	F^{1-}	mg/kg	ND	9.8	2.6	3.6	6.0
chloride	Cl^{1-}	mg/kg	24	290	41	25	40
sulfate	SO_4^{2-}	mg/kg	64	154	54	102	91
phosphate	PO_4^{3-}	mg/kg	ND	ND	7.0	ND	5.7
Other Tests							
ammonium	NH_4^{1+}	mg/kg	1.9	2.9	2.0	ND	ND
nitrate	NO_3^{1-}	mg/kg	2.0	ND	ND	ND	ND
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na
% moisture	H_2O	%	4.4%	2.6%	6.7%	3.8%	3.7%

Resistivity per ASTM G-187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per AWWA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

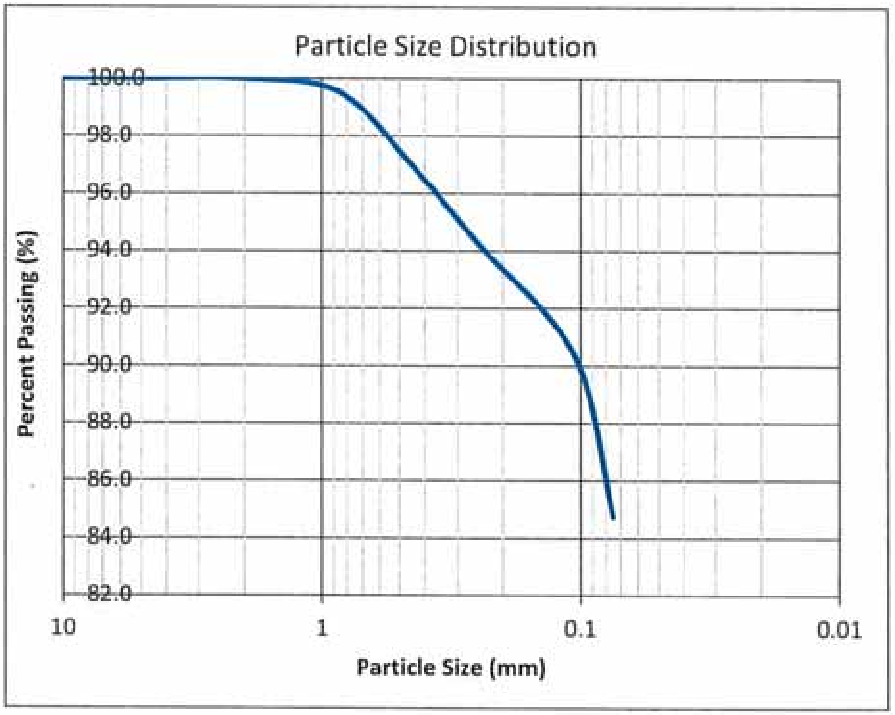
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B1-5 @ 18-19.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.5
No. 40	0.425	96.7
No. 60	0.25	94.3
No. 140	0.106	90.4
No. 200	0.075	84.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	3.3
Fine Sand (<No. 40-No. 200):	12.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	15.2	84.8

Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B1-2 @ 8-9'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.8
No. 40	0.425	99.4
No. 60	0.25	98.9
No. 140	0.106	90.9
No. 200	0.075	80.1
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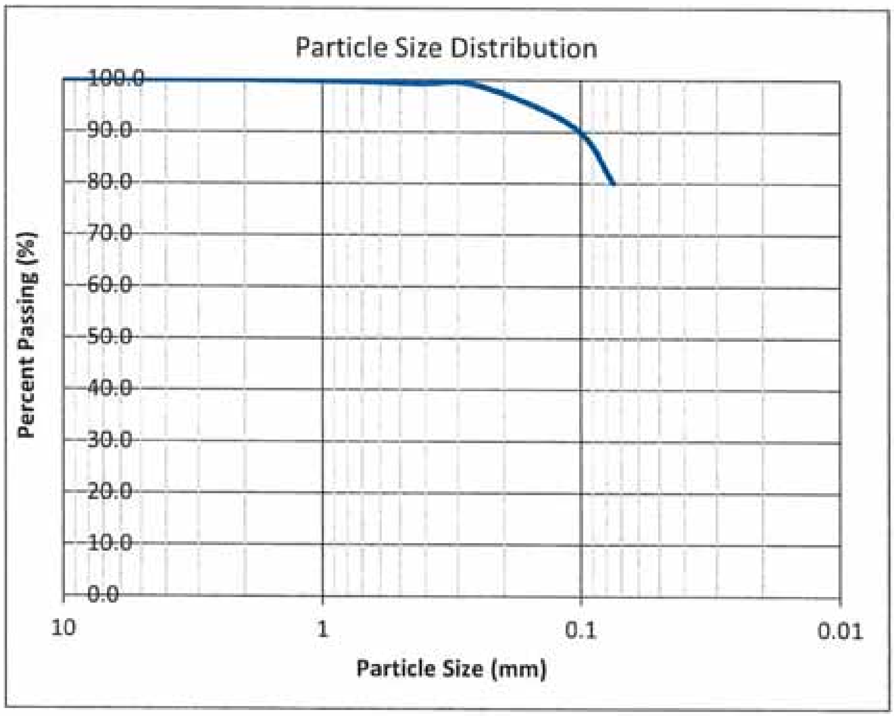
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	0.6
Fine Sand (<No. 40-No. 200):	19.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	19.9	80.1

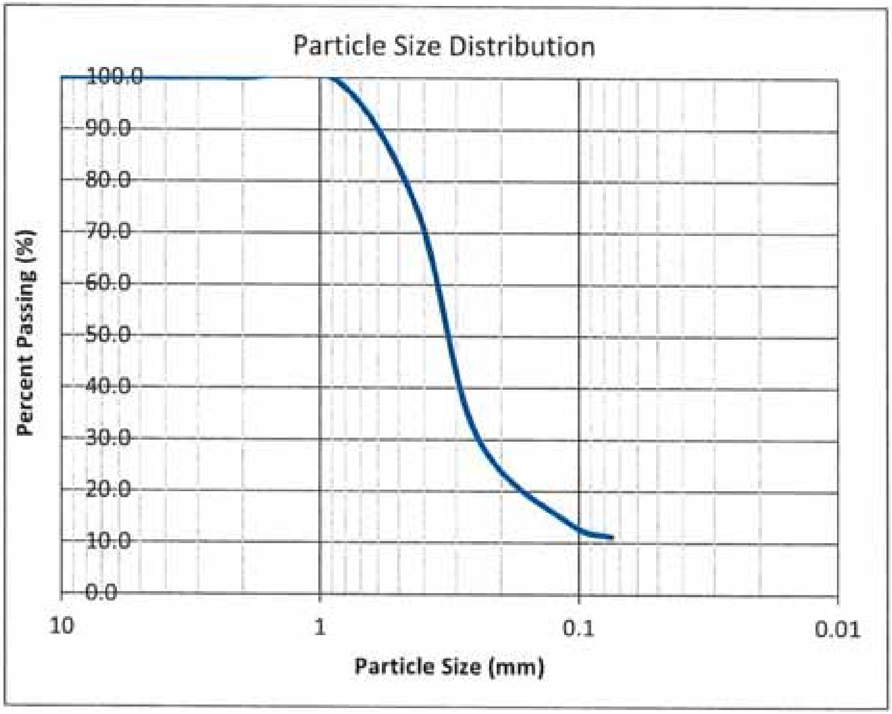
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

TB-1a-5 @ 20-21'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.2
No. 40	0.425	74.6
No. 60	0.25	30.9
No. 140	0.106	13.5
No. 200	0.075	11.2
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	25.4
Fine Sand (<No. 40-No. 200):	63.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	88.8	11.2

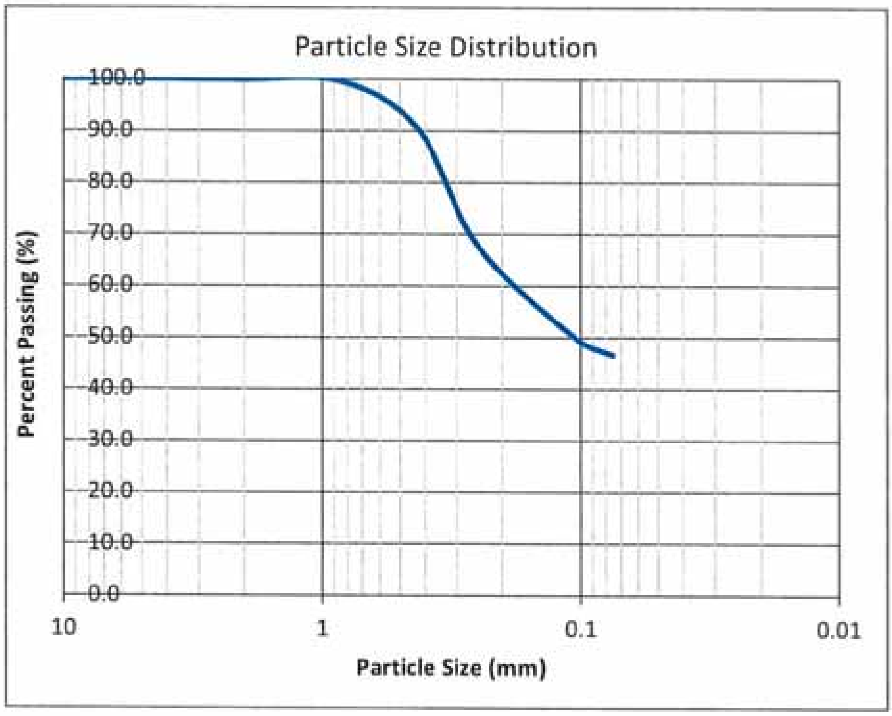
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B3-6 @ 23'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	99.9
No. 20	0.85	99.6
No. 40	0.425	90.4
No. 60	0.25	67.9
No. 140	0.106	50.1
No. 200	0.075	46.6
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.1
Medium Sand (<No. 10-No. 40):	9.5
Fine Sand (<No. 40-No. 200):	43.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	53.4	46.6

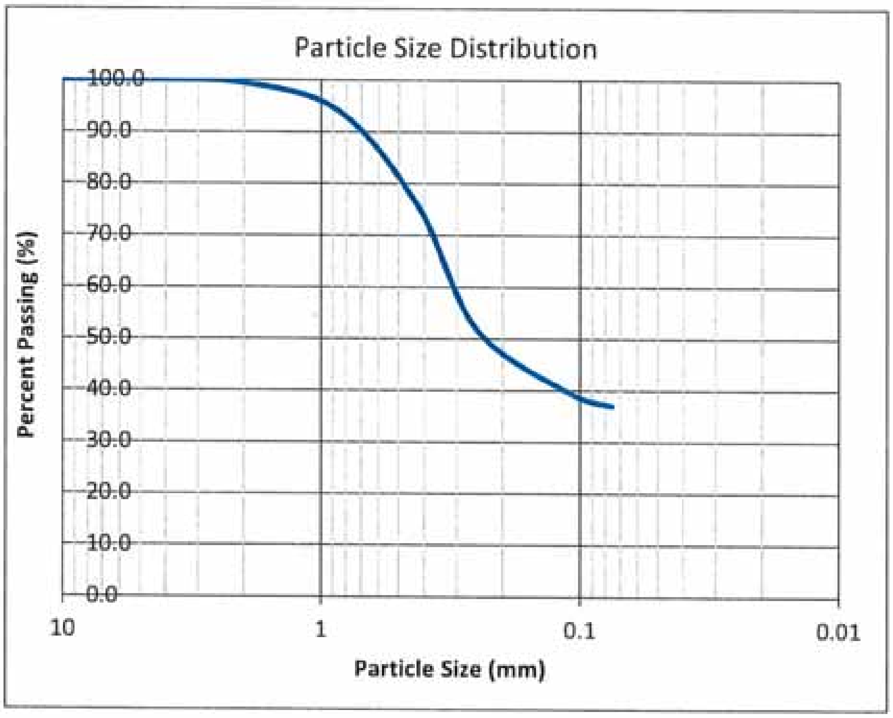
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B4-3 @ 15'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	99.4
No. 20	0.85	93.9
No. 40	0.425	75.8
No. 60	0.25	51.5
No. 140	0.106	39.2
No. 200	0.075	37.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.6
Medium Sand (<No. 10-No. 40):	23.6
Fine Sand (<No. 40-No. 200):	38.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	63.0	37.0

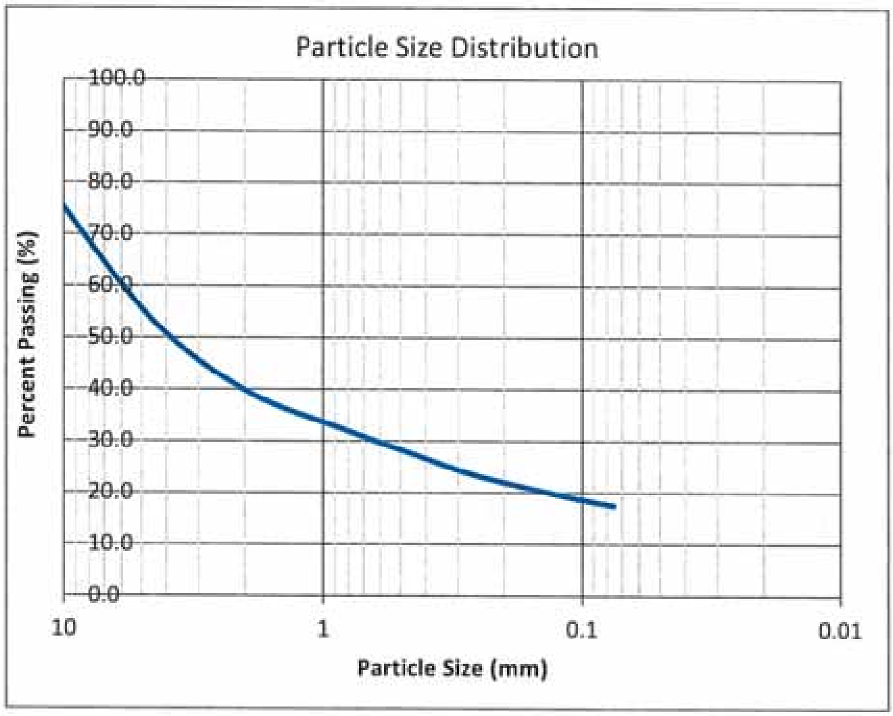
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B5-3 @ 12-14'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	81.6
No. 4	4.75	54.5
No. 10	2	39.9
No. 20	0.85	32.4
No. 40	0.425	27.1
No. 60	0.25	23.2
No. 140	0.106	18.9
No. 200	0.075	17.5
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	18.4
Course Sand (<No. 4-No.10):	41.8
Medium Sand (<No. 10-No. 40):	12.8
Fine Sand (<No. 40-No. 200):	9.6
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
18.4	64.1	17.5

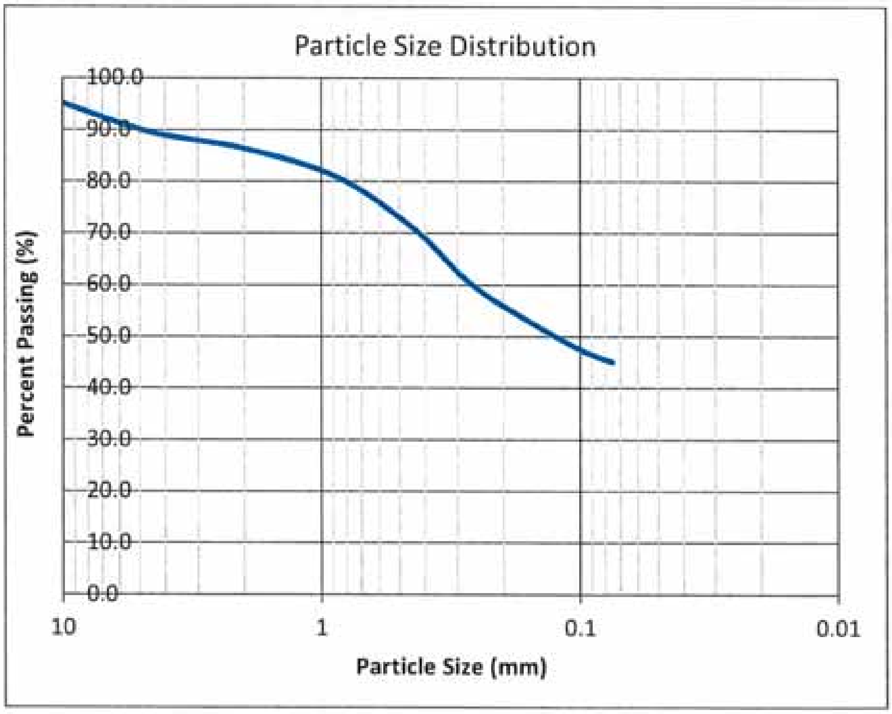
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B6-2 @ 10-12'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	96.8
No. 4	4.75	89.7
No. 10	2	86.4
No. 20	0.85	80.7
No. 40	0.425	70.3
No. 60	0.25	59.1
No. 140	0.106	48.0
No. 200	0.075	45.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	3.2
Course Sand (<No. 4-No.10):	10.5
Medium Sand (<No. 10-No. 40):	16.1
Fine Sand (<No. 40-No. 200):	25.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
3.2	51.8	45.0

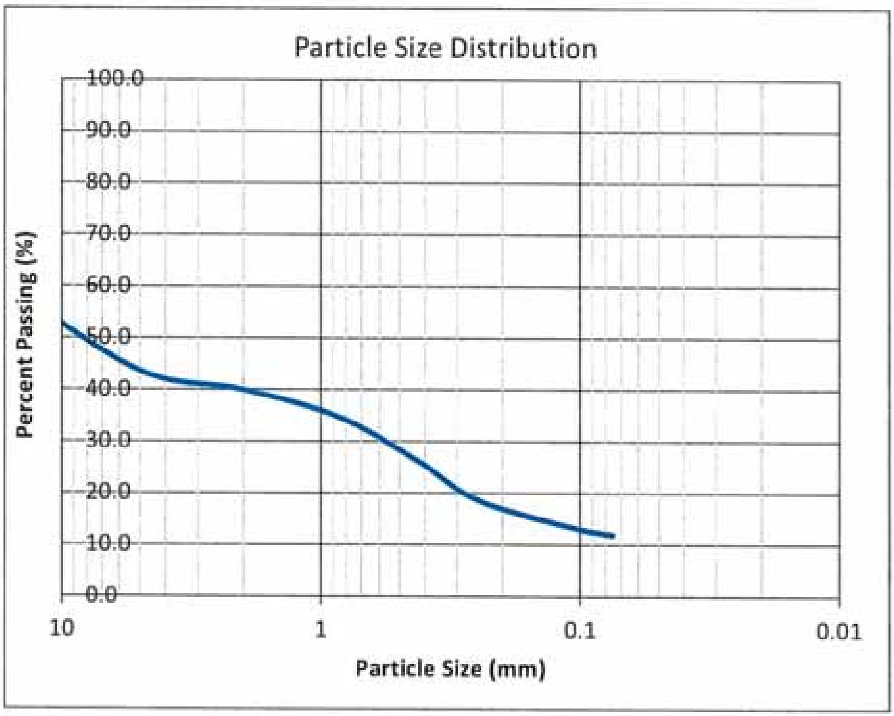
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B21-3 @ 10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	83.4
3/8"	12.5	55.7
No. 4	4.75	43.1
No. 10	2	39.9
No. 20	0.85	34.6
No. 40	0.425	26.2
No. 60	0.25	18.7
No. 140	0.106	13.3
No. 200	0.075	12.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	16.6
Fine Gravel (<3/4"- No. 4):	27.6
Course Sand (<No. 4-No.10):	15.8
Medium Sand (<No. 10-No. 40):	13.7
Fine Sand (<No. 40-No. 200):	14.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
44.3	43.8	12.0

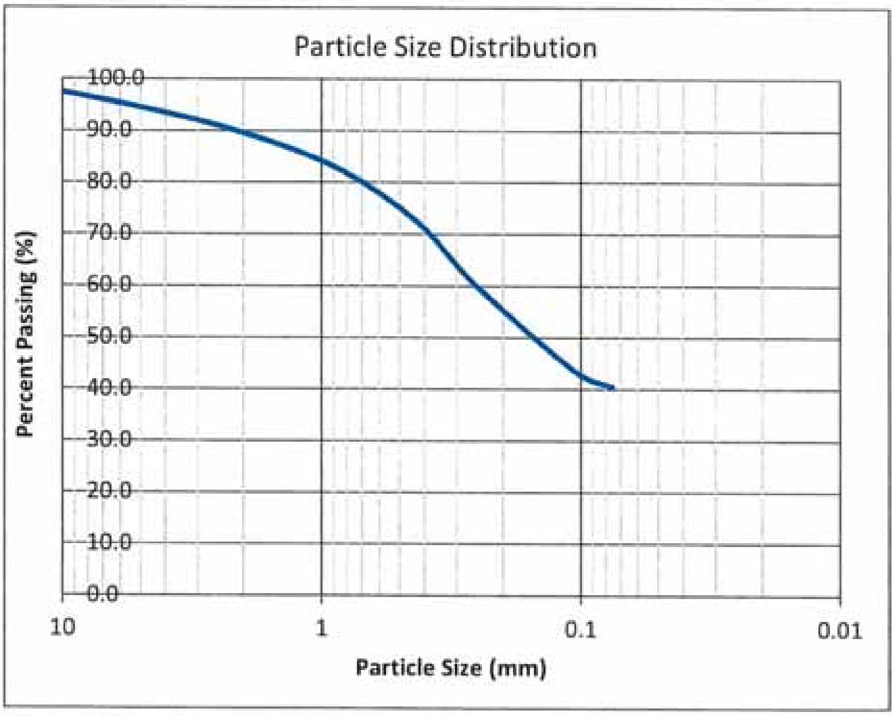
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B7-5 @ 16-18'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	98.4
No. 4	4.75	94.2
No. 10	2	89.5
No. 20	0.85	82.5
No. 40	0.425	72.2
No. 60	0.25	59.8
No. 140	0.106	43.6
No. 200	0.075	40.4
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	1.6
Course Sand (<No. 4-No.10):	8.8
Medium Sand (<No. 10-No. 40):	17.3
Fine Sand (<No. 40-No. 200):	31.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
1.6	58.0	40.4

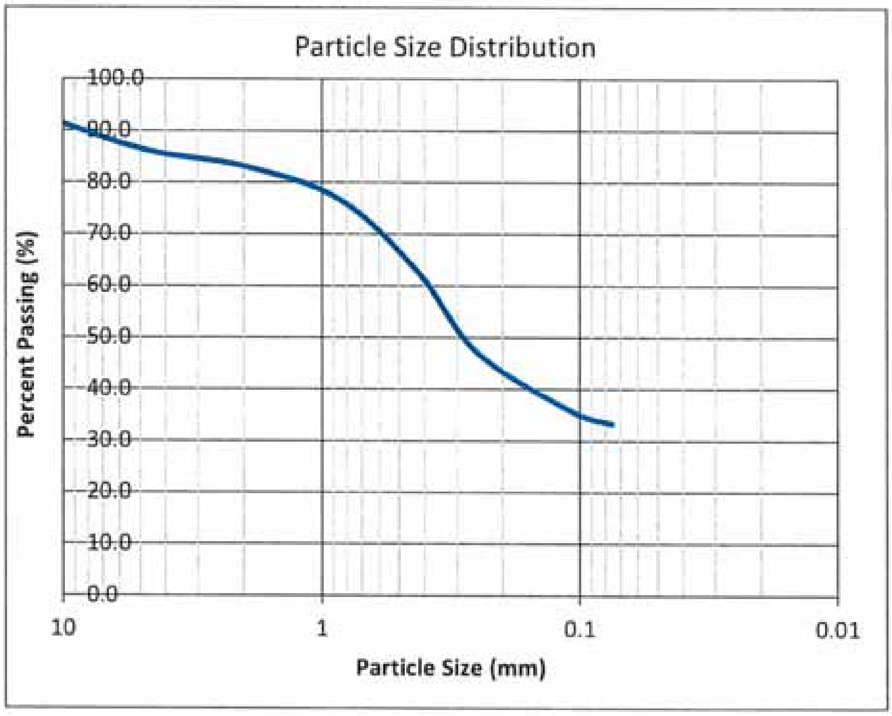
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B8-2 @ 10-12'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	92.9
No. 4	4.75	86.2
No. 10	2	83.1
No. 20	0.85	76.7
No. 40	0.425	62.8
No. 60	0.25	47.0
No. 140	0.106	35.5
No. 200	0.075	33.3
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	7.1
Course Sand (<No. 4-No.10):	9.8
Medium Sand (<No. 10-No. 40):	20.3
Fine Sand (<No. 40-No. 200):	29.5
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
7.1	59.6	33.3

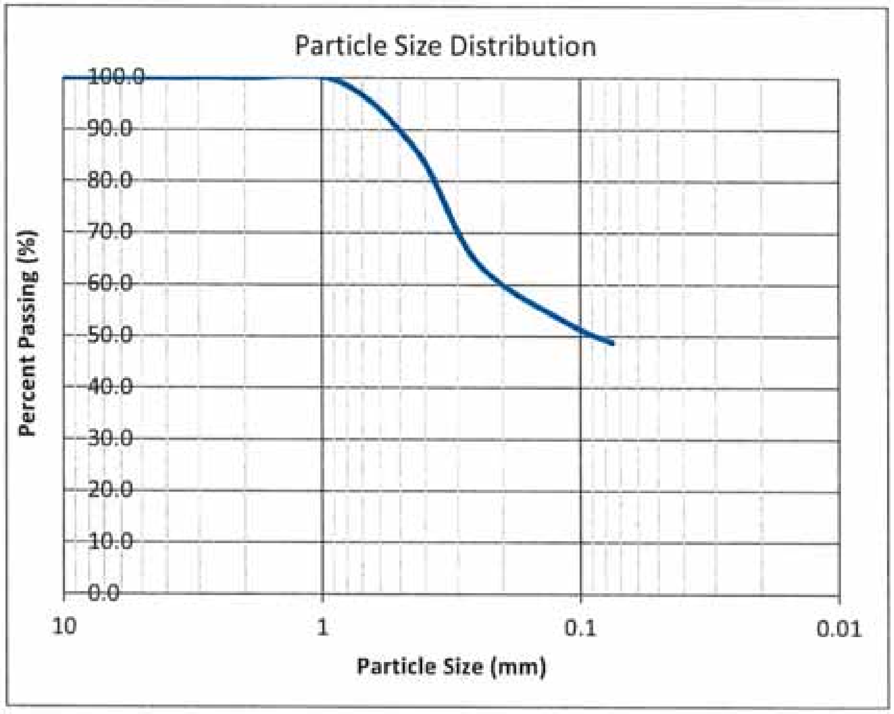
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B9-3 @ 11-13'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.2
No. 40	0.425	85.5
No. 60	0.25	64.1
No. 140	0.106	52.0
No. 200	0.075	48.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	14.5
Fine Sand (<No. 40-No. 200):	36.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	51.2	48.8

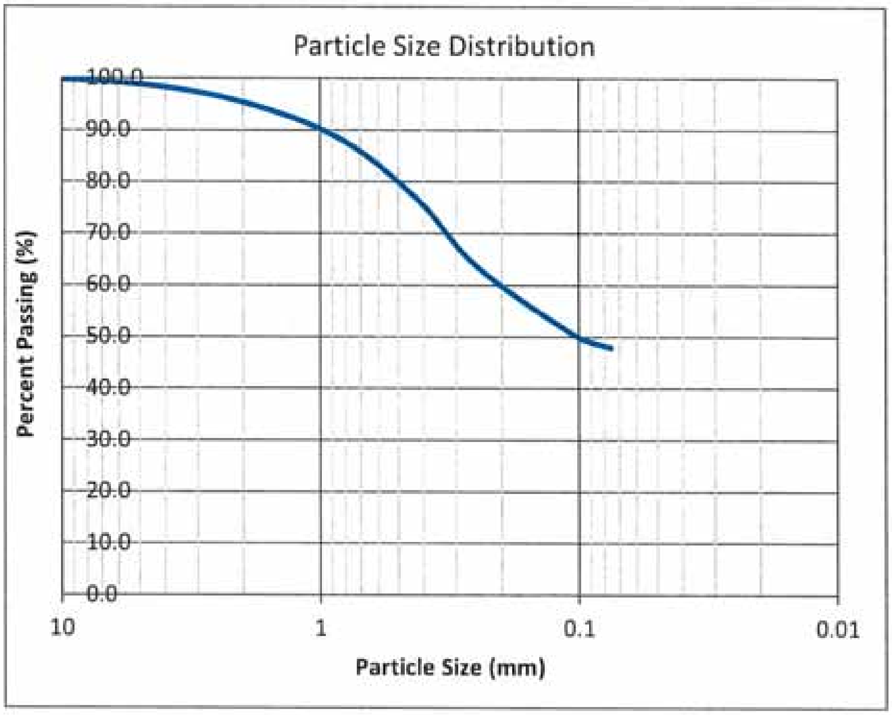
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B10-2 @ 10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	98.7
No. 10	2	95.3
No. 20	0.85	88.4
No. 40	0.425	76.7
No. 60	0.25	63.6
No. 140	0.106	50.6
No. 200	0.075	47.9
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	4.7
Medium Sand (<No. 10-No. 40):	18.6
Fine Sand (<No. 40-No. 200):	28.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	52.1	47.9

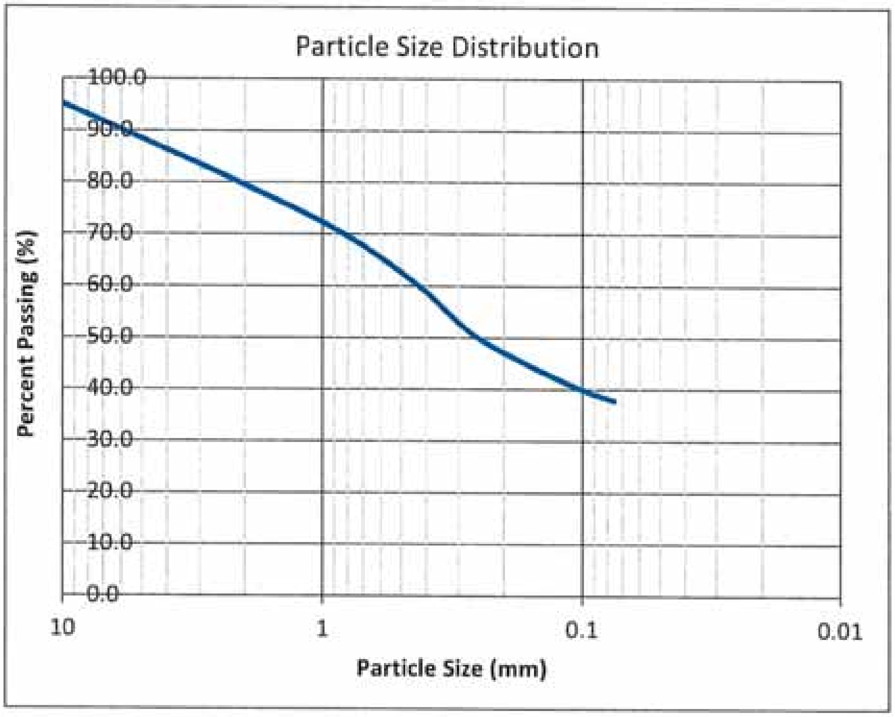
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B11-2 @ 8-10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	97.3
No. 4	4.75	87.9
No. 10	2	79.5
No. 20	0.85	70.3
No. 40	0.425	59.9
No. 60	0.25	49.9
No. 140	0.106	40.5
No. 200	0.075	37.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	2.7
Course Sand (<No. 4-No.10):	17.8
Medium Sand (<No. 10-No. 40):	19.6
Fine Sand (<No. 40-No. 200):	22.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
2.7	59.5	37.8

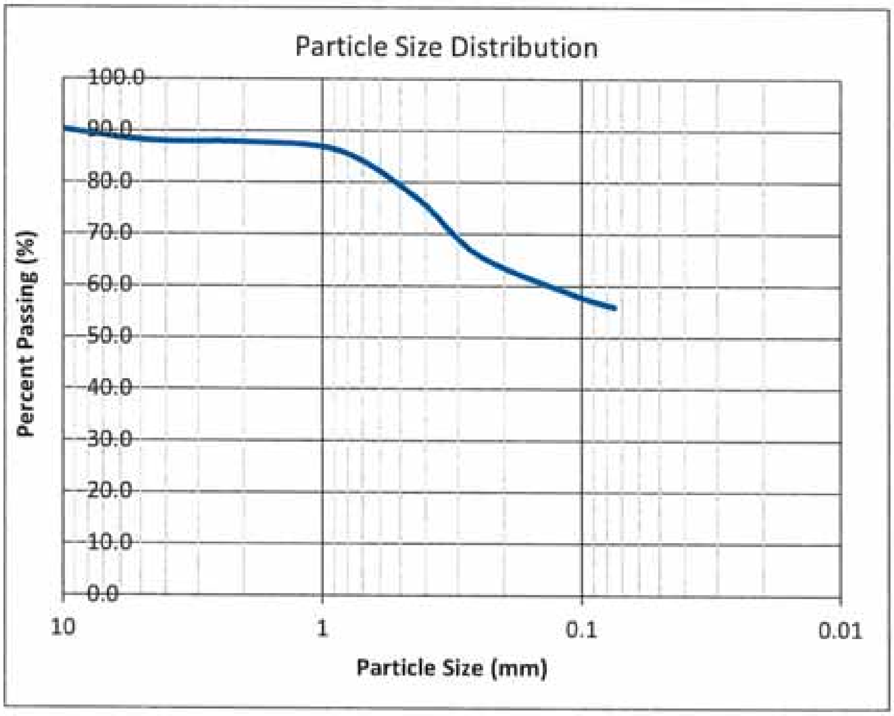
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B12-3 @ 10-11'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	90.9
No. 4	4.75	88.1
No. 10	2	87.8
No. 20	0.85	86.0
No. 40	0.425	76.7
No. 60	0.25	65.9
No. 140	0.106	58.1
No. 200	0.075	55.9
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	9.1
Course Sand (<No. 4-No.10):	3.1
Medium Sand (<No. 10-No. 40):	11.1
Fine Sand (<No. 40-No. 200):	20.9
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
9.1	35.0	55.9

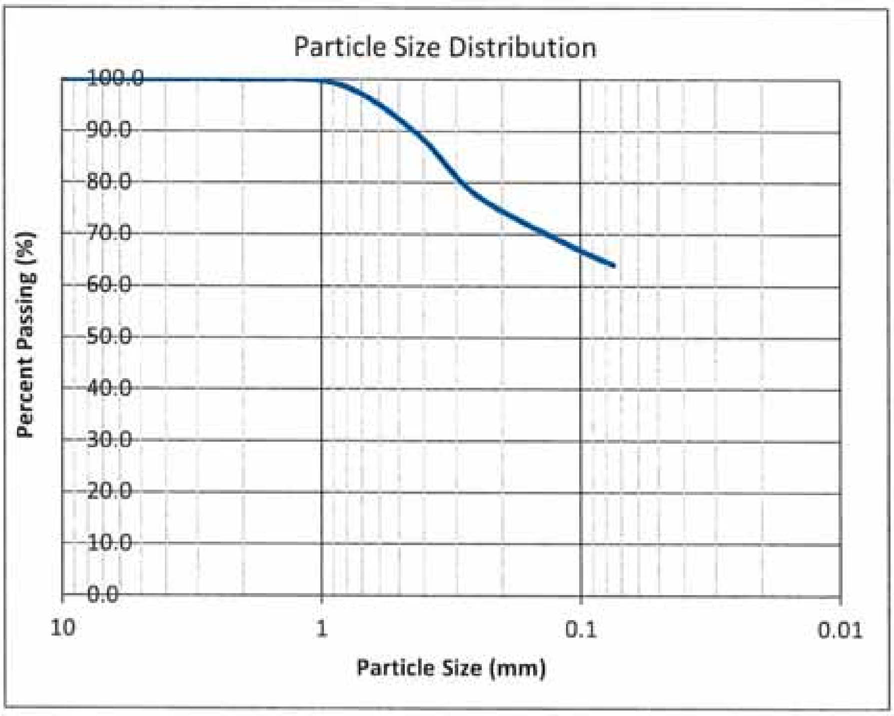
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B13-2 @ 7-8'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.0
No. 40	0.425	89.5
No. 60	0.25	77.5
No. 140	0.106	67.4
No. 200	0.075	64.1
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	10.5
Fine Sand (<No. 40-No. 200):	25.4
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	35.9	64.1

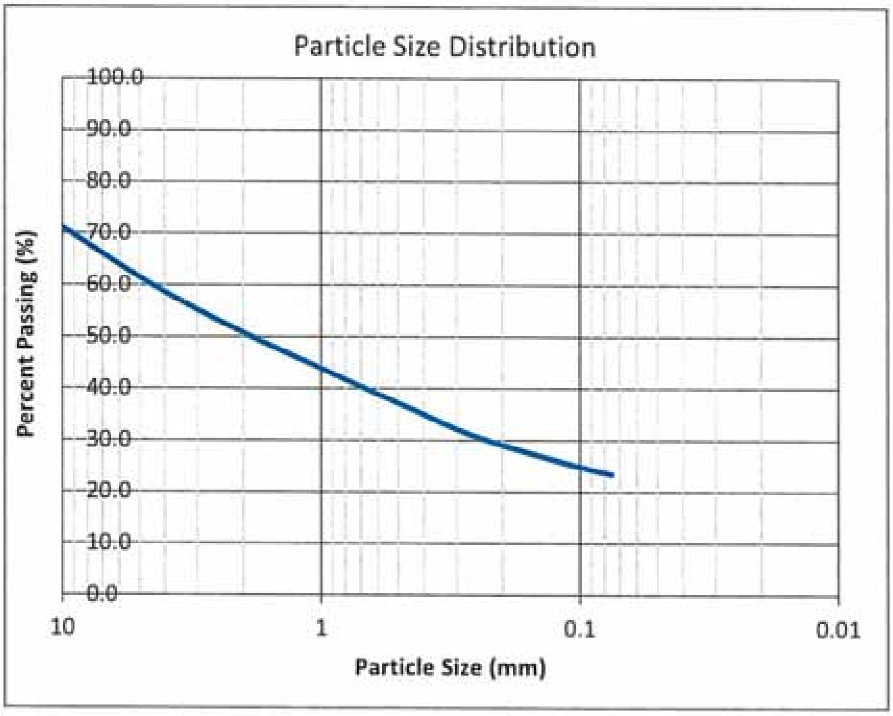
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B14-3 @ 13-14'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	74.2
No. 4	4.75	60.7
No. 10	2	50.8
No. 20	0.85	42.3
No. 40	0.425	35.7
No. 60	0.25	30.6
No. 140	0.106	25.2
No. 200	0.075	23.4
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	25.8
Course Sand (<No. 4-No.10):	23.5
Medium Sand (<No. 10-No. 40):	15.1
Fine Sand (<No. 40-No. 200):	12.2
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
25.8	50.8	23.4

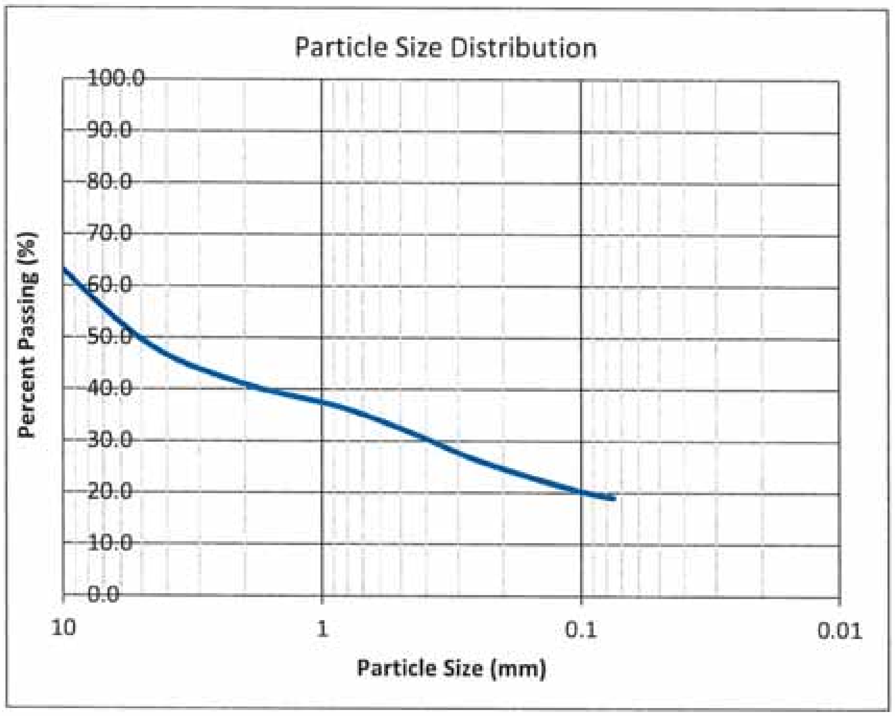
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

TB-2a-7 @ 32'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	67.6
No. 4	4.75	48.9
No. 10	2	41.0
No. 20	0.85	36.5
No. 40	0.425	31.0
No. 60	0.25	26.2
No. 140	0.106	20.6
No. 200	0.075	19.1
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	32.4
Course Sand (<No. 4-No.10):	26.6
Medium Sand (<No. 10-No. 40):	10.0
Fine Sand (<No. 40-No. 200):	11.9
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
32.4	48.5	19.1

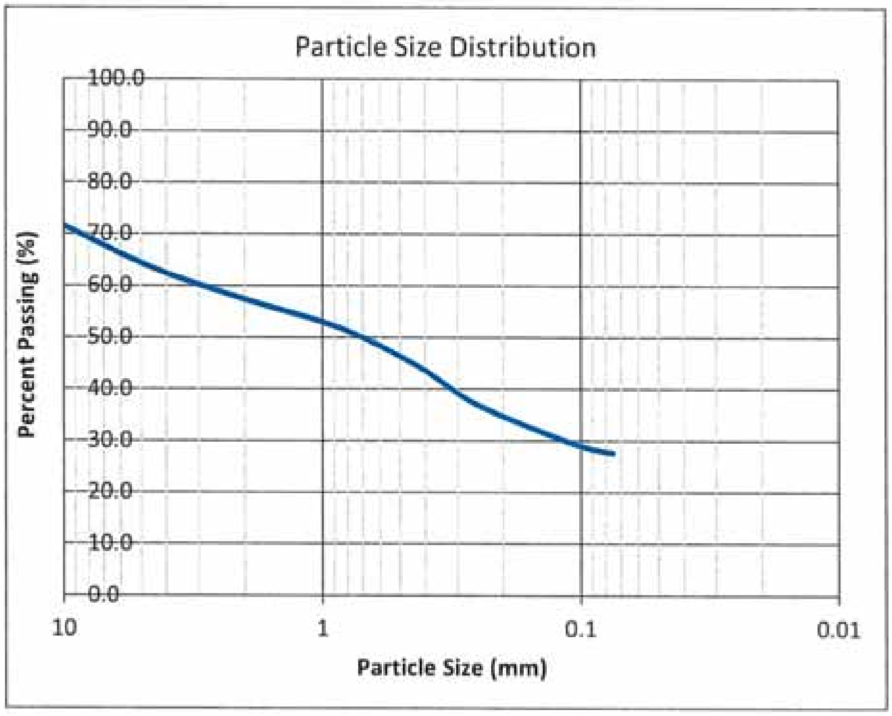
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B16-4 @ 15-16.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	74.0
No. 4	4.75	63.8
No. 10	2	57.3
No. 20	0.85	51.7
No. 40	0.425	44.3
No. 60	0.25	36.9
No. 140	0.106	29.4
No. 200	0.075	27.6
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	26.0
Course Sand (<No. 4-No.10):	16.7
Medium Sand (<No. 10-No. 40):	13.0
Fine Sand (<No. 40-No. 200):	16.7
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
26.0	46.4	27.6

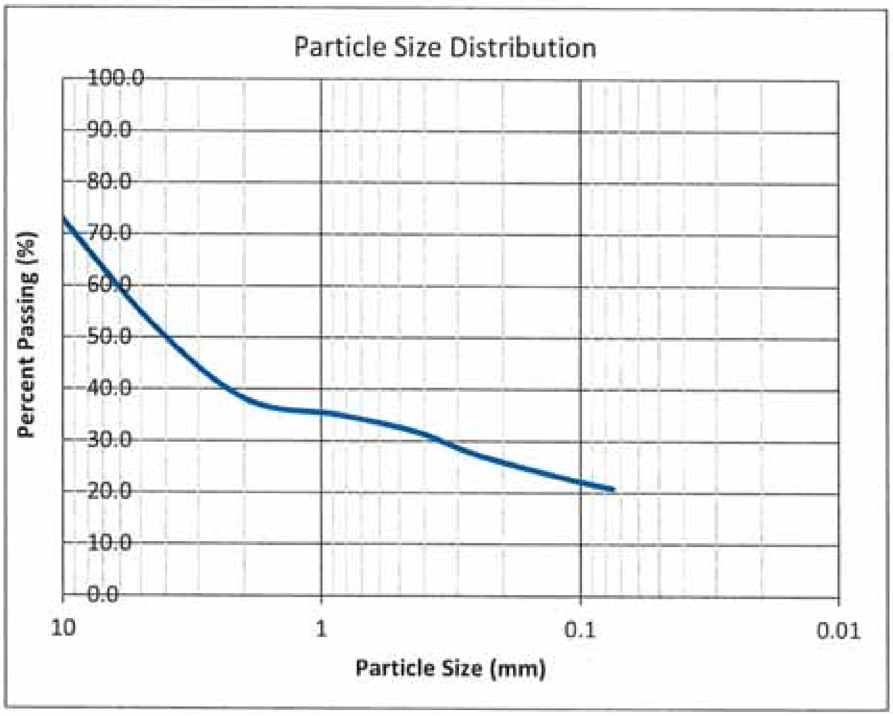
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B17-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	78.8
No. 4	4.75	53.8
No. 10	2	38.2
No. 20	0.85	35.0
No. 40	0.425	31.7
No. 60	0.25	27.3
No. 140	0.106	22.4
No. 200	0.075	20.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	21.2
Course Sand (<No. 4-No.10):	40.6
Medium Sand (<No. 10-No. 40):	6.5
Fine Sand (<No. 40-No. 200):	10.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
21.2	58.0	20.8

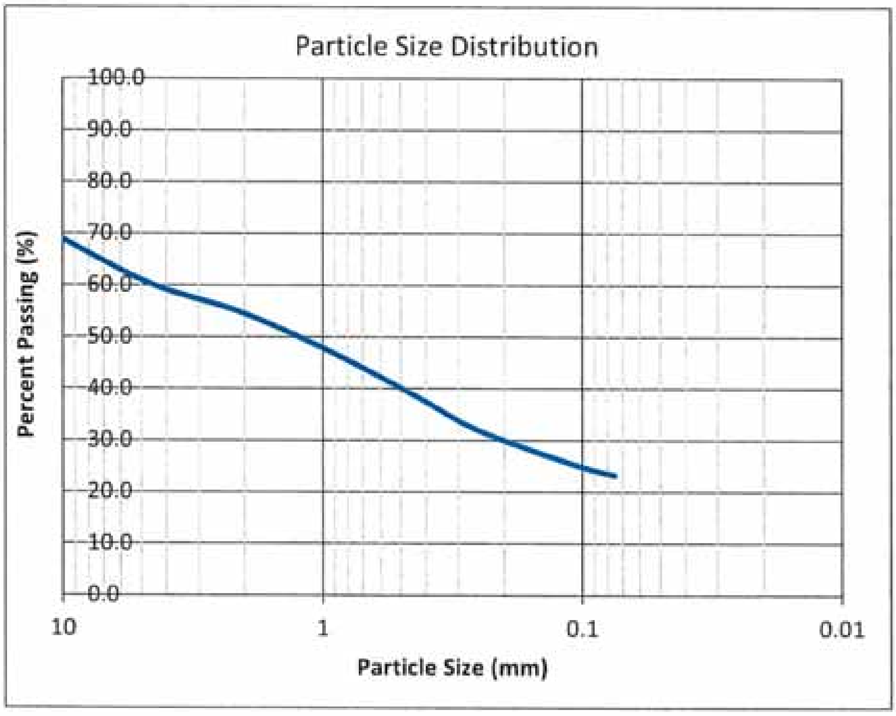
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B18-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	88.0
3/4"	19	77.4
3/8"	12.5	71.4
No. 4	4.75	60.5
No. 10	2	54.5
No. 20	0.85	46.1
No. 40	0.425	38.2
No. 60	0.25	31.9
No. 140	0.106	25.2
No. 200	0.075	23.2
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	22.6
Fine Gravel (<3/4"- No. 4):	5.9
Course Sand (<No. 4-No.10):	16.9
Medium Sand (<No. 10-No. 40):	16.3
Fine Sand (<No. 40-No. 200):	15.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
28.6	48.2	23.2

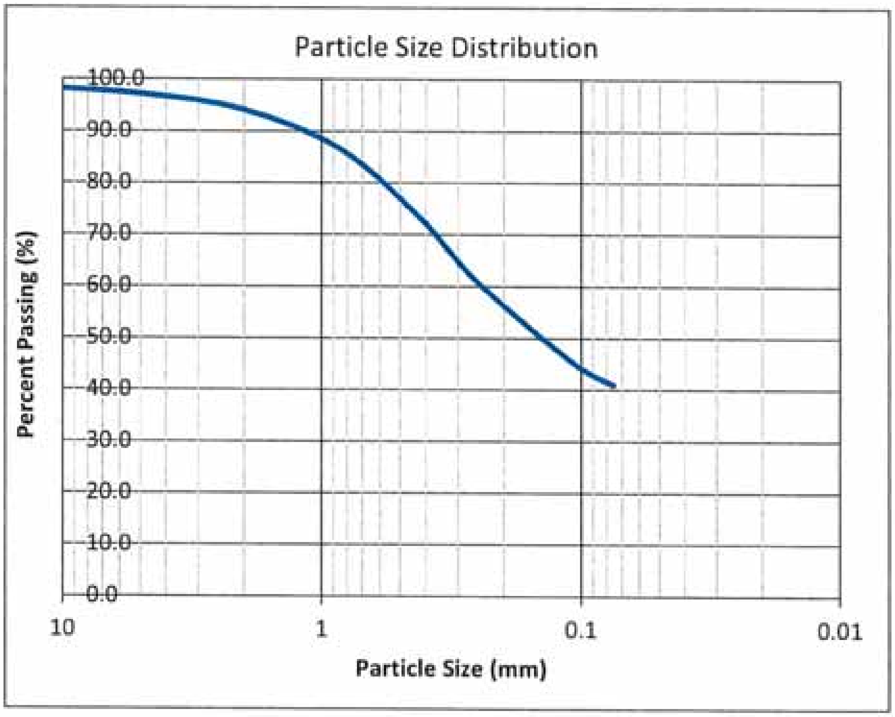
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B19-3 @ 14-15'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	98.4
No. 4	4.75	97.0
No. 10	2	94.1
No. 20	0.85	86.5
No. 40	0.425	73.4
No. 60	0.25	60.6
No. 140	0.106	45.1
No. 200	0.075	41.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	1.6
Course Sand (<No. 4-No.10):	4.3
Medium Sand (<No. 10-No. 40):	20.6
Fine Sand (<No. 40-No. 200):	32.5
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
1.6	57.4	41.0

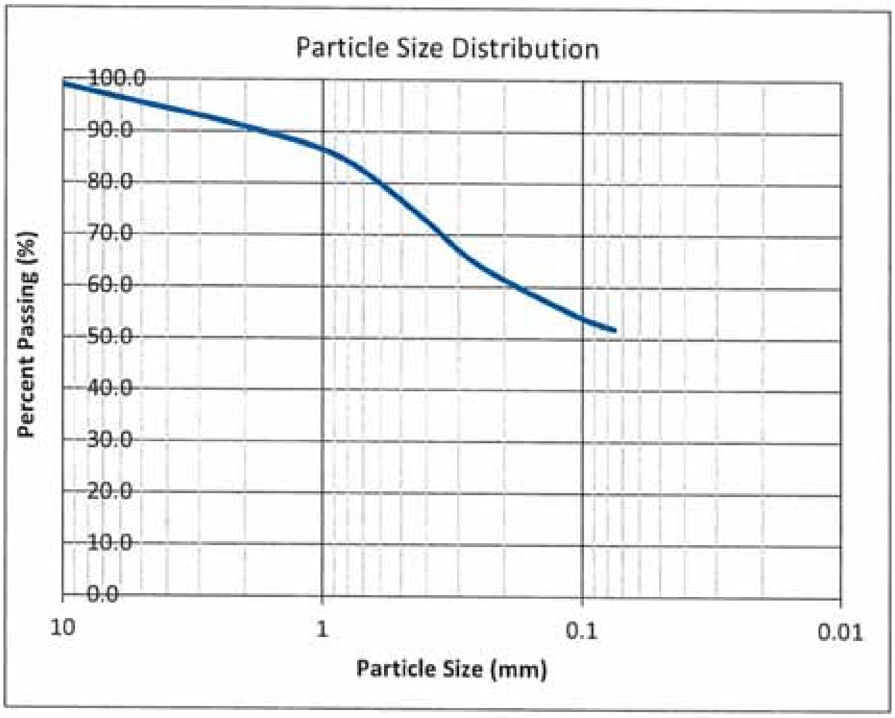
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B20-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	95.2
No. 10	2	90.9
No. 20	0.85	84.9
No. 40	0.425	73.8
No. 60	0.25	64.2
No. 140	0.106	54.5
No. 200	0.075	51.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	9.1
Medium Sand (<No. 10-No. 40):	17.0
Fine Sand (<No. 40-No. 200):	22.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	48.2	51.8

February 22, 2017

Mr. Bob Smillie
TerraCosta Consulting Group
3890 Murphy Canyon Road
San Diego, California 92123

Subject: **Limited Petrographic Analysis**
Stadium Conglomerate Clasts in Rock Cores
NCCS Pipeline
San Diego, California
AmecFW Project No. 5015-15-0031.21
AmecFW Lab Nos. 30322-30324

Dear Mr. Smillie:

Amec Foster Wheeler Environment and Infrastructure, Inc. (Amec Foster Wheeler), is pleased to present this report of our limited petrographic analysis performed for the subject project.

INTRODUCTION

Three boxes containing rock core samples were delivered to our laboratory for a limited petrographic analysis and other laboratory testing. The results of the other laboratory testing will be presented in a separate report. The 3 boxes, labeled TB4-B, TB4-C, and TB4-D were assigned laboratory tracking numbers 30322, 30323 and 30324, respectively. Each box contained approximately 15 feet of 2.4-inch diameter (HQ) rock core. We understand that the rock cores were obtained from the general elevation range (plus several feet above and below) of the proposed tunnel for the subject pipeline project.

TEST METHODS

A sieve analysis was performed on the rock core samples as part of our overall testing program. A limited petrographic analysis was performed on the clasts retained on the 1-1/2-inch, 1-inch, 3/4-inch, and 1/2-inch sieves in general accordance with the applicable sections of ASTM C295-12. Our analysis was limited to identifying the rock types comprising the clasts and estimating the relative percentages of each rock type. The estimates were based primarily on observation of the clasts with the unaided eye, although a limited number of clasts were observed through an AmScope stereographic microscope. No thin section slides were made, and no estimates were made for the clasts finer than the 1/2-inch sieve due to the small particle size and prevalence of surface weathering and coatings.

ANALYSIS RESULTS

The subject rock cores are comprised of Stadium Conglomerate with abundant, generally well-rounded clasts ranging up to about 5 inches in maximum dimension (5-inch maximum length contained within the 2.4-inch diameter rock core sample). The clasts are primarily comprised of volcanic (and/or volcaniclastic) rock and subordinate granitic rock and quartzite. In many cases, rock types were somewhat difficult to discern due to surface weathering and/or coatings on the clasts. Table 1 below presents the estimated composition of the clasts as percent by weight of the observed clasts (not percent by weight of the overall core sample). Photographs of typical clasts representing each rock type, as well as tables providing particle counts and the estimated percent by weight of the various rock types per sieve size, are attached to this report.

The large majority of the volcanic rocks exhibited a strong porphyritic texture. An attempt was made to separate the volcanic rocks into general compositional classifications based on color and correlation to more detailed petrographic analyses we have previously performed on aggregate samples derived from the Stadium Conglomerate and Pomerado Conglomerate (see discussion below). Conclusive classification of volcanic rocks typically requires thins section and/or chemical analysis, and the classifications and compositional estimates provided herein should be considered accurate only to the degree implied by the methods employed. The volcanic rock clasts were divided into the following categories:

- Rhyolite: volcanic rock that appears to fall within the compositional range of rhyolite, generally exhibiting a reddish to purplish to light brown color.
- Dacite: volcanic rock that appears to fall within the compositional range of dacite, generally exhibiting a dark gray to dark grayish-green color.
- Rhyodacite: volcanic rock that appears to have a composition borderline between rhyolite and dacite, exhibiting a range of intermediate grayish, brownish, greenish, or weakly purplish (purplish-gray or purplish-brown) colors.

The clasts identified as granitic rock exhibited a fairly wide range of colors, textures, and degree of weathering, and generally tend to be more weathered than the volcanic rock or quartzite clasts. The observed granitic rocks generally ranged from dark gray, very fine grained rock that appeared to be of dioritic composition, to light colored, fine to medium grained rock of somewhat variable granitic composition.

The clasts identified as quartzite generally exhibited typical quartzite textures and a wide variety of colors ranging from brown to orangish-brown to dark gray to black. A little less than two percent of the sample by weight consisted of a variety of undifferentiated rock types ("other" in Table 1) that appeared to include some foliated metamorphic rocks and very fine grained mafic rocks.

Table 1 – Estimated Composition of Subject Clasts

Rock Type	Estimated % of Clasts by Weight
Rhyolite	36.5
Rhyodacite	28.7
Dacite	21.5
(Total Volcanic Rock)	(86.7)
Granitic Rock	8.0
Quartzite	3.5
Other	1.9

PREVIOUS PETROGRAPHIC ANALYSIS RESULTS

Amec Foster Wheeler has performed several previous petrographic analyses on samples of concrete aggregate derived from the crushing of Stadium Conglomerate and Pomerado Conglomerate clasts. The Stadium Conglomerate samples were from the Carroll Canyon area of San Diego, whereas as the Pomerado Conglomerate samples were from the Lakeside/Santee area.

The lithology of the clasts contained within the Pomerado Conglomerate is generally considered to be very similar, if not identical, to that of the Stadium Conglomerate. Tables summarizing the results of these previous analyses are presented below. Volcanic rocks are not differentiated because of somewhat variable rock classification categories and/or analysis methods used in the previous analyses.

Table 2 – Previous Analyses on Aggregate from Stadium Conglomerate (Carroll Canyon)

Rock Type	% of Clasts by Weight (2008)	% of Clasts by Weight (2014-1)	% of Clasts by Weight (2014-2)
Volcanic Rock	88	87	84
Granitic Rock	8	7	9
Quartzite	2	3	3
Other	2	3	4

Table 3 – Previous Analyses on Aggregate from Pomerado Conglomerate (Lakeside/Santee)

Rock Type	% of Clasts by Weight (2008)	% of Clasts by Weight (2009)	% of Clasts by Weight (2010)	% of Clasts by Weight (2014)	% of Clasts by Weight (2016)
Volcanic Rock	94	91	89	88	87
Granitic Rock	3	2	2	7	5
Quartzite	2	3	8	2	3
Other	1	4	1	3	5

We trust this information meets your current needs. If more information is needed, please contact Mike Farr at 760-683-4117.

Sincerely,
AMEC FOSTER WHEELER ENVIRONMENT AND INFRASTRUCTURE, INC.

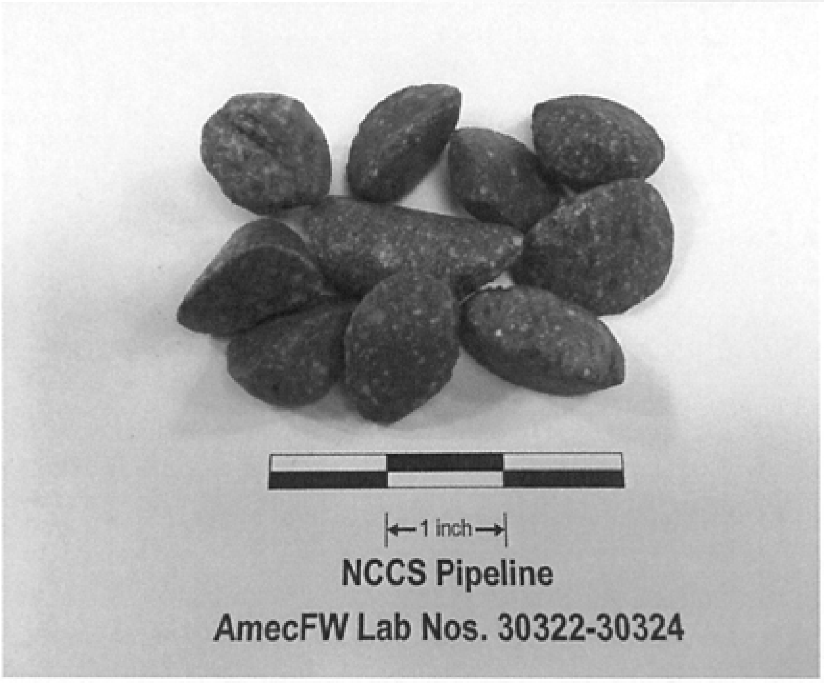
Michael P. Farr, CEG 1938
Associate Engineering Geologist

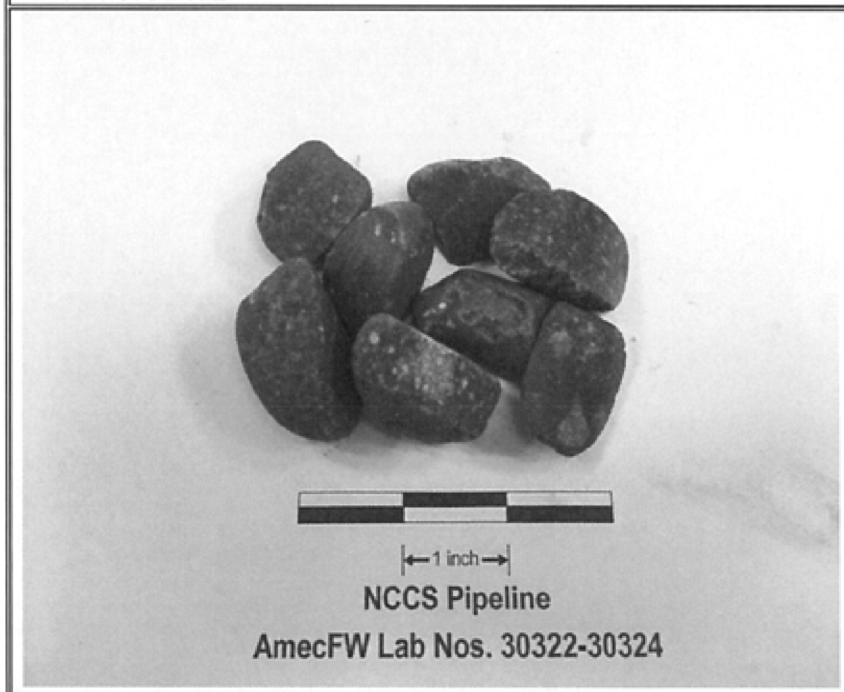
David C. Wilson, PE C54734
Senior Associate Engineer – Materials

MPF/DCW/mf

Attachments: Sample Photographs
Petrographic Analysis Tables

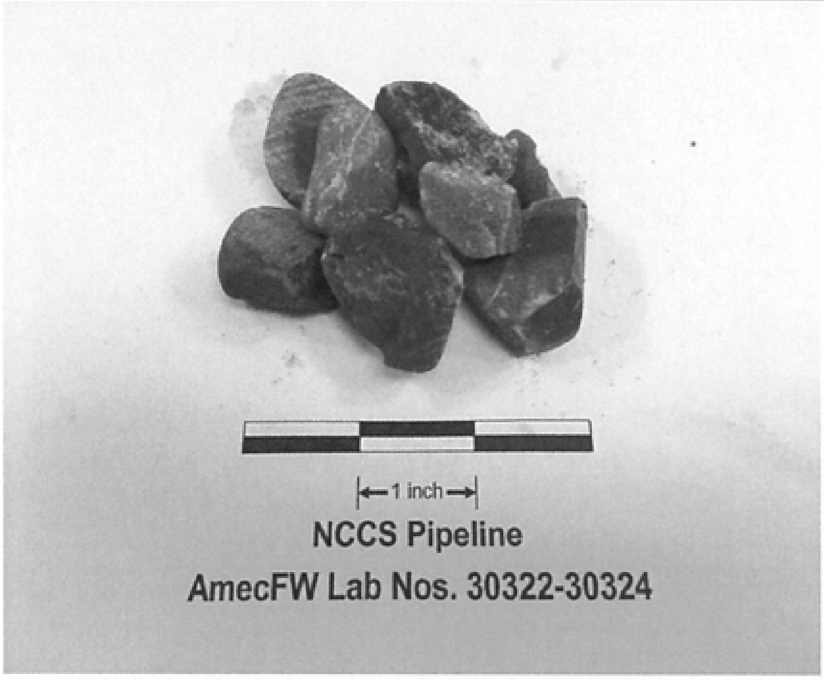
ATTACHMENTS

PHOTOGRAPH 1	**REMARKS**
	Typical clasts identified as rhyolite.

PHOTOGRAPH 2	**REMARKS**
	Typical clasts identified as rhyodacite.

PHOTOGRAPH 3	**REMARKS**
	Typical clasts identified as dacite.

PHOTOGRAPH 4	**REMARKS**
	Typical clasts identified as granitic rock.

PHOTOGRAPH 5	**REMARKS**
	Typical clasts identified as quartzite.

NCCS Pipeline - Stadium Conglomerate Clast Composition by Particle Count

Clasts Retained On Sieve Size:	**1-1/2"**		**1"**		**3/4"**		**1/2"**		**3/8"**		**#4**	
Rock Type	**# of Particles***	**% of Size Fraction**	**# of Particles***	**% of Size Fraction**	**# of Particles**	**% of Size Fraction**	**# of Particles**	**% of Size Fraction**	**# of Particles**	**% of Size Fraction**	**# of Particles**	**% of Size Fraction**
Rhyolite*	43	37.7	84	33.6	141	38.2	137	35.4	0	0.0	0	0.0
Rhyodacite*	29	25.4	82	32.8	106	28.7	125	32.3	0	0.0	0	0.0
Dacite*	24	21.1	55	22.0	82	22.2	83	21.4	0	0.0	0	0.0
Quartzite	4	3.5	9	3.6	11	3.0	14	3.6	0	0.0	0	0.0
Granitic Rock	11	9.6	17	6.8	25	6.8	23	5.9	0	0.0	0	0.0
Other	3	2.6	3	1.2	4	1.1	5	1.3	0	0.0	0	0.0
	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
Totals	**114**		**250**		**369**		**387**		**0**		**0**	

*Volcanic/volcaniclastic rock, estimated rock name based largely on clast color.

NCCS Pipeline - Stadium Conglomerate Clast Composition - Percent of Observed Clasts by Weight

Particles Retained On Sieve Size:	1-1/2"		1"		3/4"		1/2"		3/8"		#4	
Rock and Mineral Constituents	**% of Size Fraction**	**% of Sample**	**% of Size Fraction**	**% of Sample**	**% of Size Fraction**	**% of Sample**	**% of Size Fraction**	**% of Sample**	**% of Size Fraction**	**% of Sample**	**% of Size Fraction**	**% of Sample**
Rhyolite*	37.7	17.8	33.6	8.6	38.2	5.5	35.4	4.6	0.0	0.0	0.0	0.0
Rhyodacite*	25.4	12.0	32.8	8.4	28.7	4.1	32.3	4.2	0.0	0.0	0.0	0.0
Dacite*	21.1	9.9	22.0	5.6	22.2	3.2	21.4	2.8	0.0	0.0	0.0	0.0
Quartzite	3.5	1.7	3.6	0.9	3.0	0.4	3.6	0.5	0.0	0.0	0.0	0.0
Granitic Rock	9.6	4.5	6.8	1.7	6.8	1.0	5.9	0.8	0.0	0.0	0.0	0.0
Other	2.6	1.2	1.2	0.3	1.1	0.2	1.3	0.2	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

*Volcanic/volconiclastsic rock, estimated rock name based largely on clast color.
**Sieve analysis results normalized to include only clasts retained on the 1-1/2", 1", 3/4", and 1/2" sieves.

Constituents	% of Clasts by Weight
Rhyolite*	36.5
Rhyodacite*	28.7
Dacite*	21.5
Quartzite	3.5
Granitic Rock	8.0
Other	1.9

Sieve Analysis Results** (ASTM C136-06)	
Sieve Size	**% Retained**
1-1/2"	47.1
1"	25.5
3/4"	14.4
1/2"	12.9

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PHYSICAL PROPERTIES FOR POINT LOAD TESTING OF SAMPLES
ASTM D5731-08

PROJECT: #2934 NCCS Pipeline TB 4b @ 77'	LAB NO.: 30322	PROJECT NO.: 5015-15-0030.21
	SAMPLED BY: GAS	DATE: 11/2016
	SUBMITTED BY:	DATE: 01/27/17
	AUTHORIZED BY:	DATE: 01/27/17
	REVIEWED BY: L. Collins	REPORT DATE:

#	Type: (a) Diametral, (b) Axial, (c) Block (d) Irregular	AVG. L (in.)	AVG. D (in.)	AVG. W (in.)	Time Load Applied (s)	Total Load (psi)	I_s (psi)	$I_{s(50)}$ (psi)
1	(a)	2.550	2.402	---	26	733	127	139
2	(d)	2.406	1.922	2.406	15	587	100	110
3	(a)	1.926	2.389	---	57	241	42	46
4	(a)	2.107	2.412	---	87	4296	738	809
5	(d)	2.377	2.064	2.414	93	6236	982	1098
6	(d)	2.861	1.655	2.410	65	5020	988	1050
7	(d)	2.792	2.287	2.396	22	3595	515	588
8	(a)	3.561	2.407	---	35	7023	1212	1327
9	(a)	2.680	2.408	---	24	4332	747	818
10	(a)	3.773	2.398	---	48	8365	1455	1590
	Mean Value (disregarding lowest 2 and highest 2 values), psi =							750

Distribution:
TerraCosta Consulting Inc./ M. Eckert

Amec Foster Wheeler

Reviewed By:_______________________
David C. Wilson, CE#54734
Senior Principal Engineer

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PHYSICAL PROPERTIES FOR POINT LOAD TESTING OF SAMPLES
ASTM D5731-08

PROJECT: #2934 NCCS Pipeline	LAB NO.: 30323	PROJECT NO.: 5015-15-0030.21
TB 4c @ 86'	SAMPLED BY: GAS	DATE: 11/2016
	SUBMITTED BY:	DATE: 01/27/17
	AUTHORIZED BY:	DATE: 01/27/17
	REVIEWED BY: L. Collins	REPORT DATE:

#	Type: (a) Diametral, (b) Axial, (c) Block (d) Irregular	AVG. L (in.)	AVG. D (in.)	AVG. W (in.)	Time Load Applied (s)	Total Load (lbs)	I_s (psi)	$I_{s(50)}$ (psi)
1	(a)	8.131	2.406	---	58	7005	1210	1325
2	(a)	2.796	2.407	---	37	10065	1737	1902
3	(d)	1.733	1.987	2.411	16	2073	340	376
4	(a)	3.465	2.382	---	47	8707	1535	1672
5	(d)	2.610	1.071	2.289	46	3390	1086	1034
6	(a)	2.315	2.399	---	18	3280	570	623
7	(d)	3.082	1.822	2.212	56	5429	1057	1127
8	(d)	2.303	1.474	2.333	38	2115	483	496
9	(d)	2.296	1.466	2.294	19	3975	928	949
10	(d)	1.998	1.989	2.406	32	2466	405	448
	Mean Value (disregarding lowest 2 and highest 2 values), psi =							926

Distribution:
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Reviewed By:____________________
David C. Wilson, CE#54734
Senior Principal Engineer

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PHYSICAL PROPERTIES FOR POINT LOAD TESTING OF SAMPLES
ASTM D5731-08

PROJECT: #2934 NCCS Pipeline	LAB NO.: 30324	PROJECT NO.: 5015-15-0030.21
TB 4d @ 93'	SAMPLED BY: GAS	DATE: 11/2016
	SUBMITTED BY:	DATE: 01/27/17
	AUTHORIZED BY:	DATE: 01/27/17
	REVIEWED BY: L. Collins	REPORT DATE:

#	Type: (a) Diametral, (b) Axial, (c) Block (d) Irregular	AVG. L (in.)	AVG. D (in.)	AVG. W (in.)	Time Load Applied (s)	Total Load (lbs)	I_s (psi)	$I_{s(50)}$ (psi)
1	(a)	1.728	2.408	---	12	3662	632	692
2	(d)	2.513	1.596	2.363	31	5157	1073	1127
3	(d)	1.881	2.142	2.408	24	5184	789	889
4	(d)	3.256	1.542	2.333	58	2250	491	510
5	(d)	3.156	1.524	2.063	36	5121	1279	1288
6	(d)	2.876	1.568	2.359	53	5783	591	617
7	(a)	6.435	2.413	---	36	328	56	62
8	(d)	5.501	1.737	2.405	25	4903	921	990
9	(d)	3.909	2.360	2.403	48	3968	549	632
10	(a)	4.620	2.406	---	29	5263	909	995
	Mean Value (disregarding lowest 2 and highest 2 values), psi =							802

Distribution:
TerraCosta Consulting Inc./ M. Eckert

Amec Foster Wheeler

Reviewed By:____________________
David C. Wilson, CE#54734
Senior Principal Engineer

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PHYSICAL PROPERTIES OF SOILS

PROJECT: #2934 NCCS Pipeline	LAB NO.: 30322-30375 (page 1 of 3)	PROJECT NO.: 5015-15-0030.21
	SAMPLED BY: GAS	DATE: 11/2016
	SUBMITTED BY:	DATE: 01/27/17
	AUTHORIZED BY:	DATE: 01/27/17
	REVIEWED BY: L. Collins	REPORT DATE:

Sample I.D.	Depth (ft.)	-#200 Wash, % Loss ASTM D1140	Liquid and Plastic Limit ASTM D43189	Organic Impurity ASTM C40	Dry Density (pcf)	Moisture Content (%), as received ASTM D 2216
B1-1 (#30326)	5'	87.6	21.0/46.8/25.8	*	*	20.7
B1-3 (#30327)	10'	*	34.5/14.5/20.0	*	*	*
B2-1 (#30328)	5'					24.0
B2-3 (#30329)	15'					22.0
TB-1a-1 (#30330)	5'	14.9	25.4/18.8/6.6	*	*	8.1
TB-1a-2 (#30331)	10'	*	*	*	*	18.3
TB-1a-3 (#30332)	15'	13.8	*	*	*	12.2
TB-1a-5&6 (#30333)	25' & 30'	7.7	23.4/21.5/1.9	*	*	11.4
TB-1b-1 (#30334)	5'	39.3	25.3/19.5/5.8	*	*	7.9
TB-1b-2 (#30335)	10'	9.0	*	*	*	*
TB-1b-3&4 (#30336)	15' & 20'	8.9	26.2/19.7/6.5	*	*	9.9
B3-1 (#30337)	5'	52.8	58.0/16.9/41.1			16.7

Sample I.D.	Depth (ft.)	-#200 Wash, % Loss ASTM D1140	Liquid and Plastic Limit ASTM D43189	Organic Impurity ASTM C40	Dry Density (pcf)	Moisture Content (%), as received ASTM D 2216
B3-2 (#30338)	10'	18.2	*	*	*	9.6
B3-3 & B3-4 (#30339)	12.5' & 15'	19.2	29.8/17.6/12.2	*	*	9.3
B4-2 (#30340)	10'	12.0	30.0/17.6/12.4			12.0
B5-1 (#30341)	5'	18.1	35.1/11.8/23.3	*	*	6.4
B6-1 (#30342)	5'			Yes, closest to #5 organic plate		12.2
B6-3 (#30343)	15'	15.9	*	*	*	*
B21-1 (#30344)	3.5'	46.4	55.2/11.2/44.0	*	*	24.7
B21-4 (#30345)	16'	*	32.3/19.0/13.3	*	*	23.3
B7-1 (#30346)	5'	24.0	33.7/13.2/20.5	*	*	5.8
B7-2 (#30347)	6'-8'	27.9	*	*	*	6.9
B8-1 (#30348)	5'	21.4	*	*	*	11.9
B8-3 (#30349)	10'	17.6	*	*	*	5.7
B9-1 (#30350)	5'	*	*	*	*	7.1
B9-2 (#30351)	10'	*	*	*	*	13.1
B9-4 (#30352)	15'	*	*	*	*	15.7
B10-1 (#30353)	5'	14.7	19.2/25.7/6.5	*	*	11.6
B10-4 (#30354)	20'	*	*	*	*	15.7
B11-1 (#30355)	5'	25.4	29.2/17.6/21.6	*	*	10.9
B12-1 (#30356)	5'	*	*	*	*	3.2
B12-2 (#30357)	7'-8'	28.7	*	*	*	14.8
B13-1 (#30358)	5'	*	*	*	*	15.1
B13-3 (#30359)	10'	*	33.2/14.9/18.3	*	122.9	4.8
B13-4 (#30360)	13'	47.8	*	*	*	14.0
TB-2A-1 (#30361)	5'	*	39.8/14.3/25.5	*	*	11.4
TB-2A-2 (#30362)	10'	22.4	*	*	*	*
TB-2A-3 (#30363)	15'	*	27.6/21.0/6.6	*	*	6.0

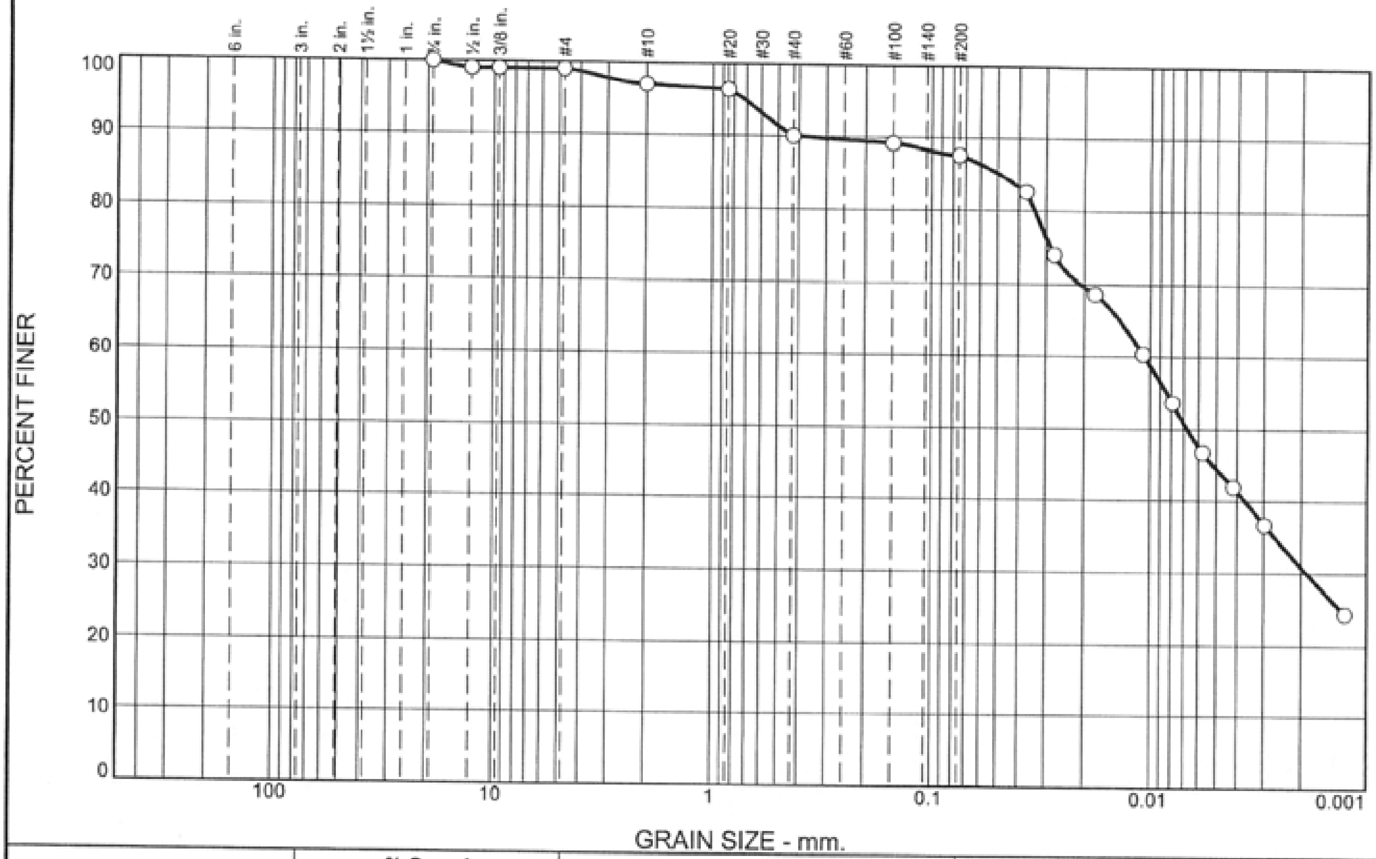
Sample I.D.	Depth (ft.)	-#200 Wash, % Loss ASTM D1140	Liquid and Plastic Limit ASTM D43189	Organic Impurity ASTM C40	Dry Density (pcf)	Moisture Content (%), as received ASTM D 2216
TB-2A-4 (#30364)	20'	32.2	*	*	*	4.2
B16-1 (#30365)	4'	19.0	*	*	*	*
B16-2 (#30366)	5'	24.6	*	*	*	11.5
B16-3 (#30367)	10'	17.9	*	*	*	9.2
B17-1 (#30368)	5'	27.4	30.6/22.3/8.3	*	*	13.0
B17-3 (#30369)	18'	*	*	*	*	3.5
B18-3 (#30370)	15'	28.1	*	*	*	9.1
B18-4 (#30371)	21'	*	*	*	*	8.5
B19-1 (#30372)	5'	*	*	*	*	10.9
B19-4 (#30373)	15'	40.3	*	*	*	11.8
B20-3 (#30375)	15'	*	*	*	*	12.0
B20-a1 to a5 (#30325)	2-18.5'	9.9	45.4/18.7/26.7	*	*	13.4
TB-4b (#30322)	77'	4.4	*	*	*	*
TB-4c (#30323)	86'	10.1	*	*	*	*
TB-4d (#30324)	93'	5.9	*	*	*	*

NOTE: *Indicates test not requested
TerraCosta Consulting Inc./ M. Eckert

Amec Foster Wheeler

Reviewed By:________________________
David C. Wilson, CE#54734
Senior Principal Engineer

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	2.0	7.0	2.4	56.7	30.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	99.0		
0.375"	99.0		
#4	99.0		
#10	97.0		
#20	96.4		
#40	90.0		
#100	89.2		
#200	87.6		

* (no specification provided)

Material Description

CL, (#30326)

Atterberg Limits

PL= 21.0 LL= 46.8 PI= 25.8

Coefficients

D_{90}= 0.4250 D_{85}= 0.0489 D_{60}= 0.0106
D_{50}= 0.0068 D_{30}= 0.0019 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(24)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B1-1
Depth: 5'

Date: 2/10/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project No: 5015-15-0030.21 **Figure** #30326

Tested By: Woodard/Gibson **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

2/21/2017

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project Number: 5015-15-0030.21
Location: B1-1
Depth: 5'
Material Description: CL, (#30326)
Date: 2/10/17 **PL:** 21.0 **LL:** 46.8 **PI:** 25.8
USCS Classification: CL **AASHTO Classification:** A-7-6(24)
Testing Remarks: Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.
Tested by: Woodard/Gibson **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.75"	100.0
0.5"	99.0
0.375"	99.0
#4	99.0
#10	97.0
#20	96.4
#40	90.0
#100	89.2
#200	87.6

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 97.0
Weight of hydrometer sample =56.61
Hygroscopic moisture correction:
Moist weight and tare = 24.77
Dry weight and tare = 24.58
Tare weight = 14.63
Hygroscopic moisture = 1.9%
Table of composite correction values:

Temp., deg. C:	20.3	21.6	22.5	23.3
Comp. corr.:	-4.0	-4.0	-5.0	-5.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - .164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	22.2	52.0	47.3	0.0133	52.0	7.8	0.0370	82.7
2.00	22.2	47.0	42.3	0.0133	47.0	8.6	0.0275	73.9
5.00	22.2	44.0	39.3	0.0133	44.0	9.1	0.0179	68.7
15.00	22.0	39.0	34.6	0.0133	39.0	9.9	0.0108	60.3
30.00	21.9	35.0	30.7	0.0133	35.0	10.6	0.0079	53.6
60.00	21.8	31.0	26.8	0.0133	31.0	11.2	0.0058	46.8
120.00	21.6	28.0	24.0	0.0134	28.0	11.7	0.0042	41.9
250.00	21.2	25.0	21.0	0.0134	25.0	12.2	0.0030	36.7
1536.00	21.1	18.0	14.0	0.0135	18.0	13.3	0.0013	24.4

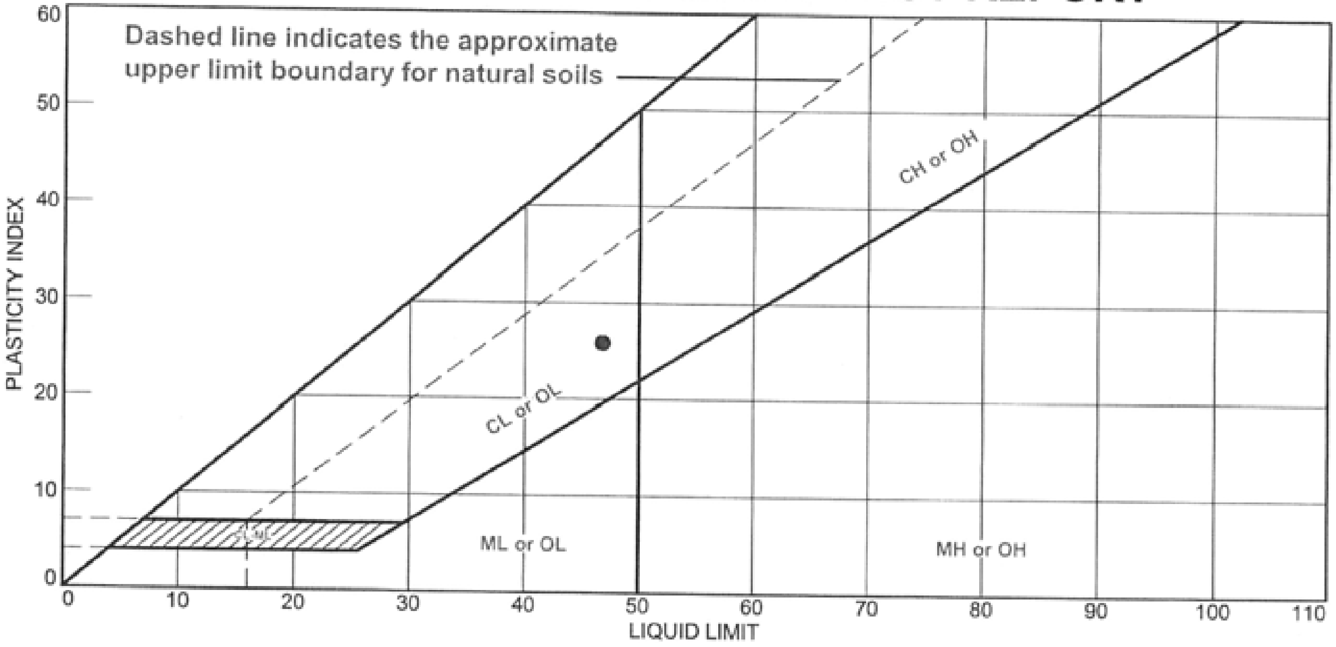
Fractional Components

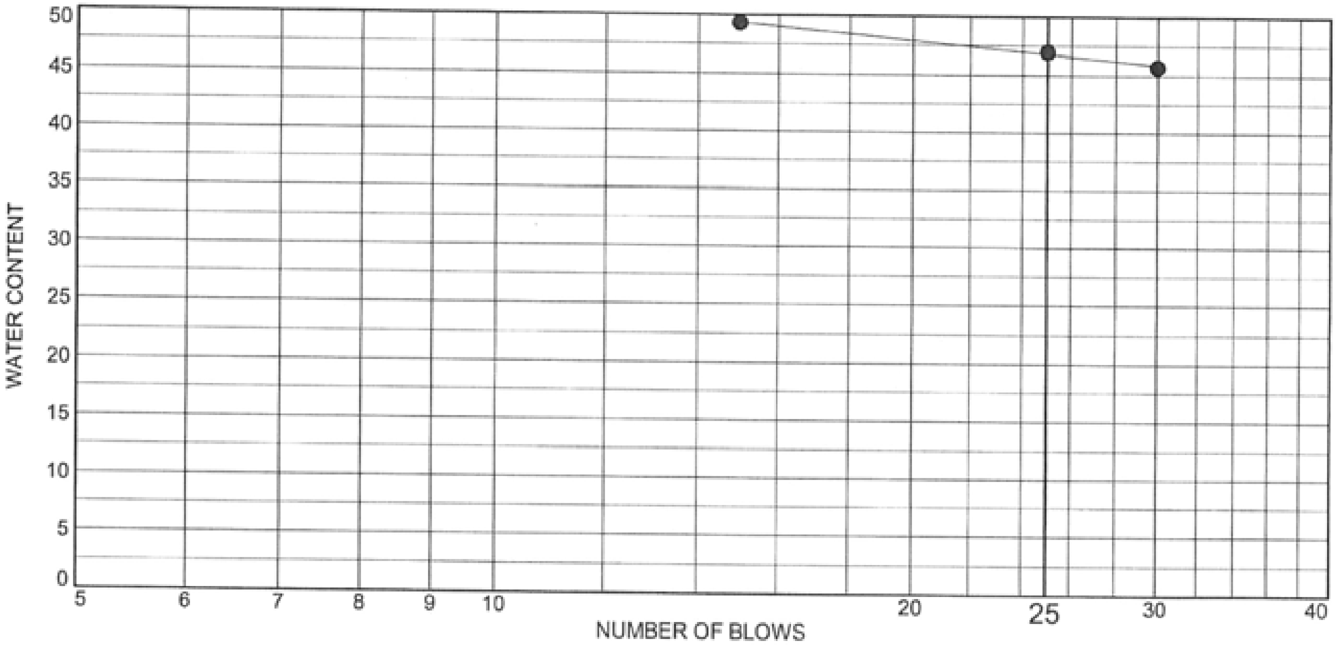
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.0	1.0	2.0	7.0	2.4	11.4	56.7	30.9	87.6

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.0019	0.0037	0.0068	0.0106	0.0338	0.0489	0.4250	0.7139

Fineness Modulus
0.36

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30326)	46.8	21.0	25.8	90.0	87.6	CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B1-1
Depth: 5'

Remarks:

amec

Figure #30326

Tested By: Iacovera **Checked By:** Collins

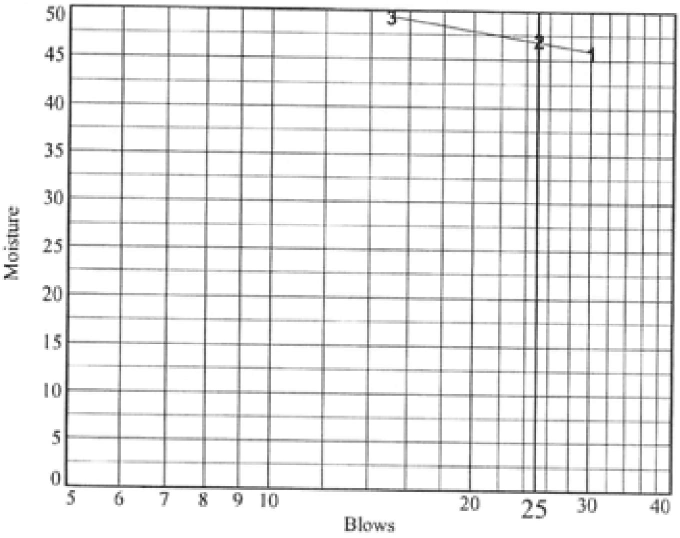
LIQUID AND PLASTIC LIMIT TEST DATA

2/21/2017

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project Number: 5015-15-0030.21
Location: B1-1
Depth: 5'
Material Description: CL, (#30326)
%<#40: 90.0 **%<#200:** 87.6 **USCS:** CL **AASHTO:** A-7-6(24)
Tested by: Iacovera **Checked by:** Collins

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	18.21	17.32	17.33			
Dry+Tare	15.96	15.3	15.24			
Tare	11.04	11.00	11			
# Blows	30	25	15			
Moisture	45.7	47.0	49.3			



Liquid Limit= 46.8
Plastic Limit= 21.0
Plasticity Index= 25.8

Plastic Limit Data

Run No.	1	2	3	4	
Wet+Tare	18.42	18.12			
Dry+Tare	17.72	17.50			
Tare	14.58	14.35			
Moisture	22.3	19.7			

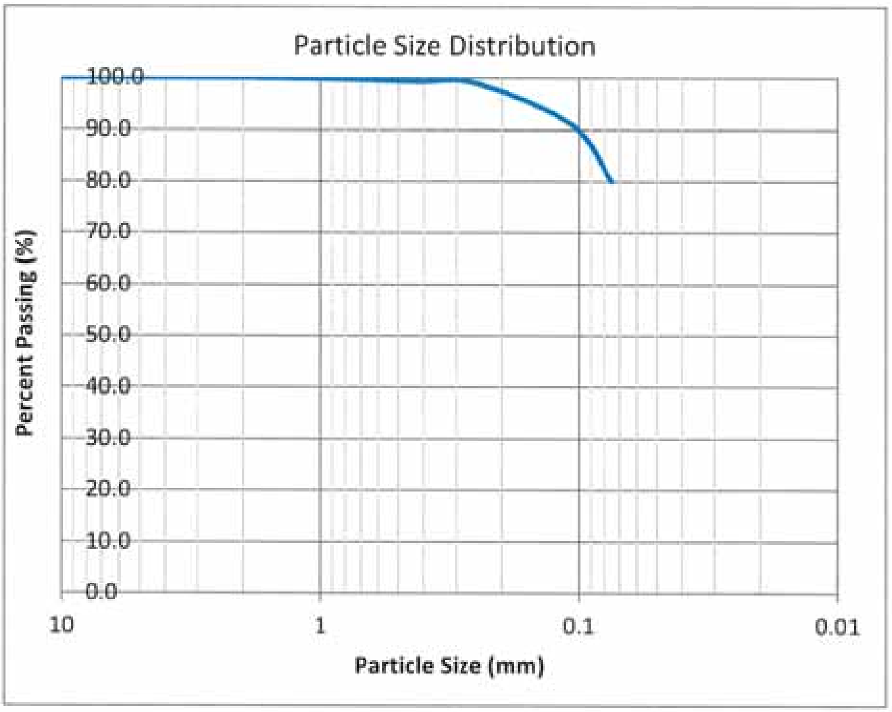
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B1-2 @ 8-9'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.8
No. 40	0.425	99.4
No. 60	0.25	98.9
No. 140	0.106	90.9
No. 200	0.075	80.1
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



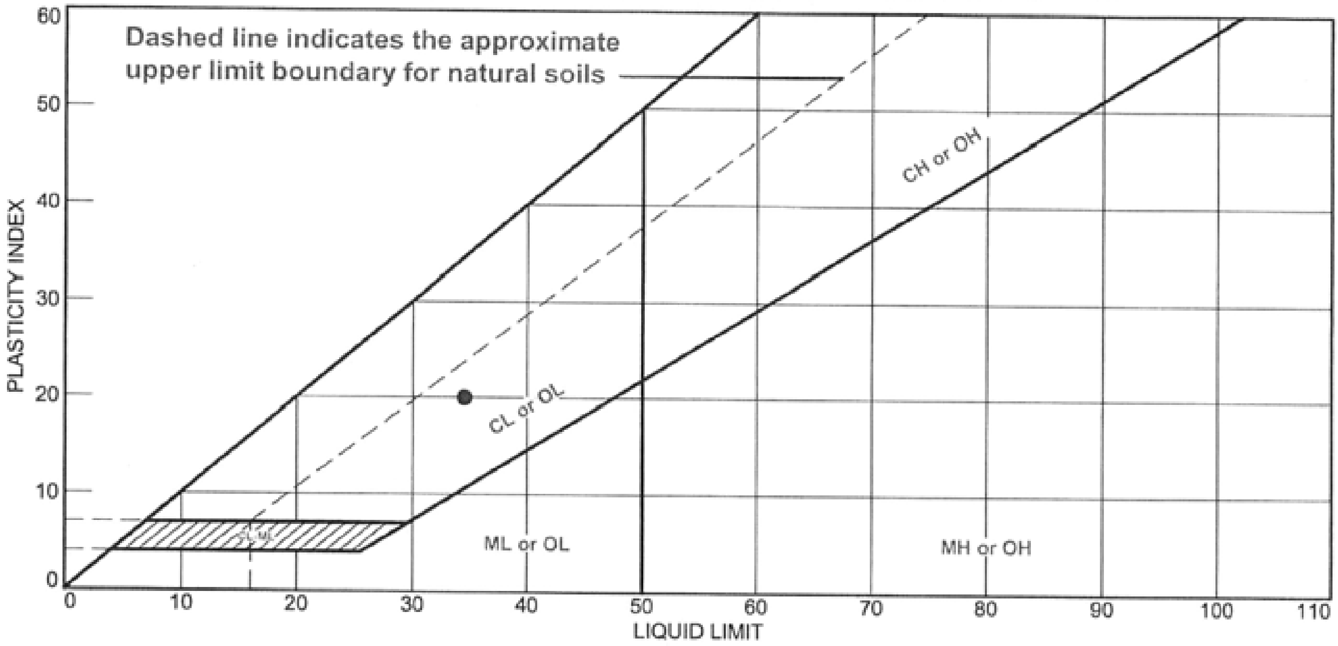
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	0.6
Fine Sand (<No. 40-No. 200):	19.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

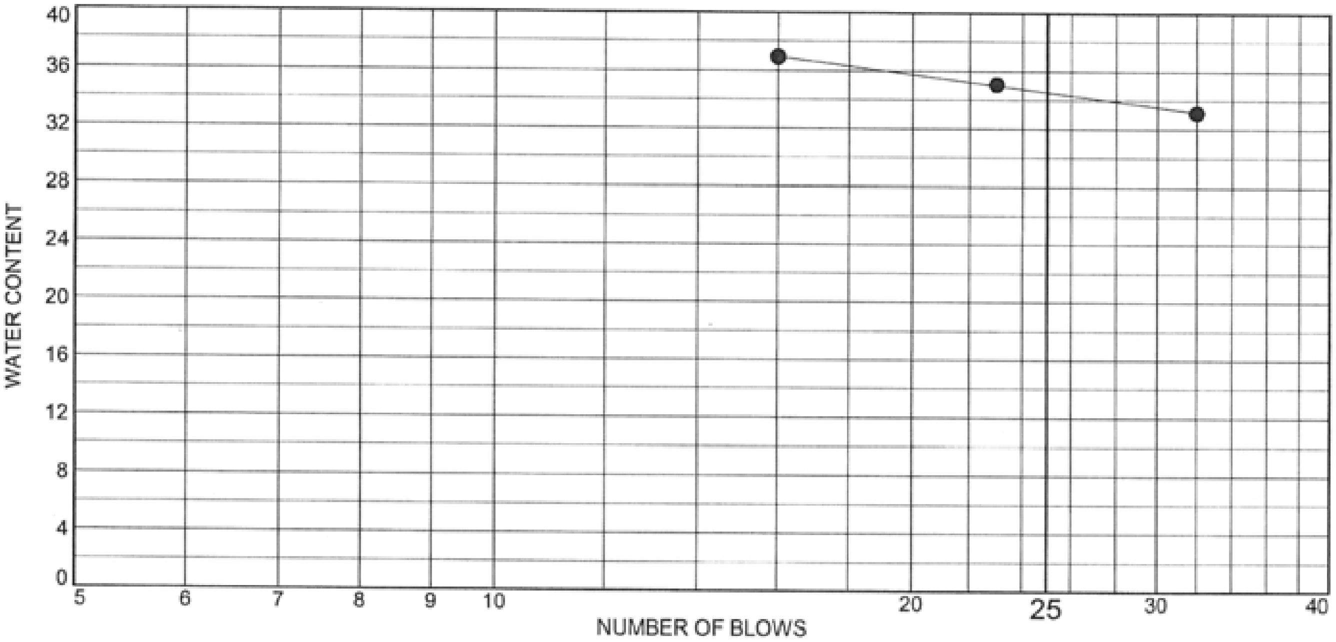
Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	19.9	80.1

LIQUID AND PLASTIC LIMITS TEST REPORT



Dashed line indicates the approximate upper limit boundary for natural soils



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30327)	34.5	14.5	20.0			

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B1-3
Depth: 10'

Remarks:

Figure #30327

Tested By: Iacovera **Checked By:** Collins

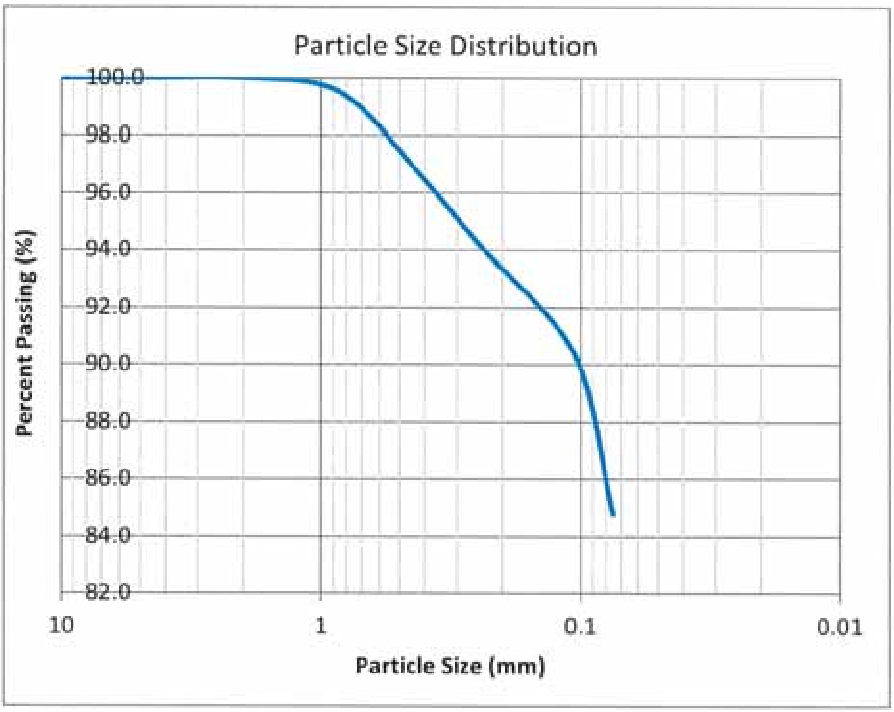
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B1-5 @ 18-19.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.5
No. 40	0.425	96.7
No. 60	0.25	94.3
No. 140	0.106	90.4
No. 200	0.075	84.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



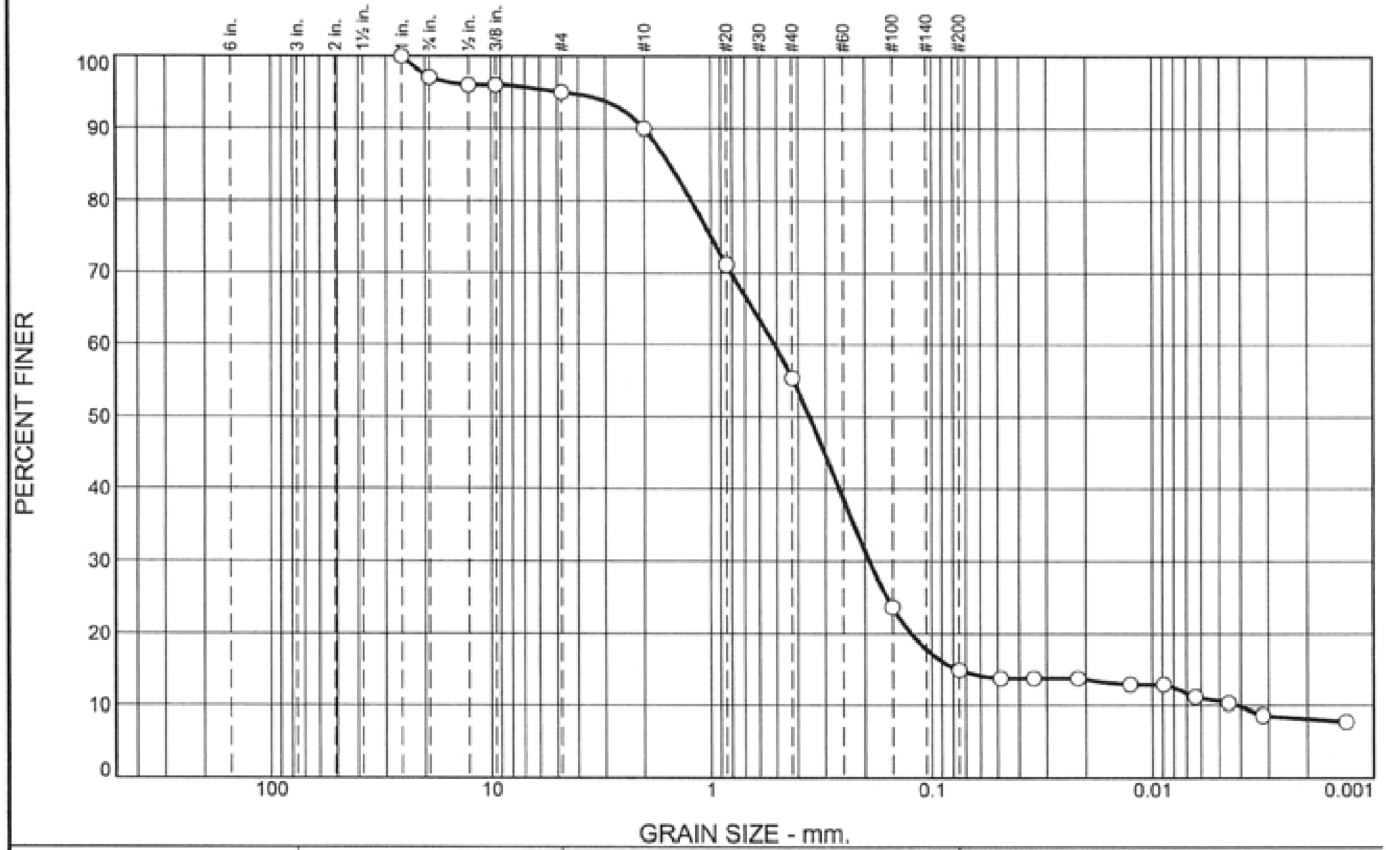
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	3.3
Fine Sand (<No. 40-No. 200):	12.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	15.2	84.8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.0	2.0	5.0	34.7	40.4	6.8	8.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	97.0		
0.5"	96.0		
0.375"	96.0		
#4	95.0		
#10	90.0		
#20	71.1		
#40	55.3		
#100	23.7		
#200	14.9		

* (no specification provided)

Material Description

SC-SM, (#30330)

Atterberg Limits

PL= 18.8 LL= 25.4 PI= 6.6

Coefficients

D_{90}= 2.0000 D_{85}= 1.5267 D_{60}= 0.5121
D_{50}= 0.3549 D_{30}= 0.1906 D_{15}= 0.0764
D_{10}= 0.0042 C_u= 121.67 C_c= 16.86

Classification

USCS= SC-SM AASHTO= A-2-4(0)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: TB-1a-1
Depth: 5'

Date: 2/6/17

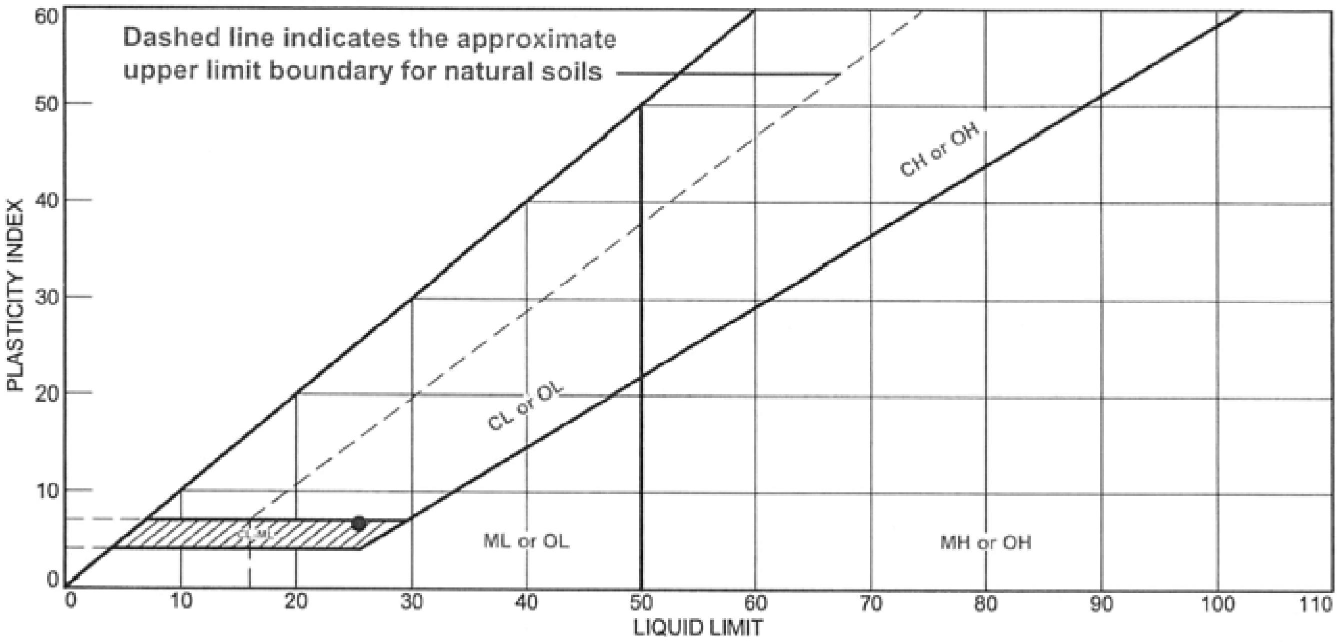
amec

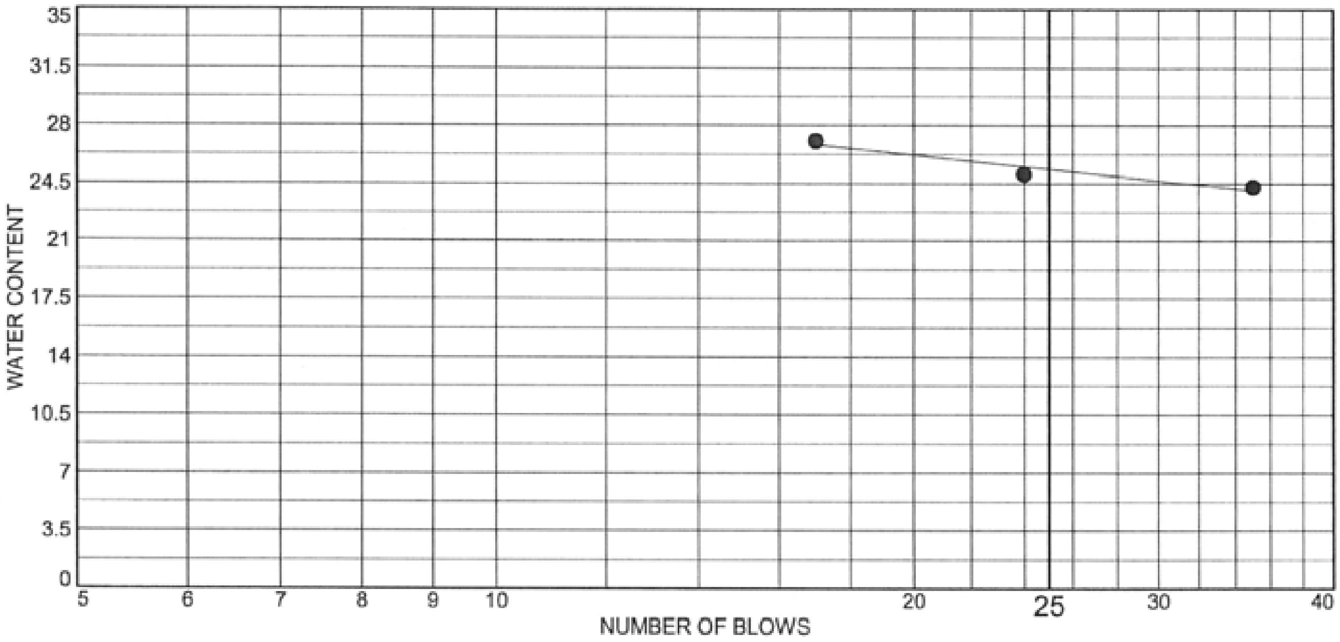
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30330

Tested By: Gibson/Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SC-SM, (#30330)	25.4	18.8	6.6	55.3	14.9	SC-SM

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

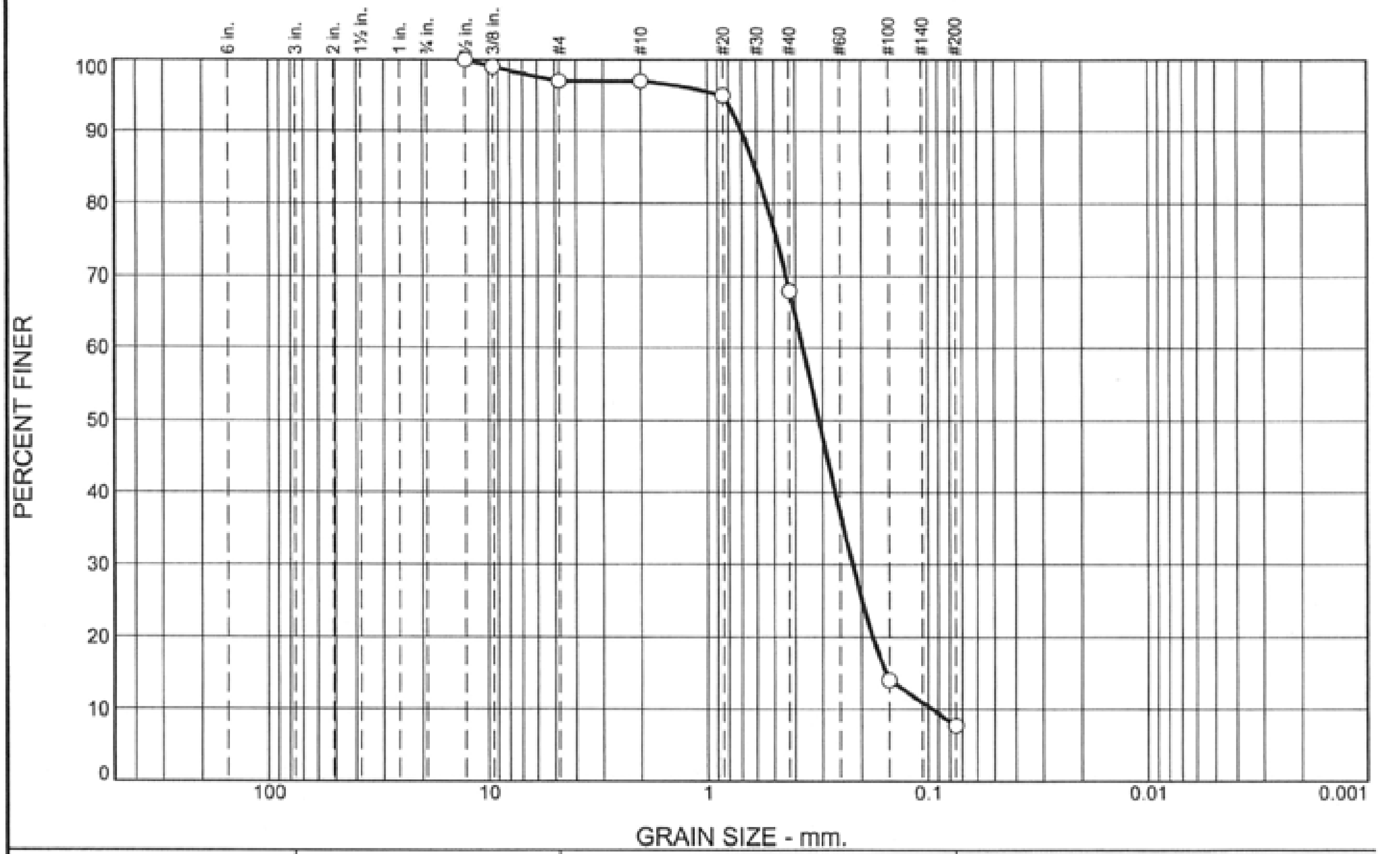
Location: TB-1a-1
Depth: 5'

Remarks:

Figure #30330

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.0	0.0	29.0	60.2	7.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.5"	100.0		
0.375"	99.0		
#4	97.0		
#10	97.0		
#20	95.0		
#40	68.0		
#100	14.0		
#200	7.8		

* (no specification provided)

Material Description

ML-OL, (#30333)

Atterberg Limits

PL= 21.5 LL= 23.4 PI= 1.9

Coefficients

D_{90}= 0.7001 D_{85}= 0.6082 D_{60}= 0.3690
D_{50}= 0.3117 D_{30}= 0.2198 D_{15}= 0.1551
D_{10}= 0.0959 C_u= 3.85 C_c= 1.37

Classification

USCS= ML-OL AASHTO=

Remarks

Location: TB-1a-5@25' and TB-1a-6@30'
Depth: 25'-30'

Date: 2/6/17

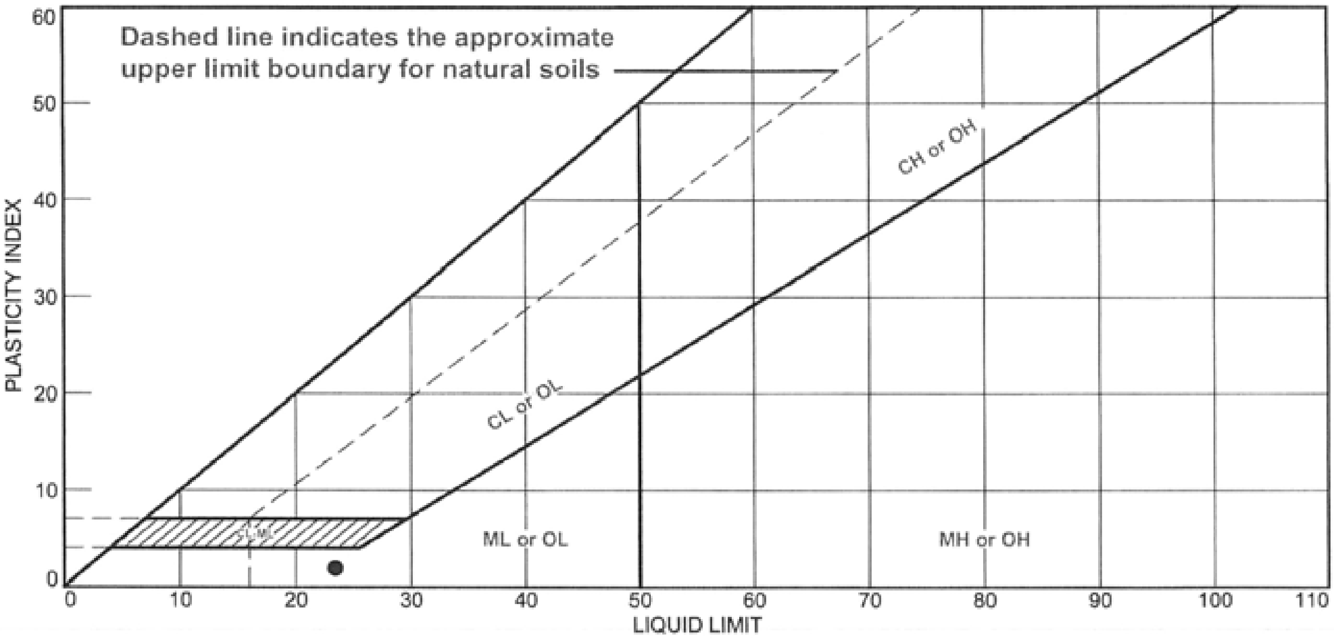
amec

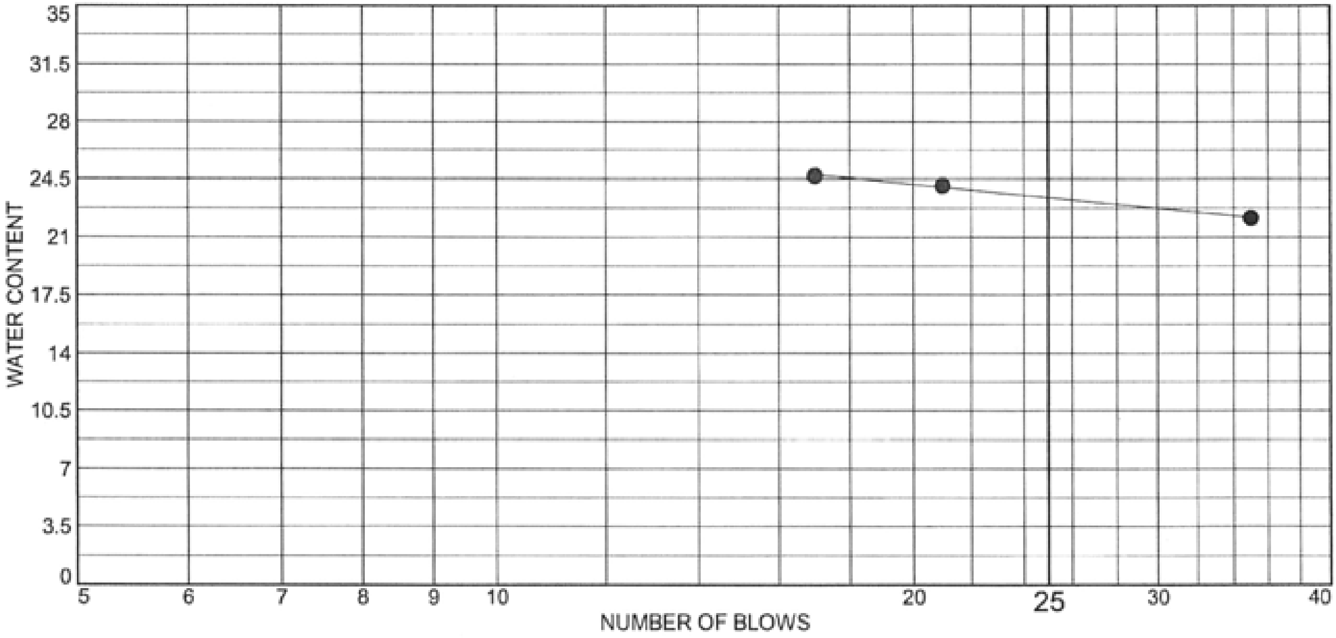
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30333

Tested By: Iacovera/Gibson **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	ML-OL, (#30333)	23.4	21.5	1.9	68.0	7.8	ML-OL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: TB-1a-5@25' and TB-1a-6@30'
Depth: 25'-30'

Remarks:

Figure #30333

Tested By: Iacovera **Checked By:** Collins

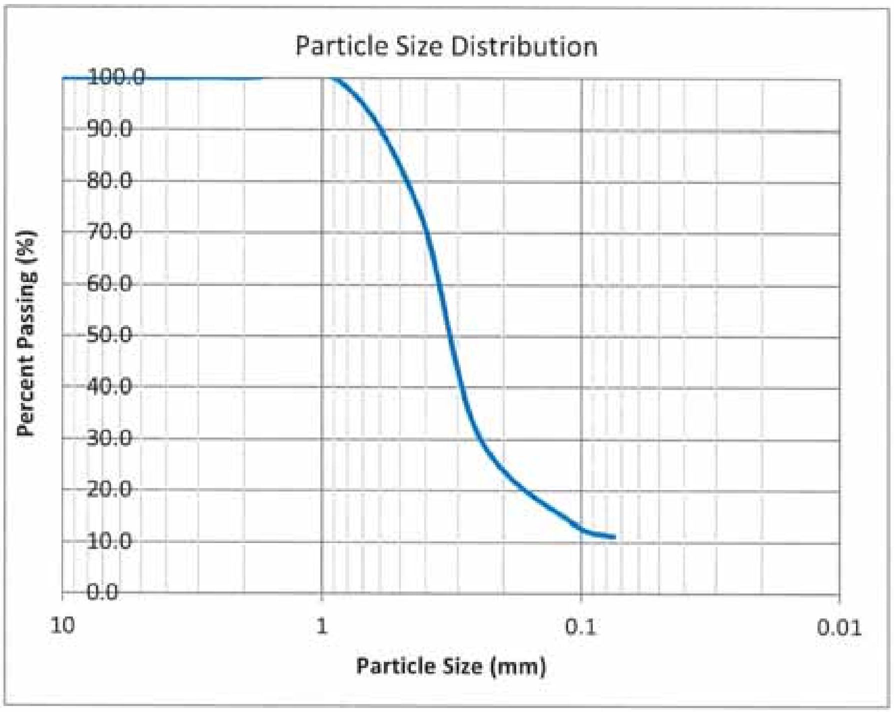
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

TB-1a-5 @ 20-21'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.2
No. 40	0.425	74.6
No. 60	0.25	30.9
No. 140	0.106	13.5
No. 200	0.075	11.2
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



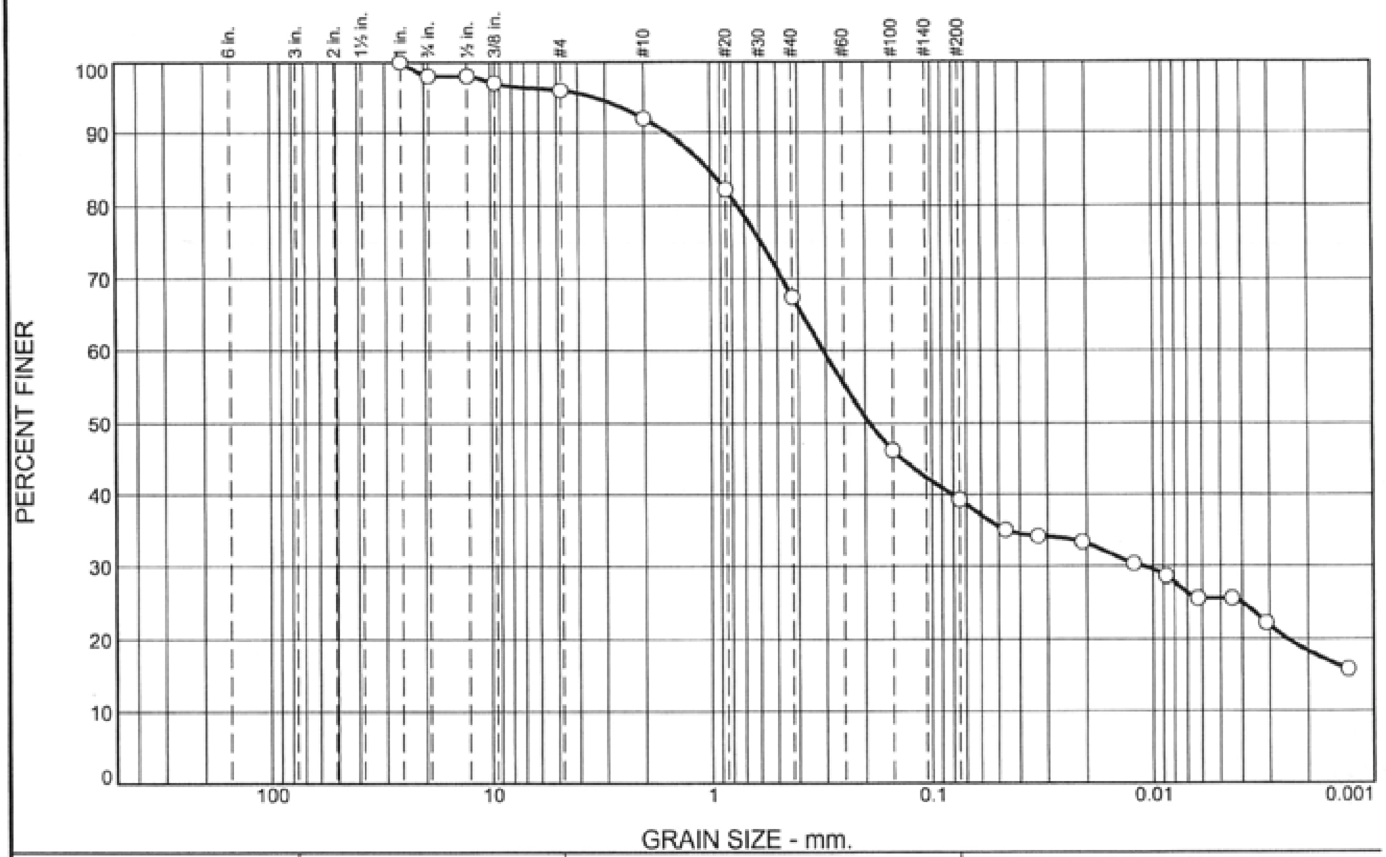
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	25.4
Fine Sand (<No. 40-No. 200):	63.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	88.8	11.2

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.0	2.0	4.0	24.5	28.2	21.1	18.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	98.0		
0.5"	98.0		
0.375"	97.0		
#4	96.0		
#10	92.0		
#20	82.2		
#40	67.5		
#100	46.2		
#200	39.3		

* (no specification provided)

Material Description

SC-SM, (#30334)

Atterberg Limits

PL= 19.5 LL= 25.3 PI= 5.8

Coefficients

D_{90}= 1.5676 D_{85}= 1.0162 D_{60}= 0.3082
D_{50}= 0.1905 D_{30}= 0.0113 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC-SM AASHTO= A-4(0)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: TB-1b-1
Depth: 5'

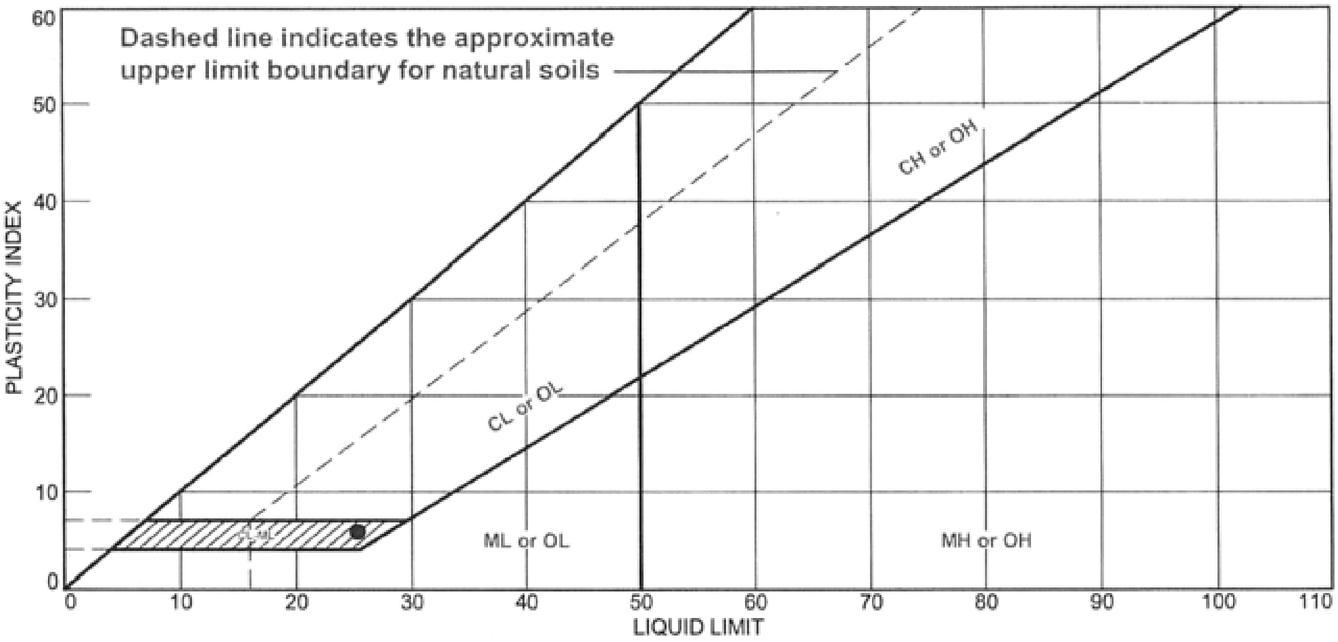
Date: 2/10/17

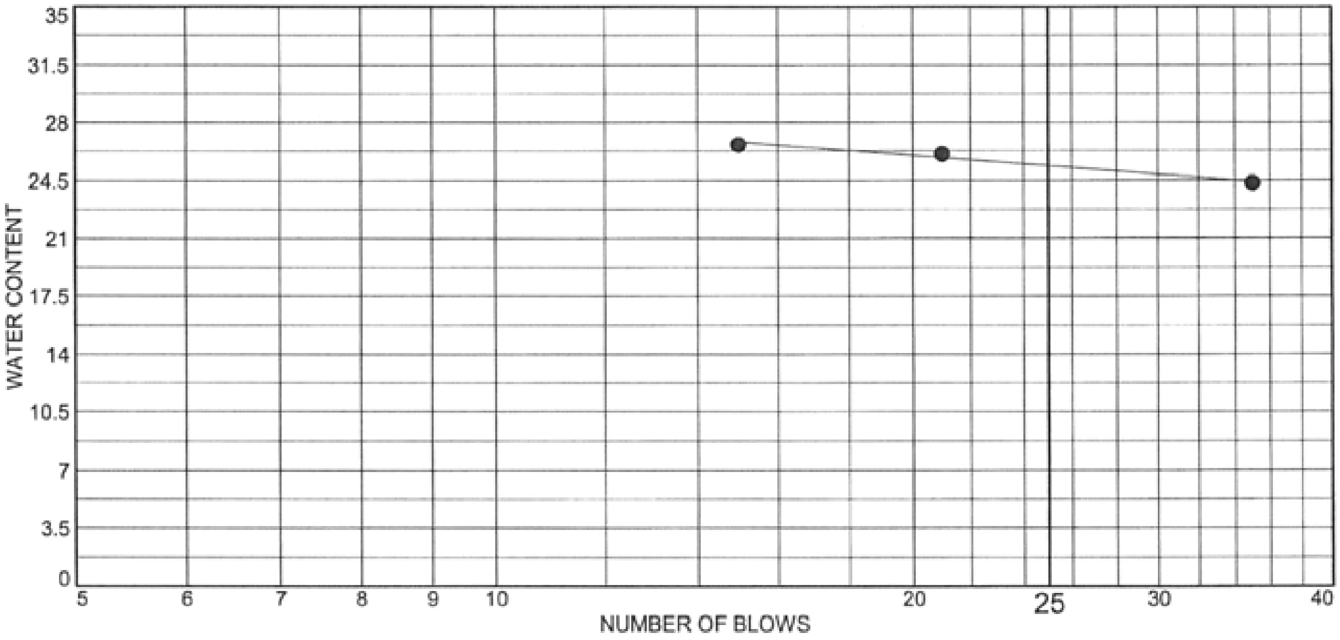
amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project No: 5015-15-0030.21 **Figure** #30334

Tested By: Gibson/Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SC-SM, (#30334)	25.3	19.5	5.8	67.5	39.3	SC-SM

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

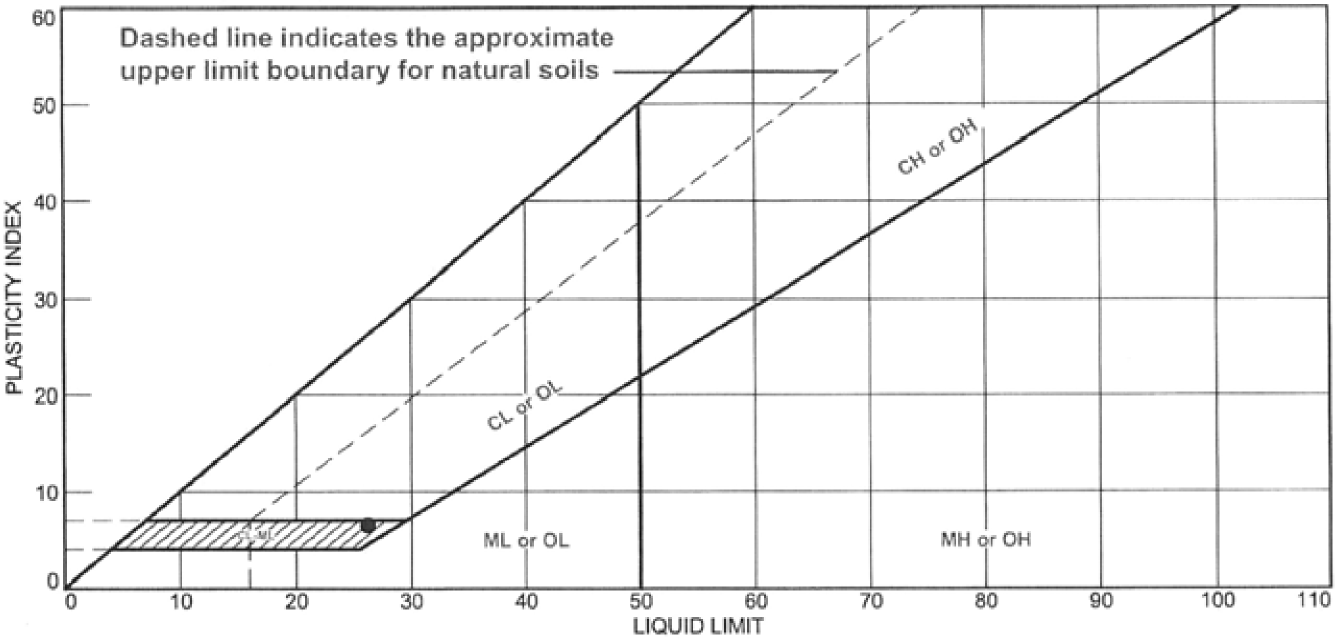
Location: TB-1b-1
Depth: 5'

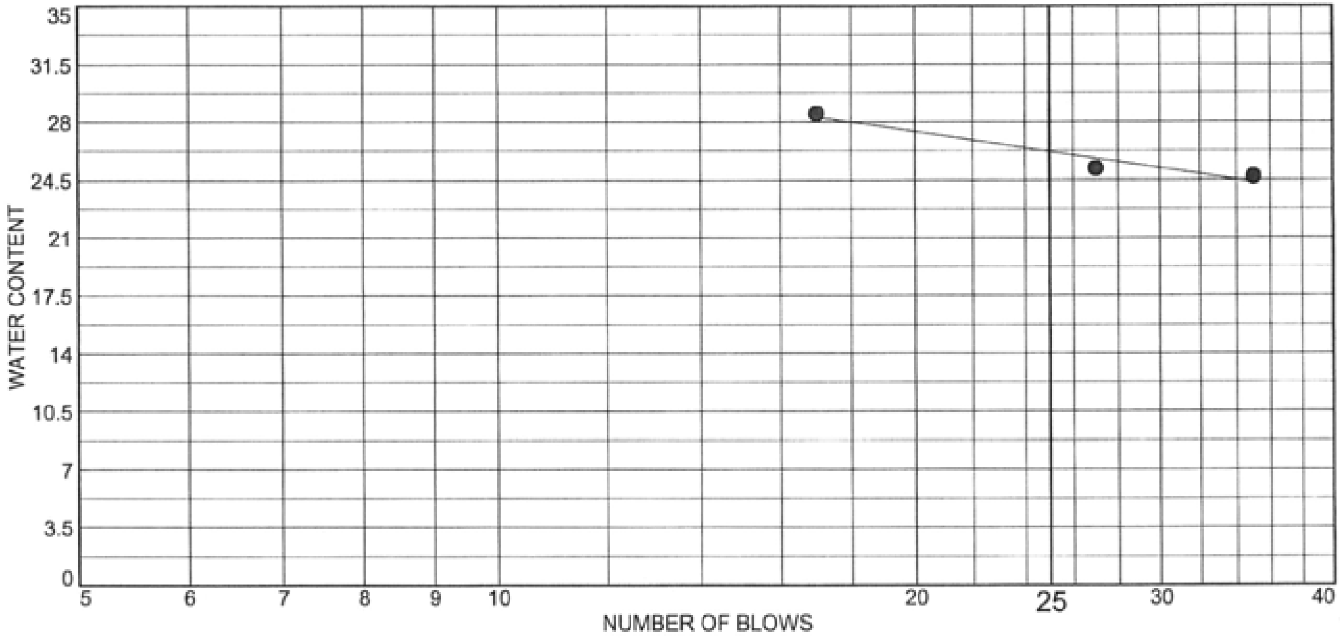
Remarks:

Figure #30334

Tested By: Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	(#30336)	26.2	19.7	6.5			CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

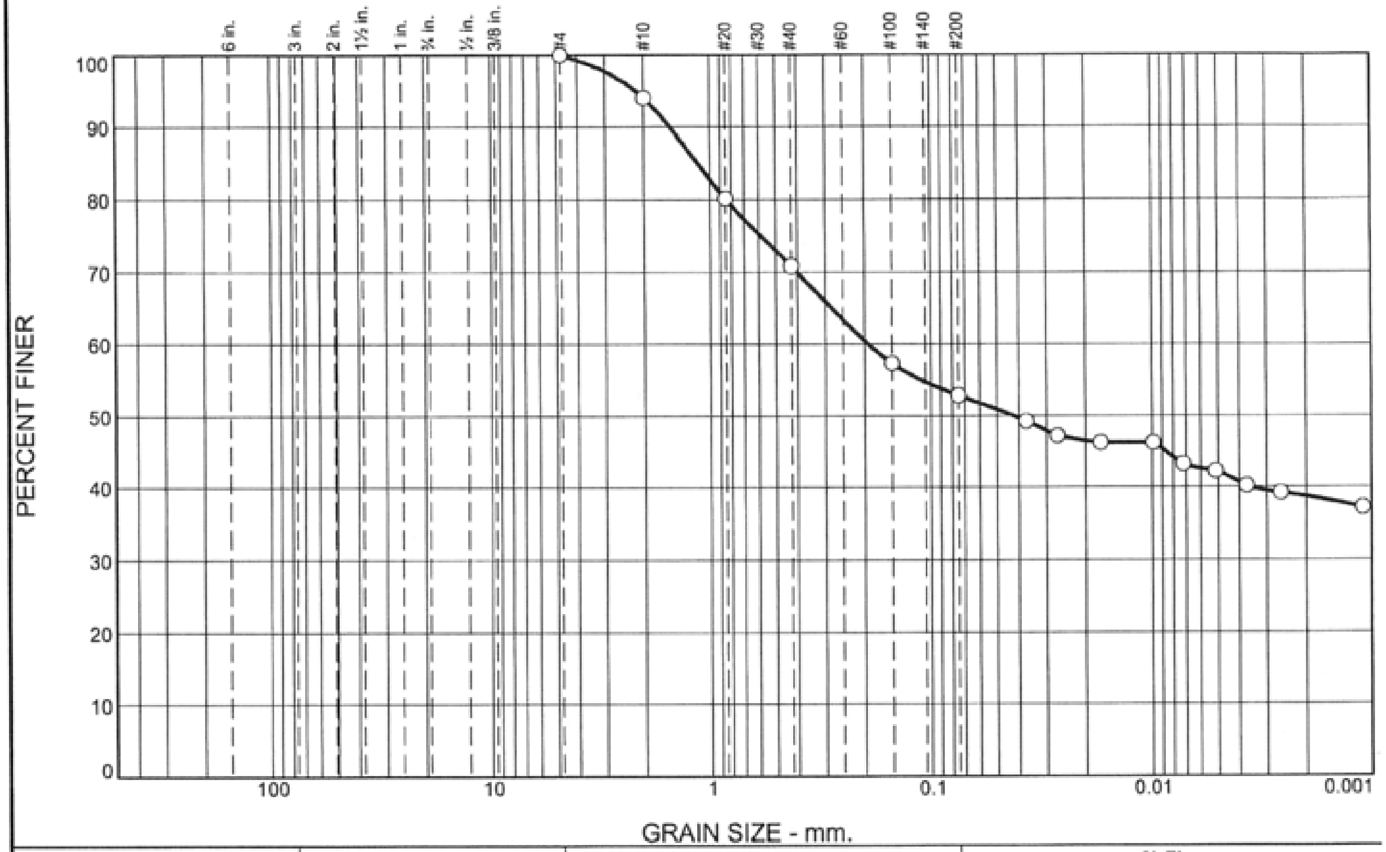
Location: TB-1b-3@15' & TB-1b-4@20'
Depth: 15'-20'

Remarks:

Figure #30336

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	6.0	23.2	18.0	14.1	38.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	94.0		
#20	80.1		
#40	70.8		
#100	57.3		
#200	52.8		

* (no specification provided)

Material Description

CH, (#30337)

Atterberg Limits

PL= 16.9 LL= 58.0 PI= 41.1

Coefficients

D_{90}= 1.5220 D_{85}= 1.1379 D_{60}= 0.1921
D_{50}= 0.0423 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(17)

Remarks

Location: B3-1
Depth: 5'

Date: 2/6/17

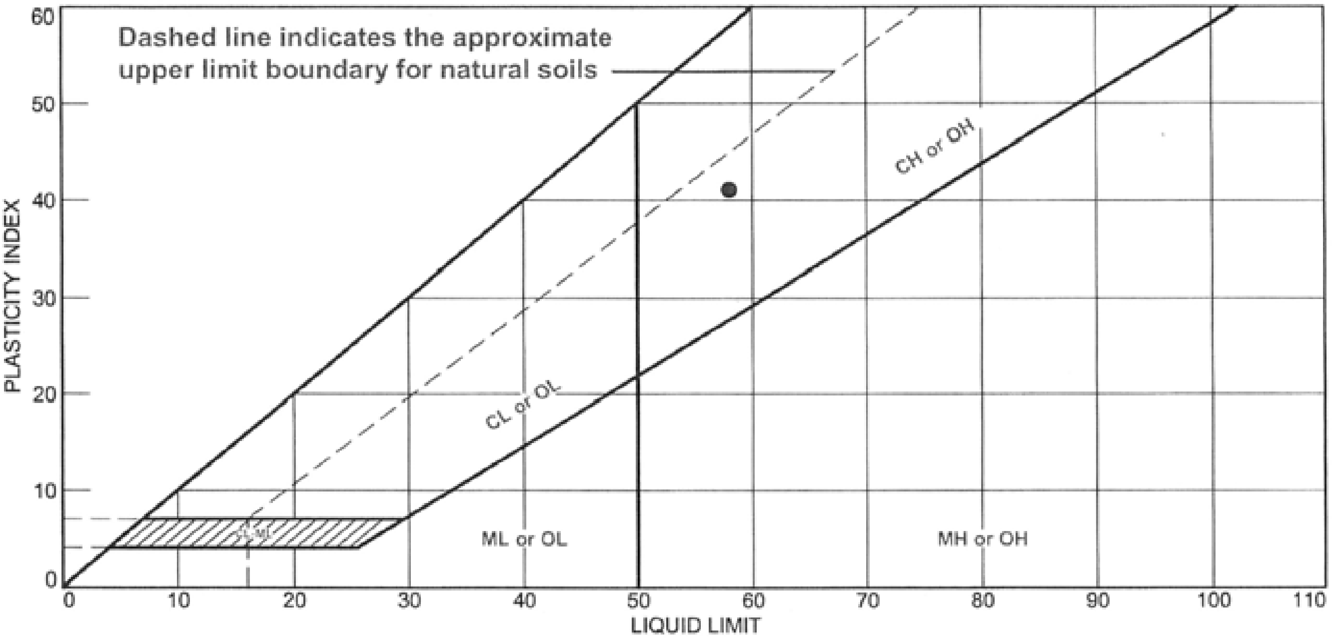
amec

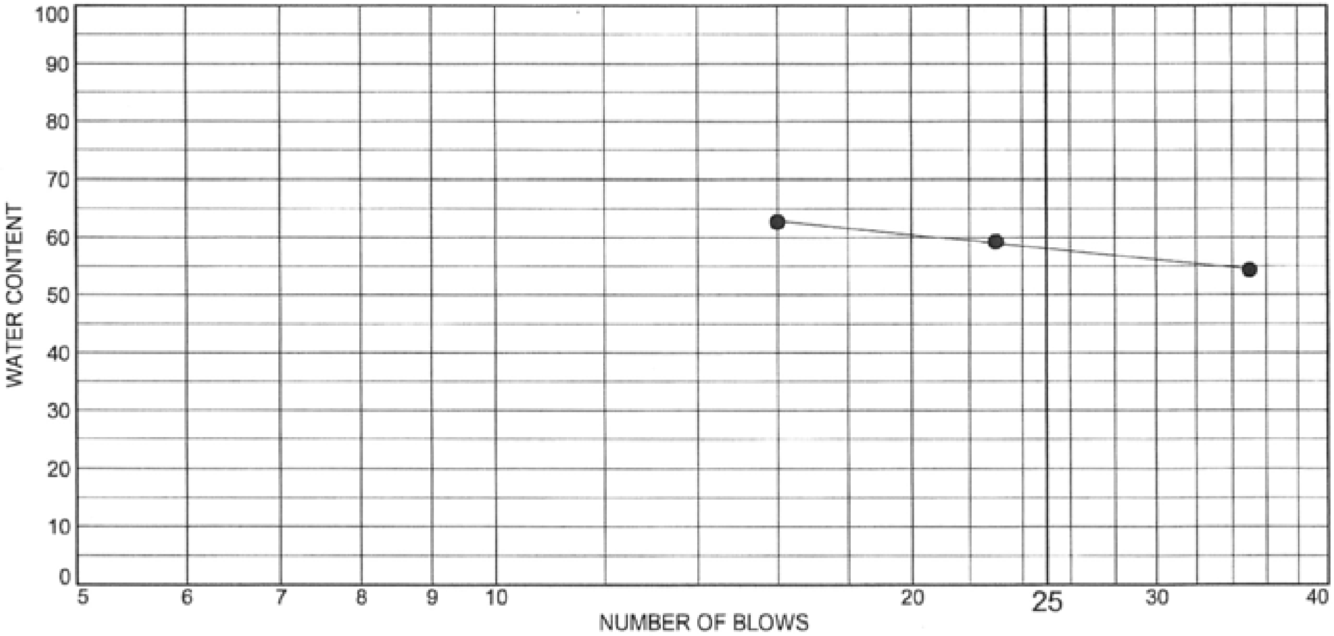
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30337

Tested By: Woodard/Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	(#30337)	58.0	16.9	41.1	70.8	52.8	CH

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

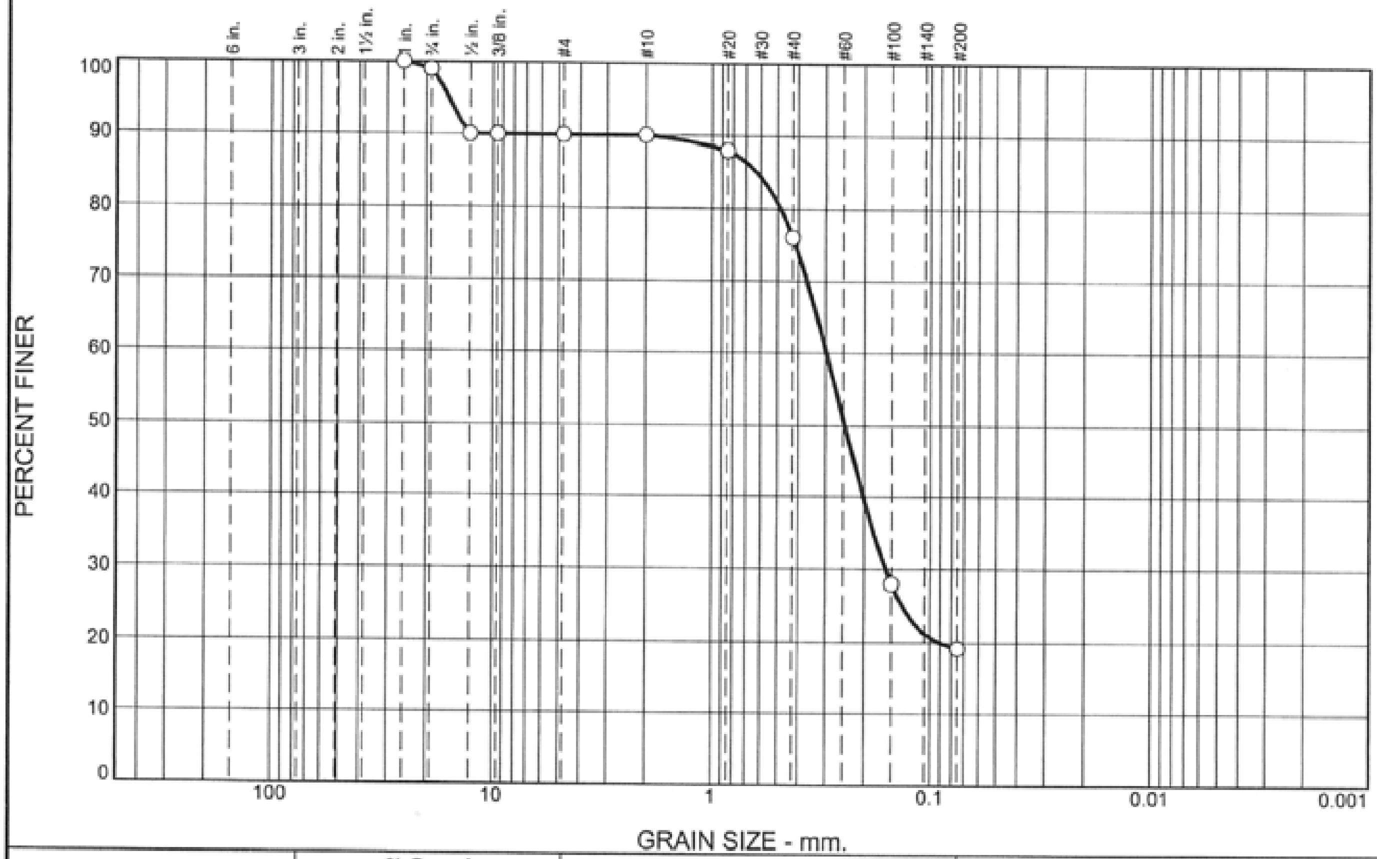
Location: B3-1
Depth: 5'

Remarks:

Figure #30337

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	1.0	9.0	0.0	14.0	56.8	19.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	99.0		
0.5"	90.0		
0.375"	90.0		
#4	90.0		
#10	90.0		
#20	88.0		
#40	76.0		
#100	28.0		
#200	19.2		

* (no specification provided)

Material Description

Poorly Graded Silty Sand, SM (#30339)

Atterberg Limits

PL=
LL=
PI=

Coefficients

D_{90}= 12.7000
D_{85}= 0.6212
D_{60}= 0.2973
D_{50}= 0.2456
D_{30}= 0.1593
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS= SM
AASHTO=

Remarks

Location: B3-3
Depth: 12.5'

Date: 2/2/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

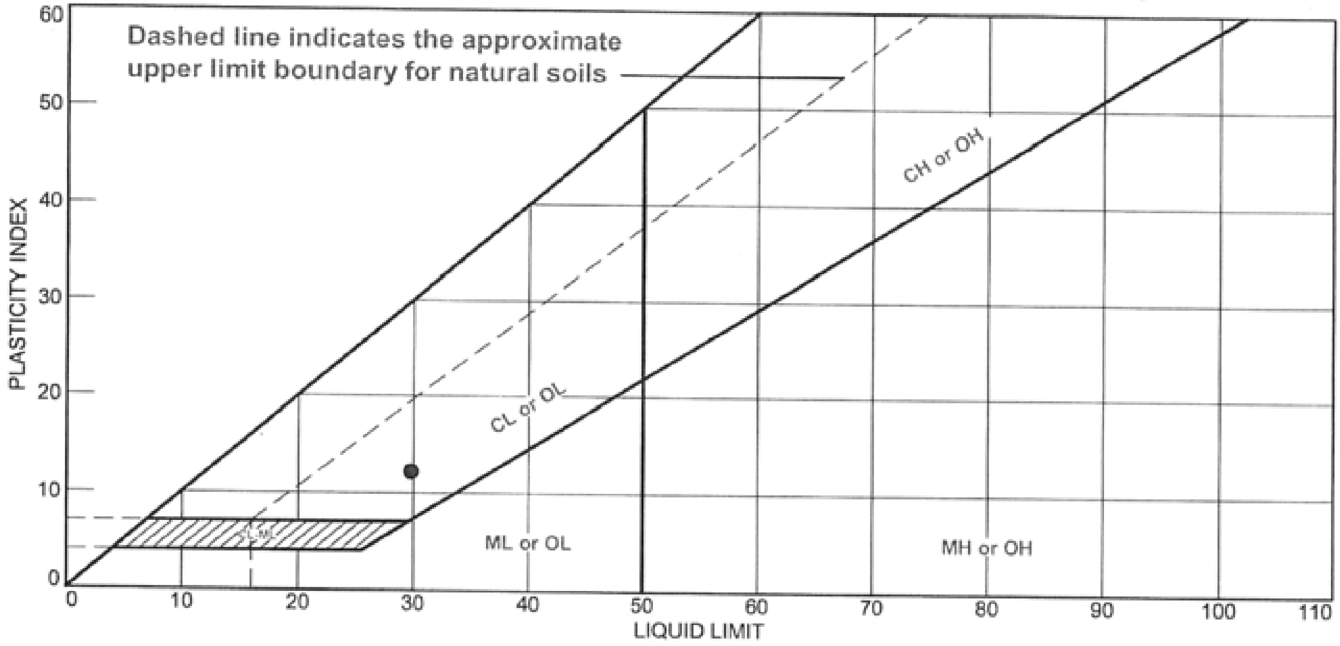
Project No: 5015-15-0030.21

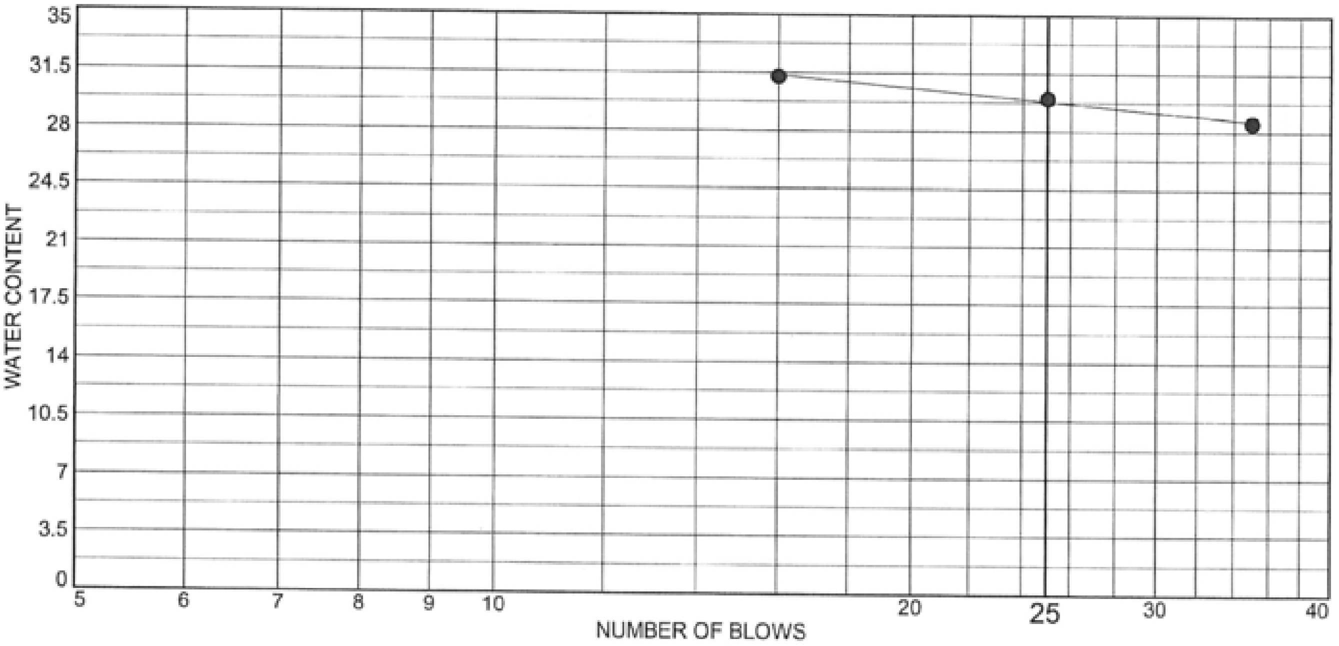
Figure

Tested By: Woodard

Checked By: Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Poorly Graded Clayey Sand, SC (#30339)	29.8	17.6	12.2	76.0	19.2	SC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B3-3
Depth: 12.5'

Remarks:

Figure #30339

Tested By: Iacovera **Checked By:** Collins

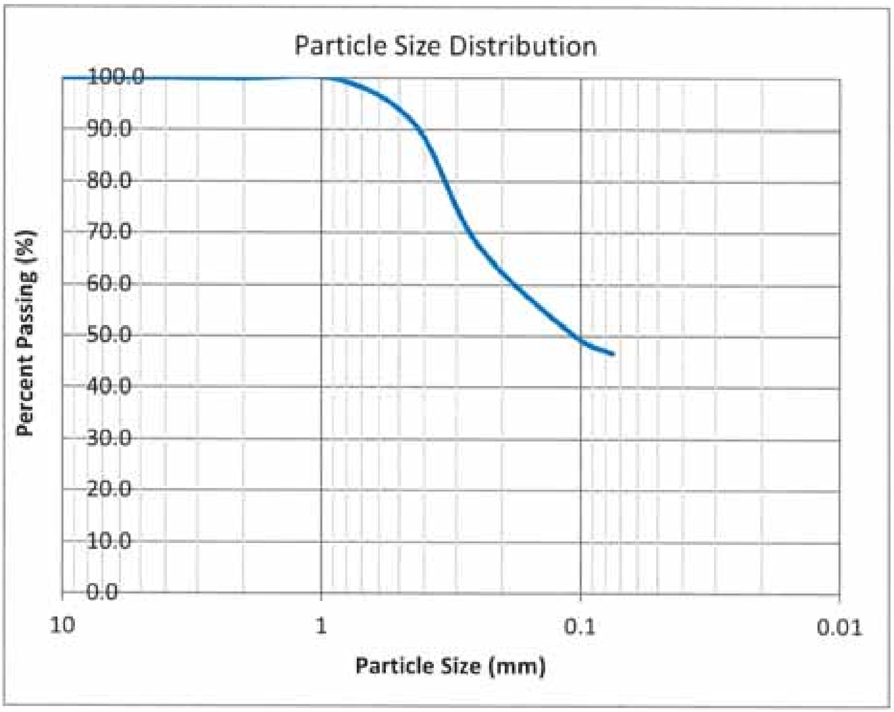
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B3-6 @ 23'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	99.9
No. 20	0.85	99.6
No. 40	0.425	90.4
No. 60	0.25	67.9
No. 140	0.106	50.1
No. 200	0.075	46.6
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



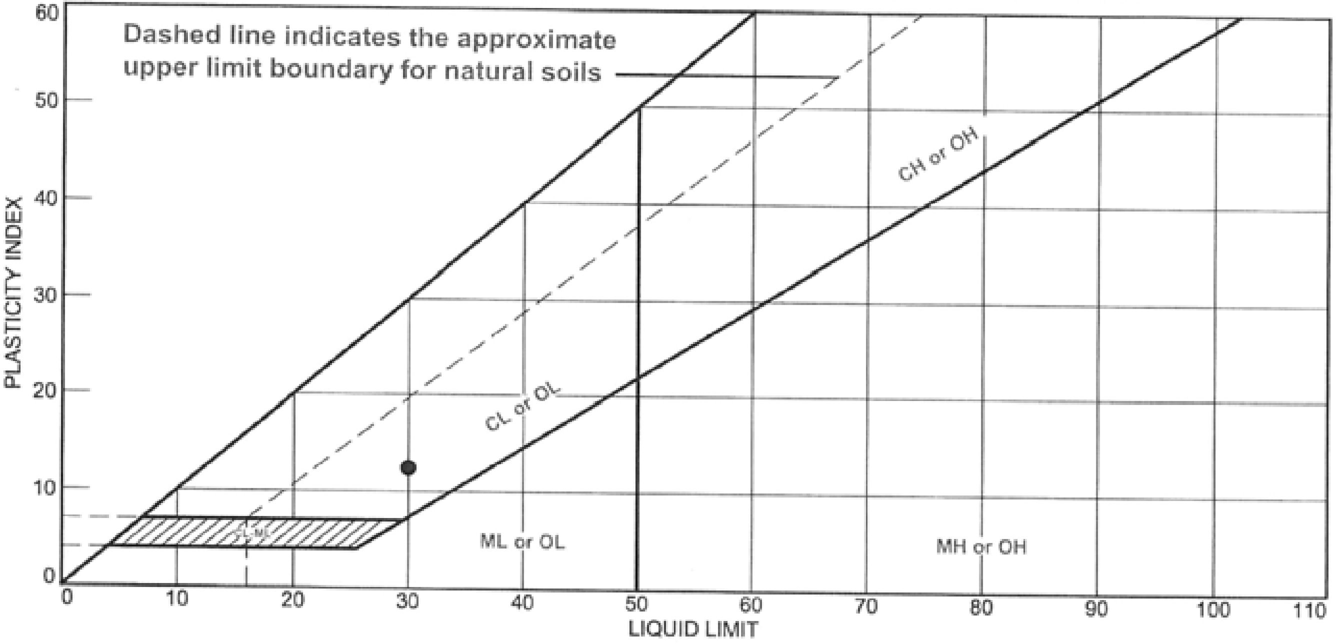
ASTM Composition of Total Sample per ASTM D422-63

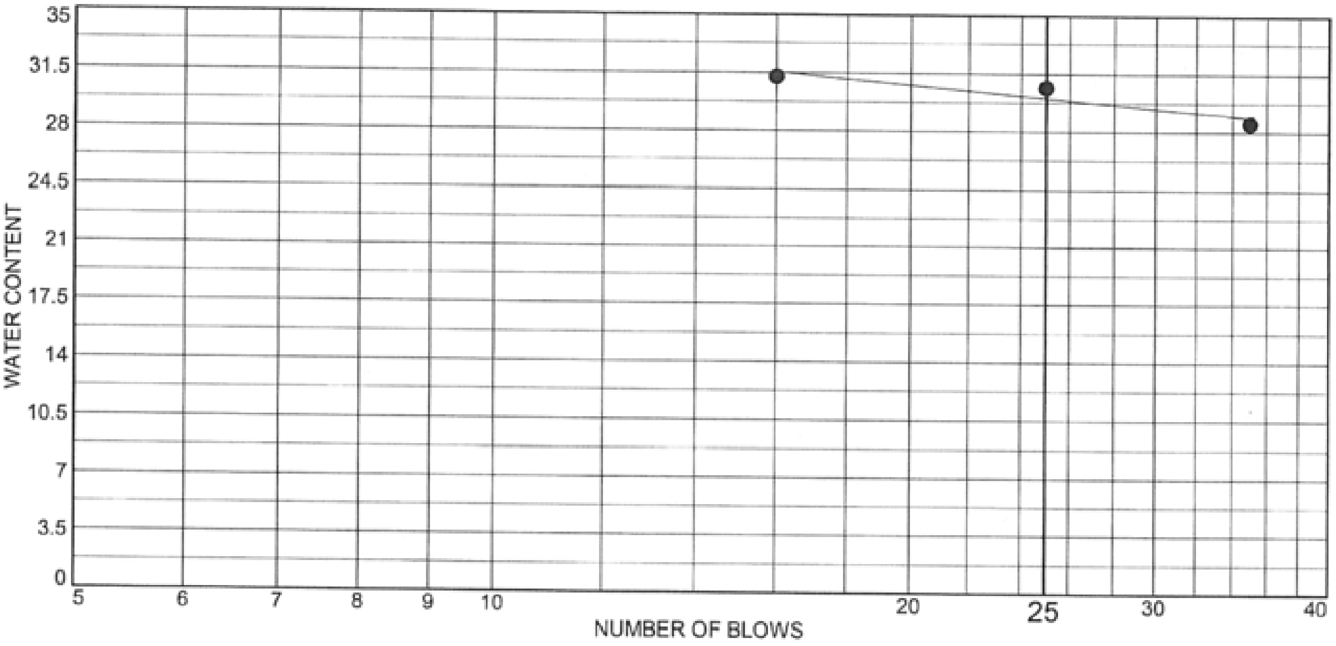
	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.1
Medium Sand (<No. 10-No. 40):	9.5
Fine Sand (<No. 40-No. 200):	43.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	53.4	46.6

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30340)	30.0	17.6	12.4			CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B4-2
Depth: 10'

Remarks:

Figure #30340

Tested By: Iacovera **Checked By:** Collins

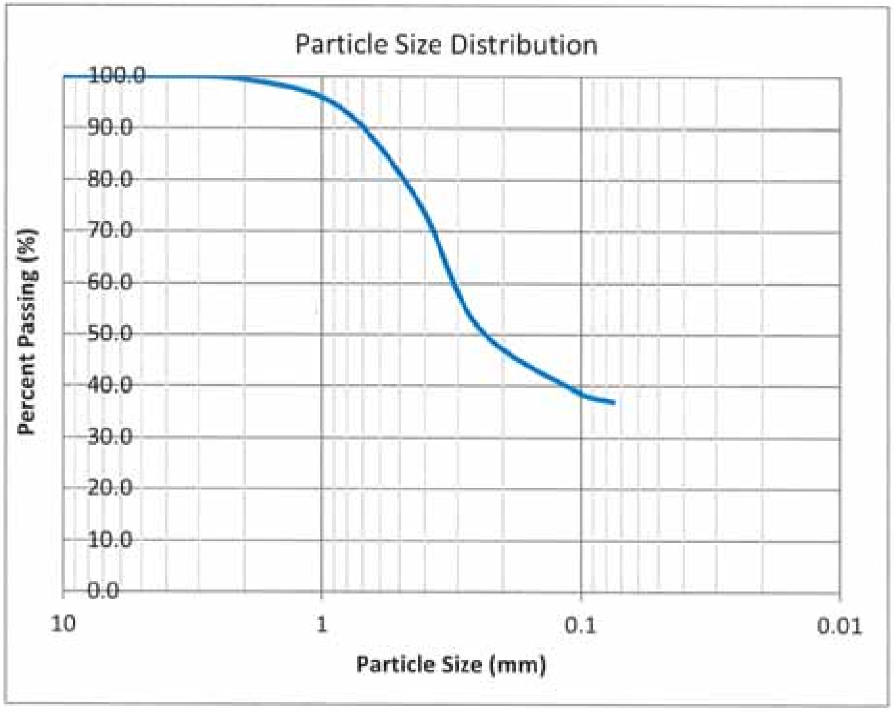
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B4-3 @ 15'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	99.4
No. 20	0.85	93.9
No. 40	0.425	75.8
No. 60	0.25	51.5
No. 140	0.106	39.2
No. 200	0.075	37.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



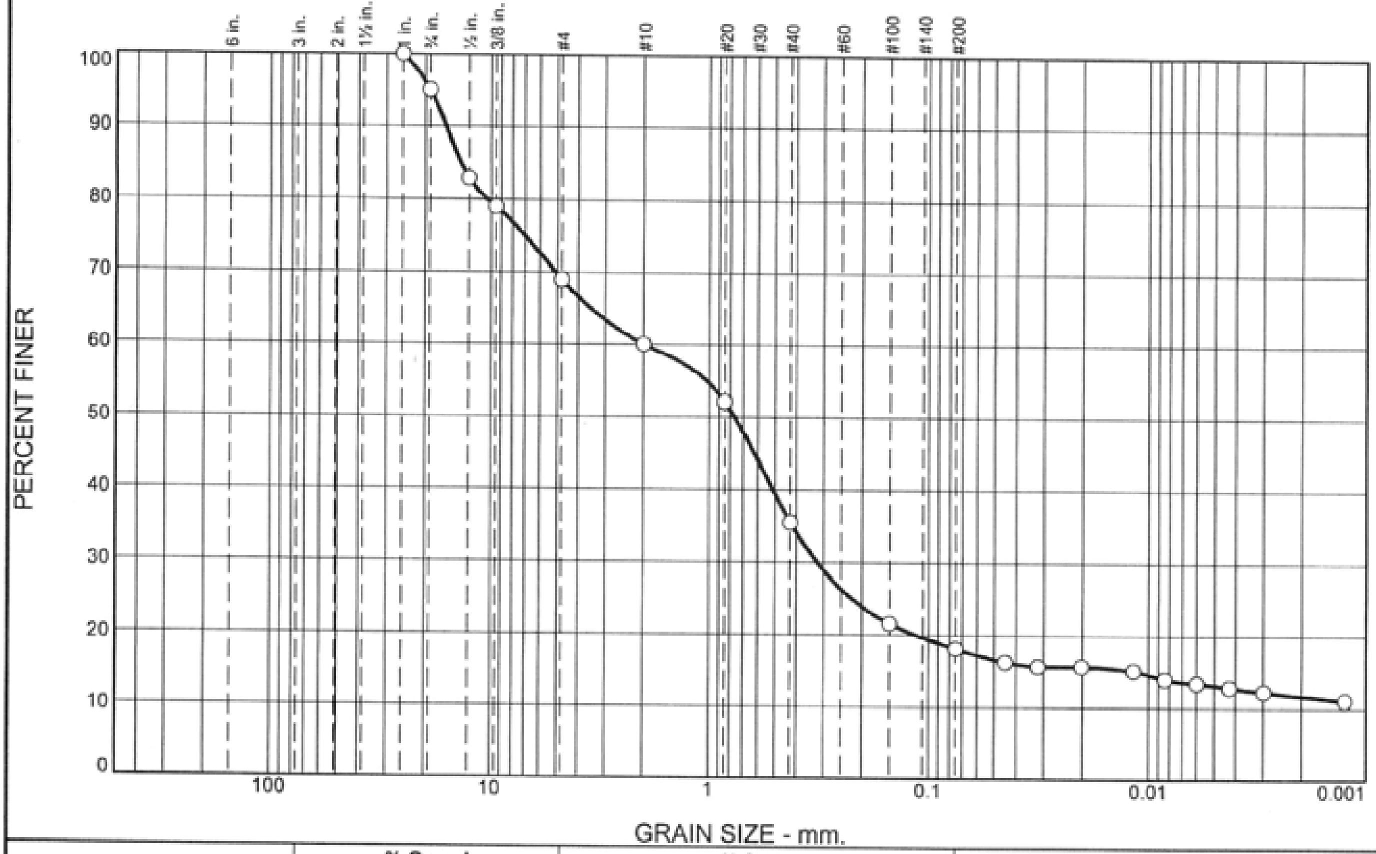
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.6
Medium Sand (<No. 10-No. 40):	23.6
Fine Sand (<No. 40-No. 200):	38.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	63.0	37.0

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.0	26.0	9.0	24.5	17.4	6.5	11.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	95.0		
0.5"	83.0		
0.375"	79.0		
#4	69.0		
#10	60.0		
#20	52.1		
#40	35.5		
#100	21.6		
#200	18.1		

* (no specification provided)

Material Description

SC, (#30341)

Atterberg Limits

PL= 11.8
LL= 35.1
PI= 23.3

Coefficients

D_{90}= 16.1128
D_{85}= 13.7379
D_{60}= 2.0000
D_{50}= 0.7654
D_{30}= 0.3213
D_{15}= 0.0115
D_{10}=
C_u=
C_c=

Classification

USCS= SC
AASHTO= A-2-6(0)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B5-1
Depth: 5'

Date: 2/6/17

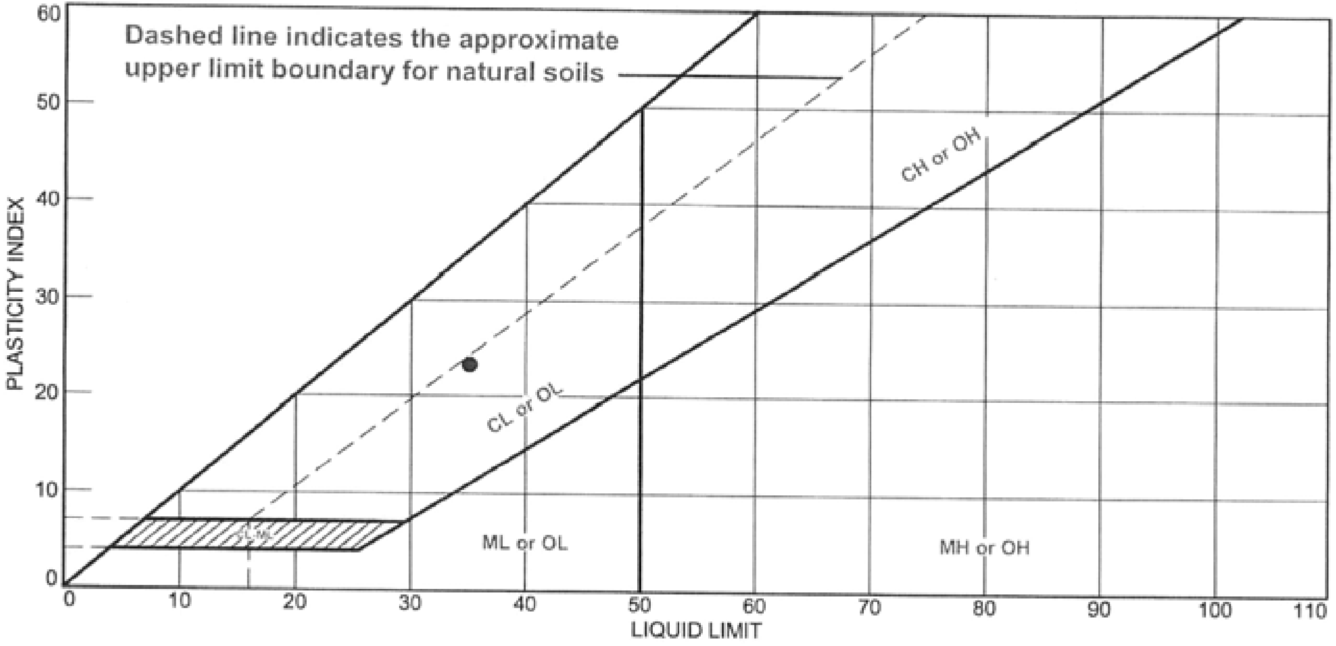
amec

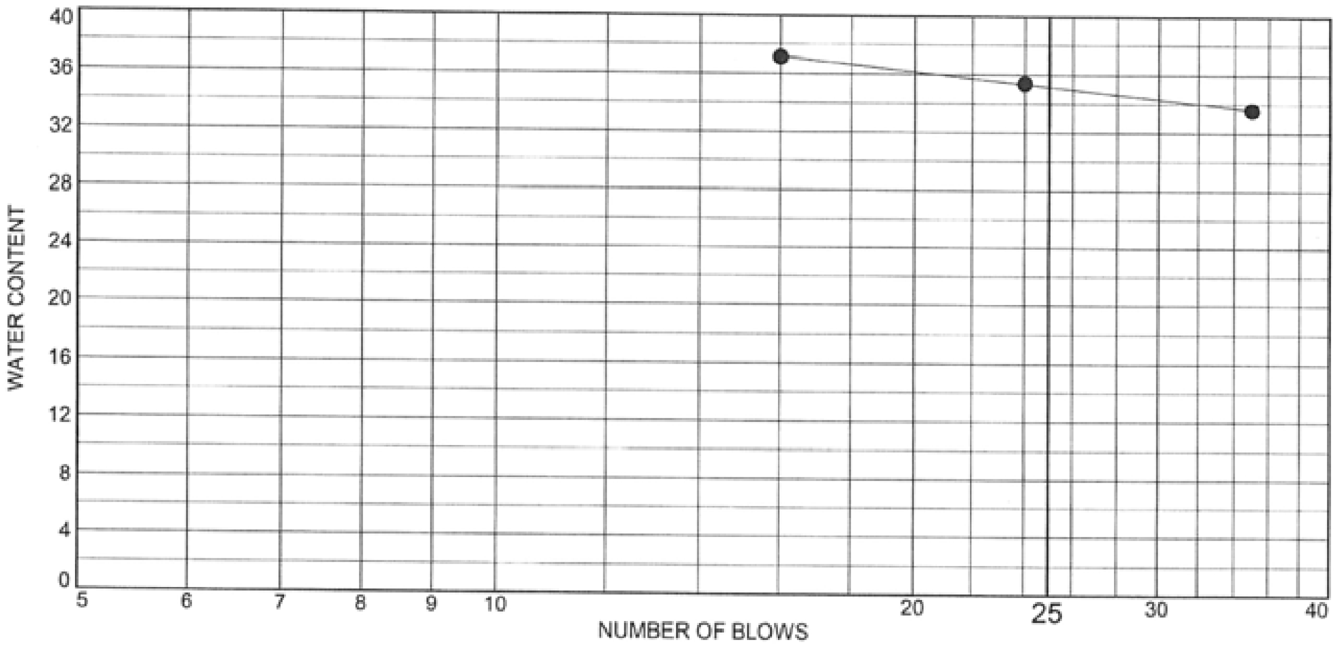
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21
Figure #30341

Tested By: Iacovera/Gibson
Checked By: Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SC, (#30341)	35.1	11.8	23.3	35.5	18.1	SC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B5-1
Depth: 5'

Remarks:

Figure #30341

Tested By: Iacovera **Checked By:** Collins

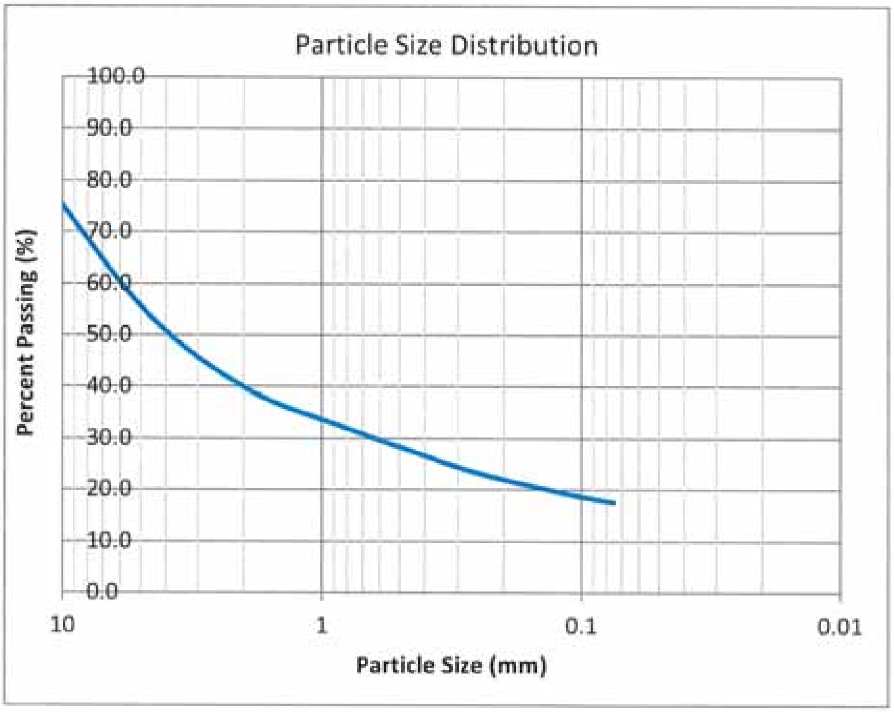
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B5-3 @ 12-14'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	81.6
No. 4	4.75	54.5
No. 10	2	39.9
No. 20	0.85	32.4
No. 40	0.425	27.1
No. 60	0.25	23.2
No. 140	0.106	18.9
No. 200	0.075	17.5
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	18.4
Course Sand (<No. 4-No.10):	41.8
Medium Sand (<No. 10-No. 40):	12.8
Fine Sand (<No. 40-No. 200):	9.6
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
18.4	64.1	17.5

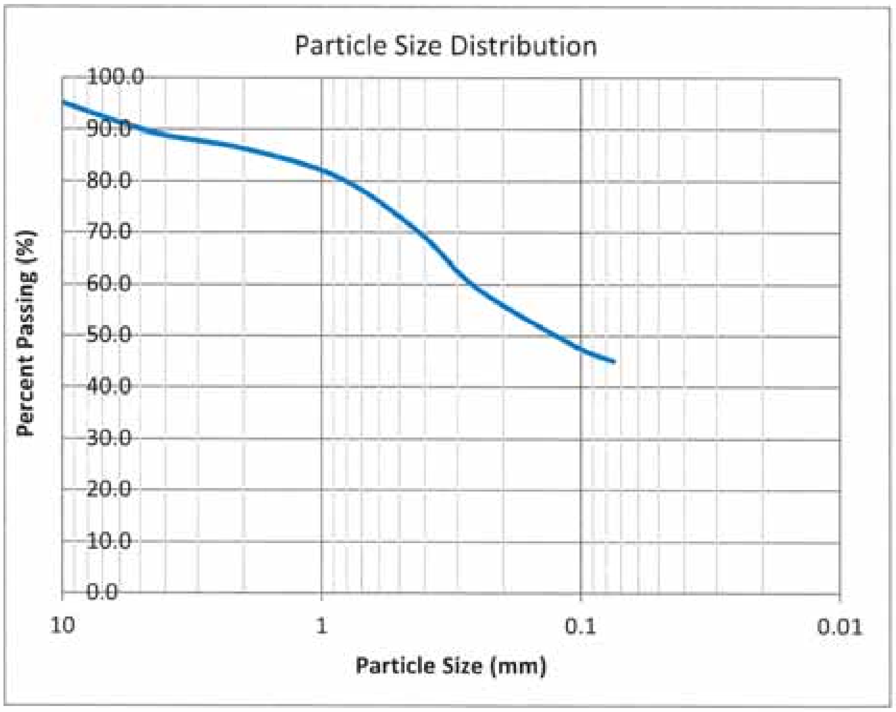
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B6-2 @ 10-12'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	96.8
No. 4	4.75	89.7
No. 10	2	86.4
No. 20	0.85	80.7
No. 40	0.425	70.3
No. 60	0.25	59.1
No. 140	0.106	48.0
No. 200	0.075	45.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



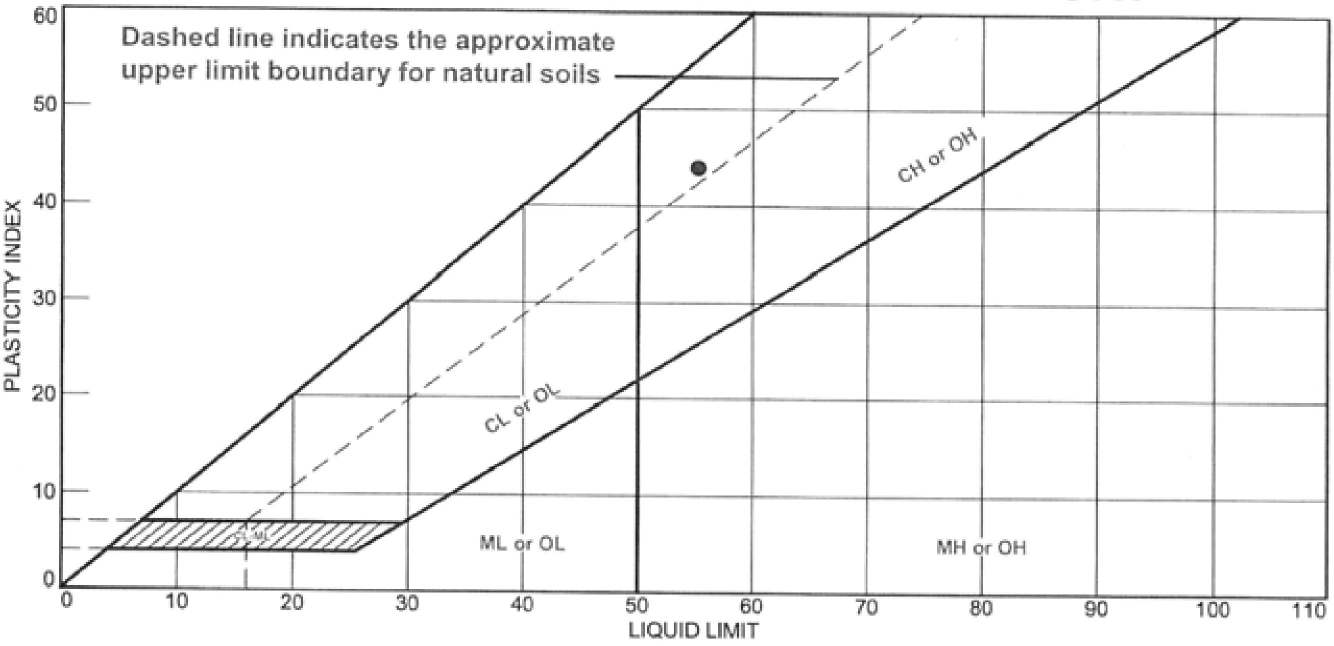
ASTM Composition of Total Sample per ASTM D422-63

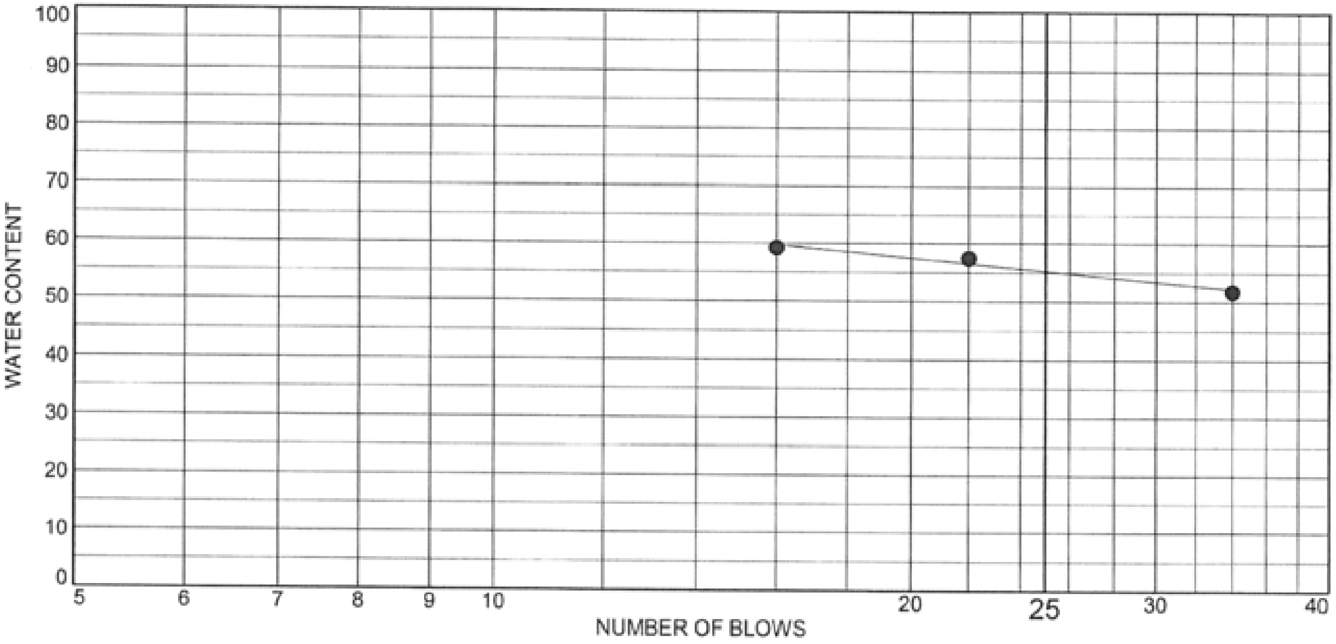
	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	3.2
Course Sand (<No. 4-No.10):	10.5
Medium Sand (<No. 10-No. 40):	16.1
Fine Sand (<No. 40-No. 200):	25.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
3.2	51.8	45.0

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CH, (#30344)	55.2	11.2	44.0			CH

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B21-1
Depth: 3.5'

Remarks:

Figure #30344

Tested By: Iacovera **Checked By:** Collins

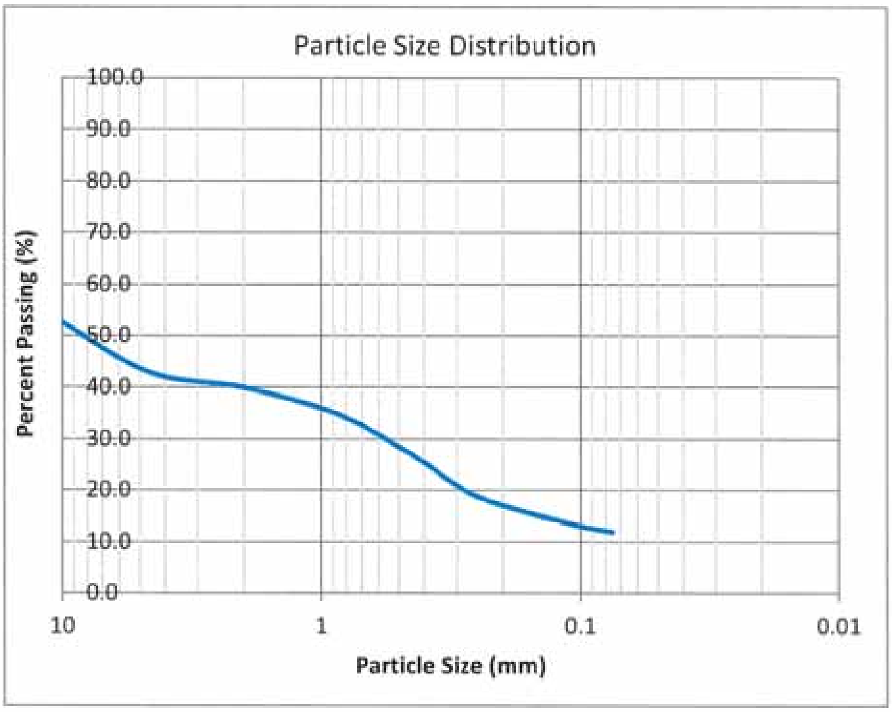
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B21-3 @ 10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	83.4
3/8"	12.5	55.7
No. 4	4.75	43.1
No. 10	2	39.9
No. 20	0.85	34.6
No. 40	0.425	26.2
No. 60	0.25	18.7
No. 140	0.106	13.3
No. 200	0.075	12.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



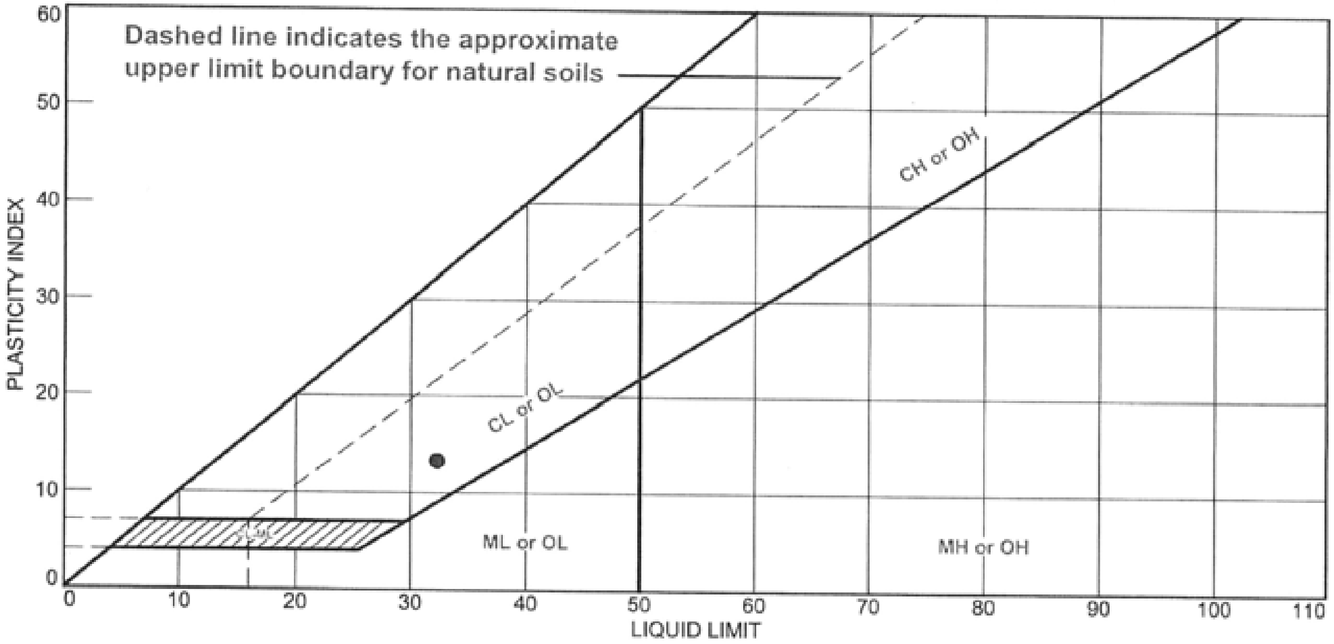
ASTM Composition of Total Sample per ASTM D422-63

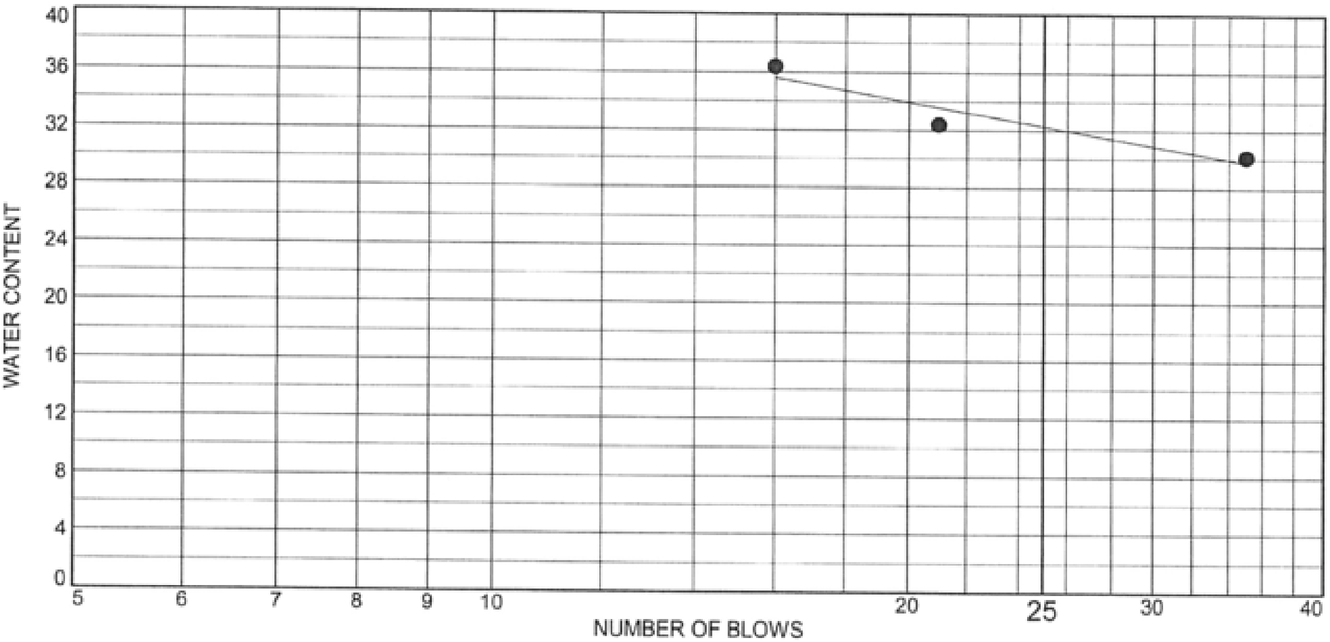
	%
Course Gravel (3"-3/4"):	16.6
Fine Gravel (<3/4"- No. 4):	27.6
Course Sand (<No. 4-No.10):	15.8
Medium Sand (<No. 10-No. 40):	13.7
Fine Sand (<No. 40-No. 200):	14.3
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
44.3	43.8	12.0

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	(#30345)	32.3	19.0	13.3			

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

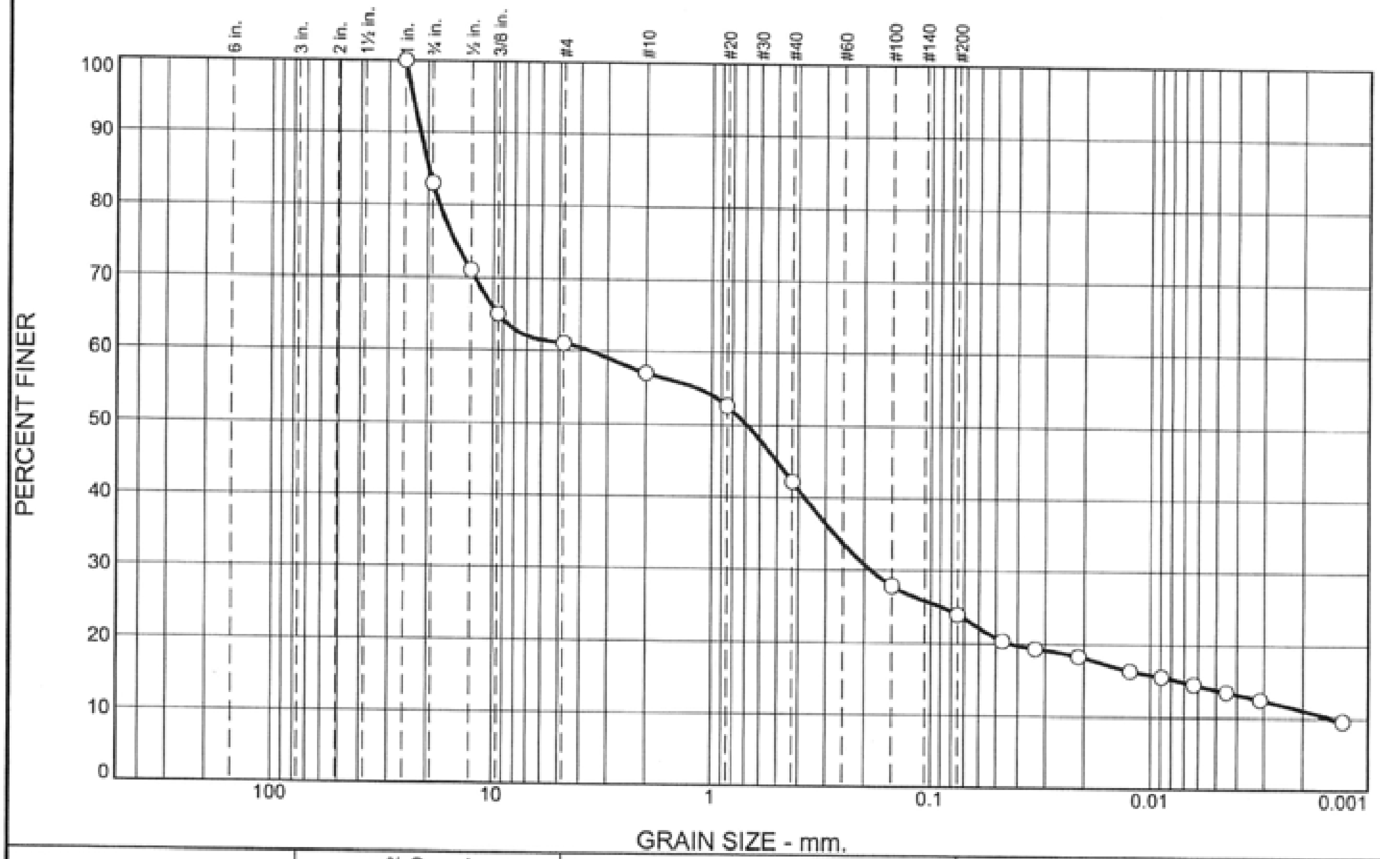
Location: B21-4
Depth: 16'

Remarks:

Figure #30345

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	17.0	22.0	4.0	14.8	18.2	13.0	11.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	83.0		
0.5"	71.0		
0.375"	65.0		
#4	61.0		
#10	57.0		
#20	52.6		
#40	42.2		
#100	27.9		
#200	24.0		

* (no specification provided)

Material Description

GC, (#30346)

Atterberg Limits

PL= 13.2 LL= 33.7 PI= 20.5

Coefficients

D_{90}= 21.6985 D_{85}= 19.8423 D_{60}= 3.6182
D_{50}= 0.6875 D_{30}= 0.1873 D_{15}= 0.0074
D_{10}= 0.0015 C_u= 2424.33 C_c= 6.50

Classification

USCS= GC AASHTO= A-2-6(1)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B7-1
Depth: 5'

Date: 2/9/17

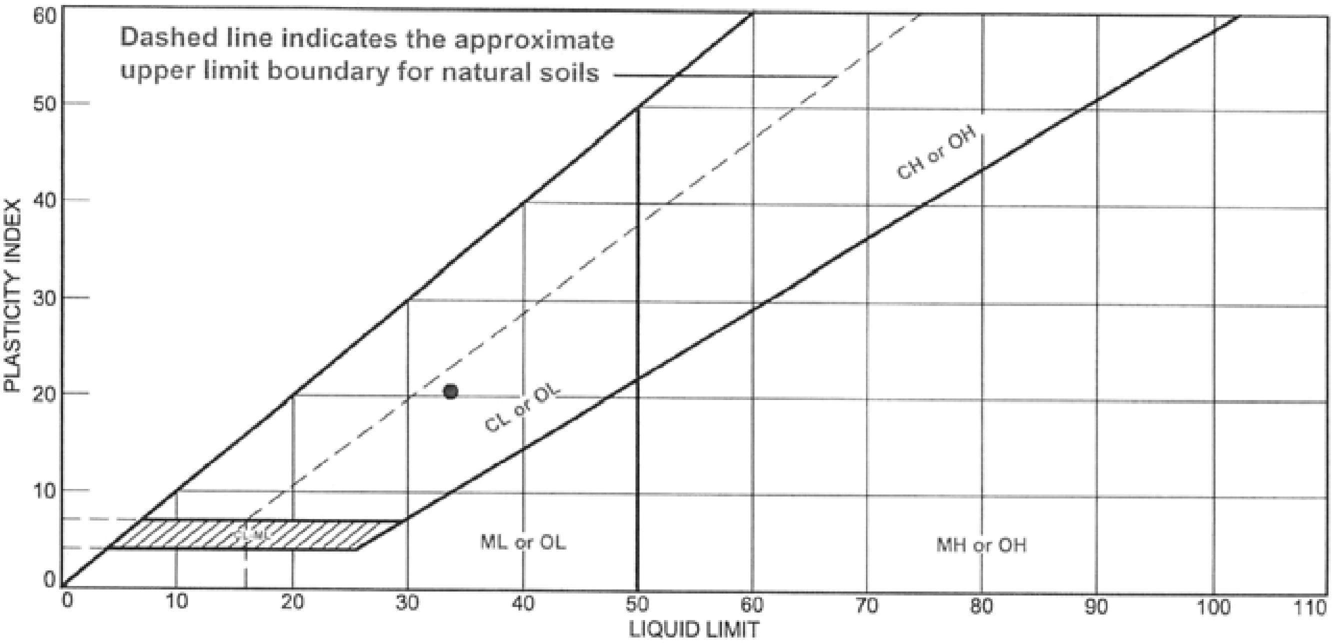
amec

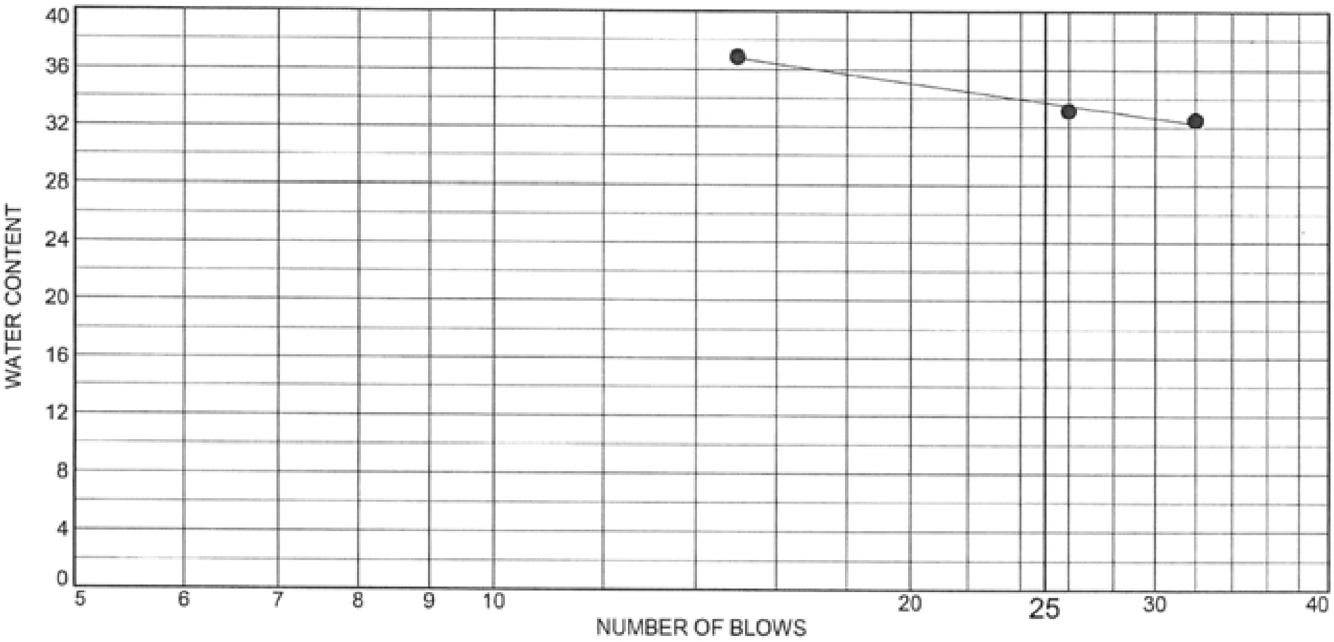
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30346

Tested By: Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	(#30346)	33.7	13.2	20.5	42.2	24.0	GC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B7-1
Depth: 5'

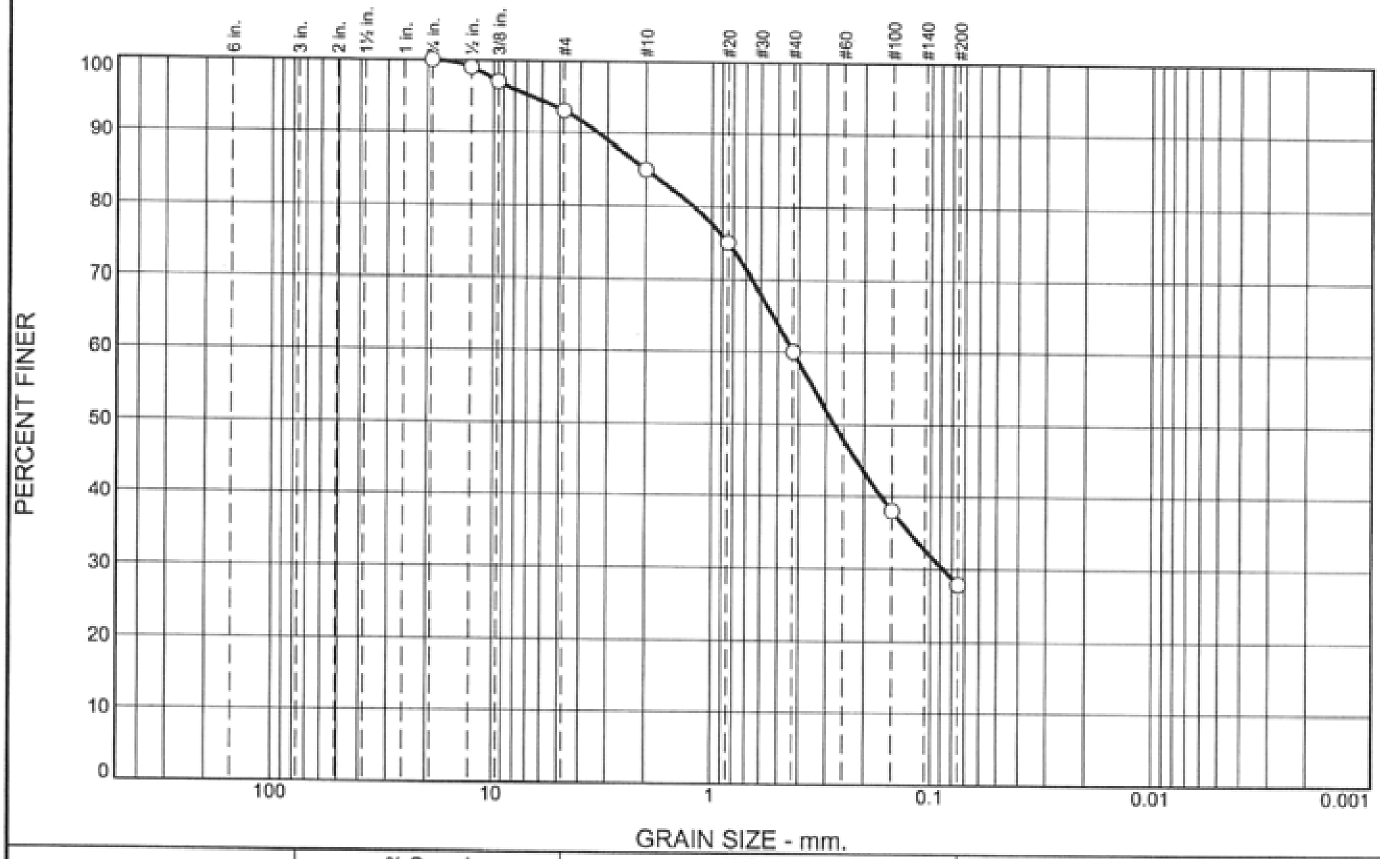
amec

Remarks:

Figure #30346

Tested By: Gibson **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.0	8.0	25.0	32.1	27.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	99.0		
0.375"	97.0		
#4	93.0		
#10	85.0		
#20	75.0		
#40	60.0		
#100	38.0		
#200	27.9		

* (no specification provided)

Material Description

Clayey Sand, (#30347)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.3126 D_{85}= 2.0000 D_{60}= 0.4250
D_{50}= 0.2749 D_{30}= 0.0877 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Location: B7-2
Depth: 6'-8'

Date: 2/6/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30347

Tested By: Adame **Checked By:** Collins

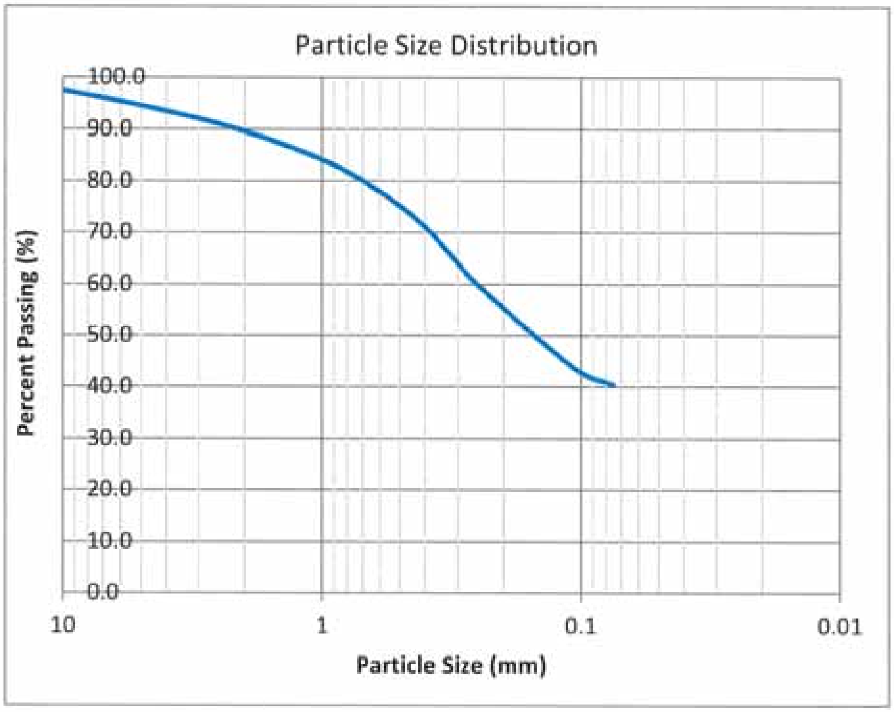
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B7-5 @ 16-18'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	98.4
No. 4	4.75	94.2
No. 10	2	89.5
No. 20	0.85	82.5
No. 40	0.425	72.2
No. 60	0.25	59.8
No. 140	0.106	43.6
No. 200	0.075	40.4
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	1.6
Course Sand (<No. 4-No.10):	8.8
Medium Sand (<No. 10-No. 40):	17.3
Fine Sand (<No. 40-No. 200):	31.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
1.6	58.0	40.4

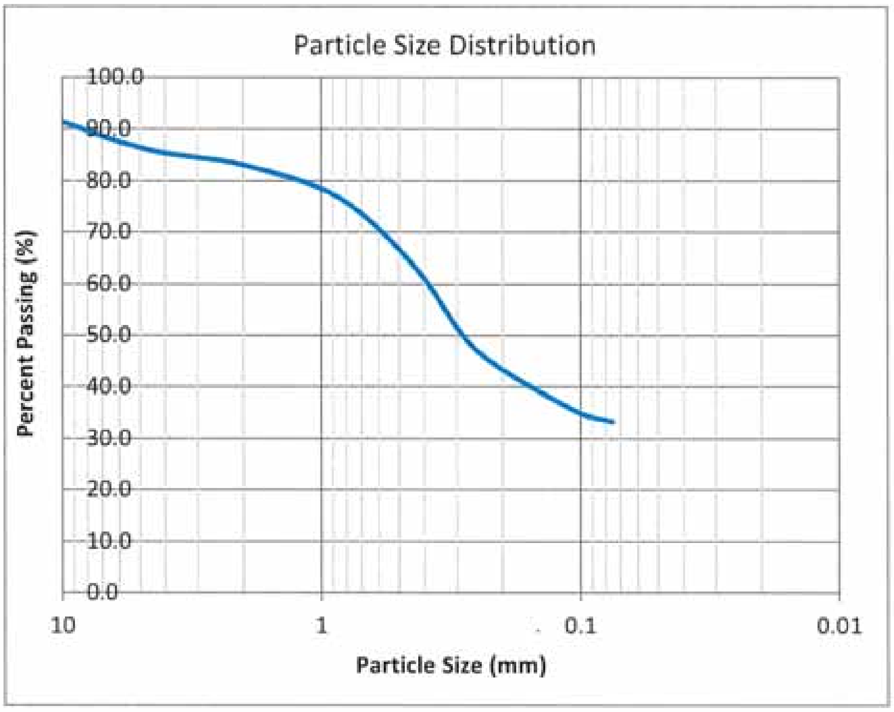
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B8-2 @ 10-12'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	92.9
No. 4	4.75	86.2
No. 10	2	83.1
No. 20	0.85	76.7
No. 40	0.425	62.8
No. 60	0.25	47.0
No. 140	0.106	35.5
No. 200	0.075	33.3
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



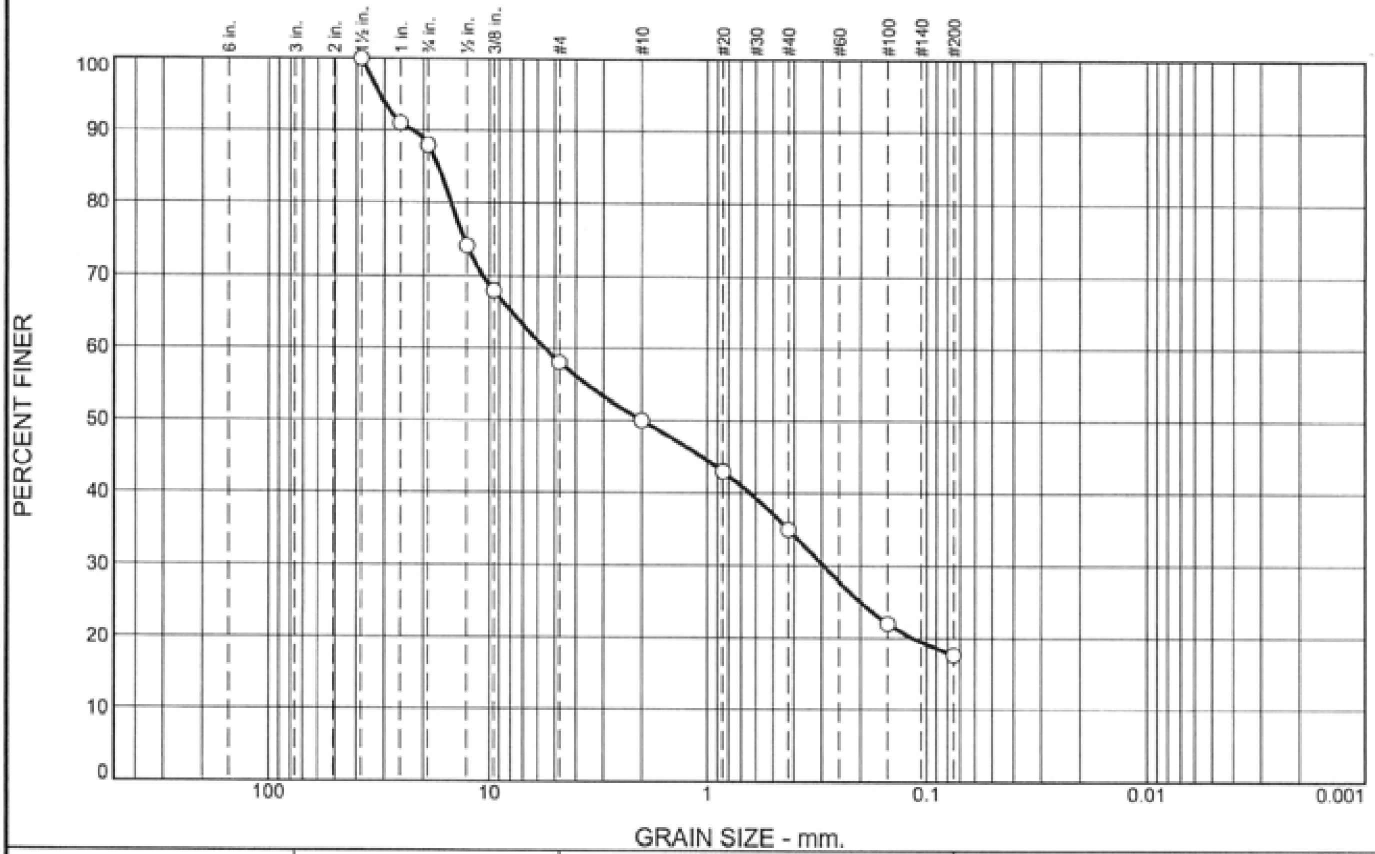
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	7.1
Course Sand (<No. 4-No.10):	9.8
Medium Sand (<No. 10-No. 40):	20.3
Fine Sand (<No. 40-No. 200):	29.5
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
7.1	59.6	33.3

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.0	30.0	8.0	15.0	17.4	17.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	91.0		
0.75"	88.0		
0.5"	74.0		
0.375"	68.0		
#4	58.0		
#10	50.0		
#20	43.0		
#40	35.0		
#100	22.0		
#200	17.6		

* (no specification provided)

Material Description

(#30349)

Atterberg Limits

PL=
LL=
PI=

Coefficients

D_{90}= 22.4269
D_{85}= 17.1108
D_{60}= 5.5572
D_{50}= 2.0000
D_{30}= 0.2938
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS=
AASHTO=

Remarks

Location: B8-3
Depth: 10'

Date: 2/2/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21
Figure #30349

Tested By: Woodard
Checked By: Collins

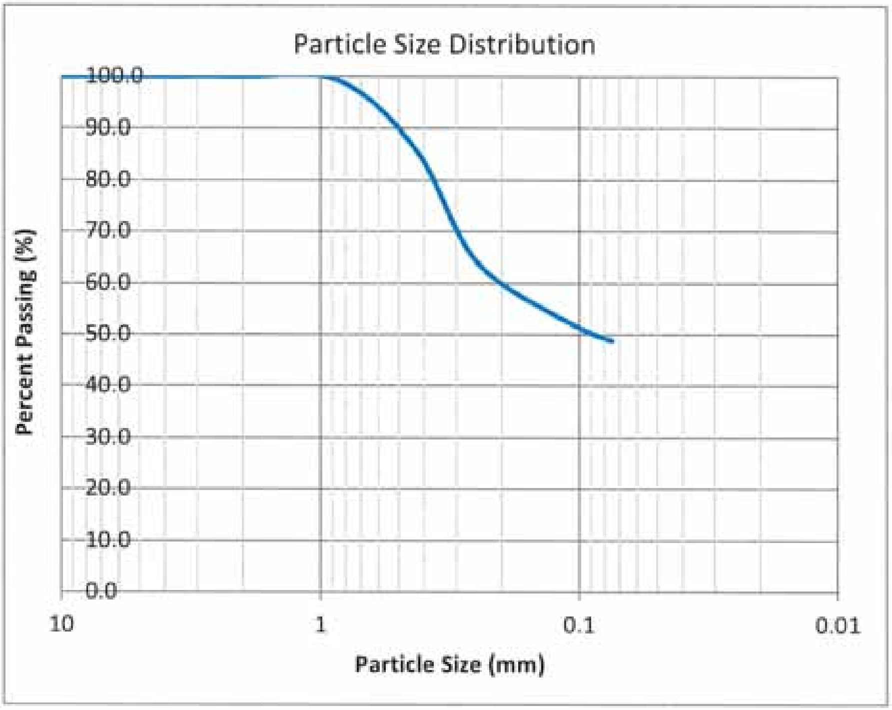
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B9-3 @ 11-13'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.2
No. 40	0.425	85.5
No. 60	0.25	64.1
No. 140	0.106	52.0
No. 200	0.075	48.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



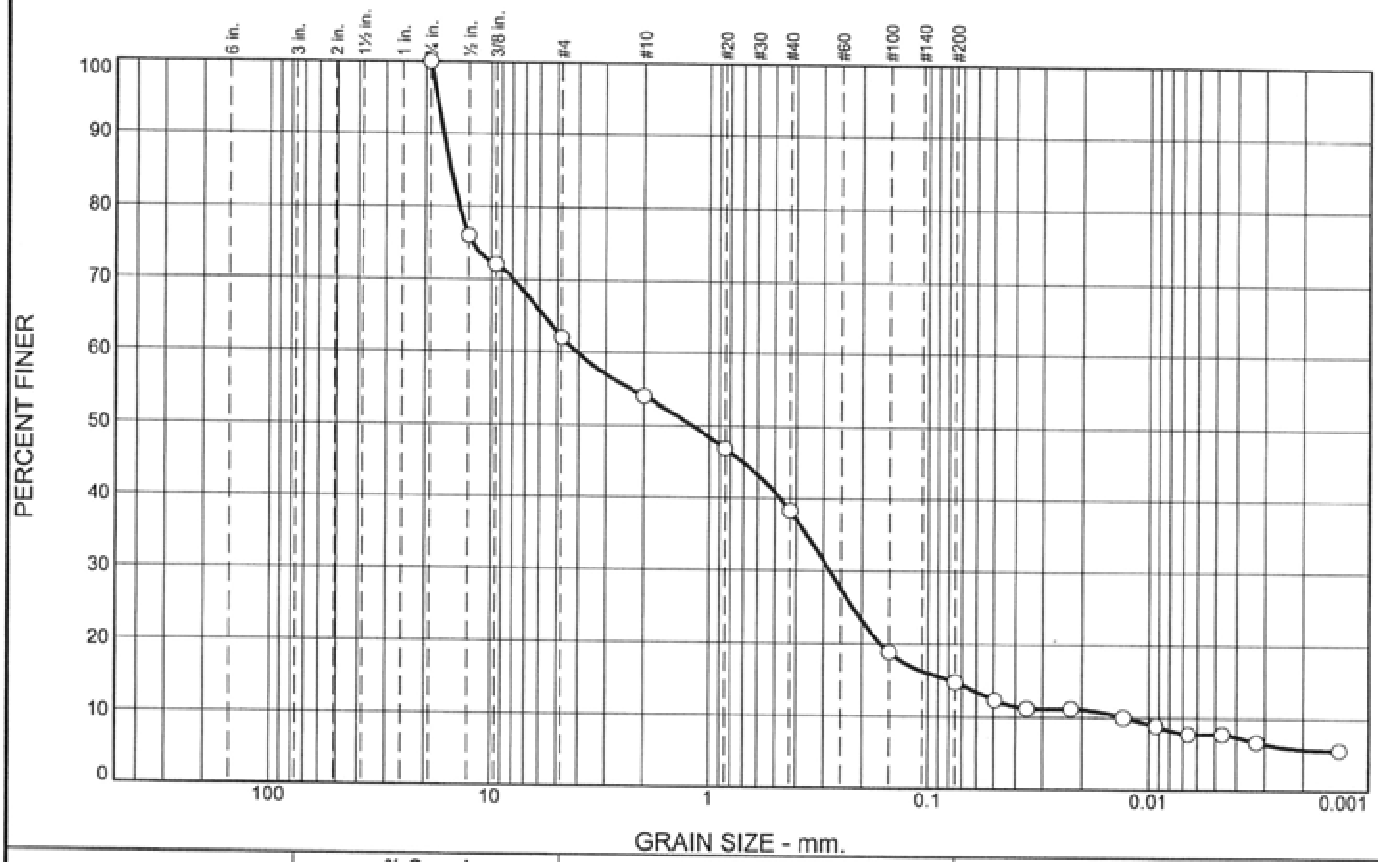
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	14.5
Fine Sand (<No. 40-No. 200):	36.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	51.2	48.8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	38.0	8.0	15.7	23.6	9.0	5.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	76.0		
0.375"	72.0		
#4	62.0		
#10	54.0		
#20	46.9		
#40	38.3		
#100	18.8		
#200	14.7		

* (no specification provided)

Material Description

SC-SM, (#30353)

Atterberg Limits

PL= 19.2 LL= 25.7 PI= 6.5

Coefficients

D_{90}= 16.5578 D_{85}= 15.3284 D_{60}= 4.0734
D_{50}= 1.2003 D_{30}= 0.2801 D_{15}= 0.0799
D_{10}= 0.0129 C_u= 316.95 C_c= 1.50

Classification

USCS= SC-SM AASHTO= A-2-4(0)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B10-1
Depth: 5'

Date: 2/7/17

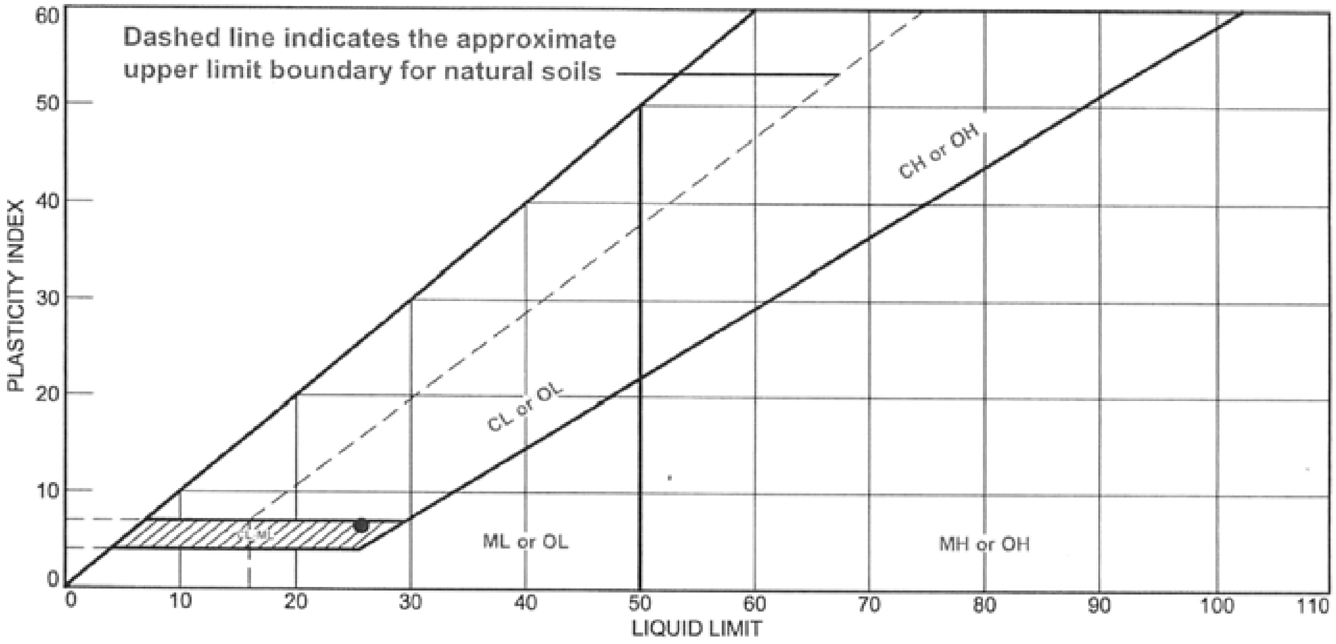
amec

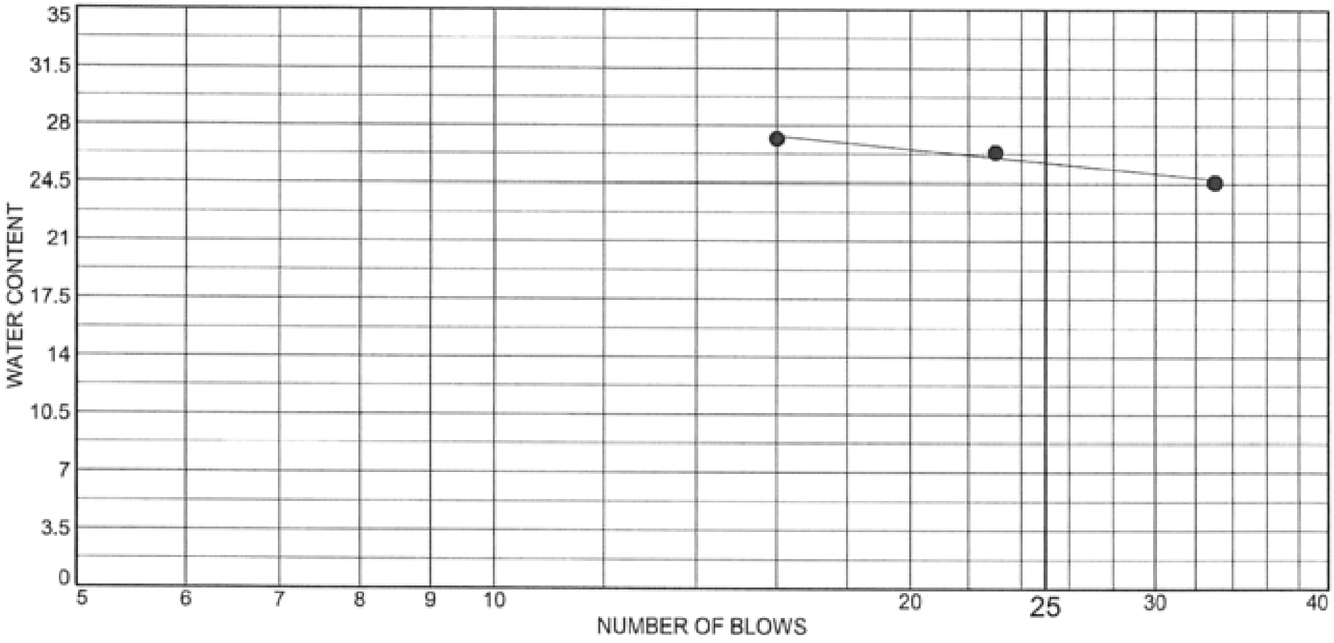
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30353

Tested By: J. Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SC-SM, (#30353)	25.7	19.2	6.5	38.3	14.7	SC-SM

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B10-1
Depth: 5'

Remarks:

Figure

Tested By: Iacovera **Checked By:** Collins

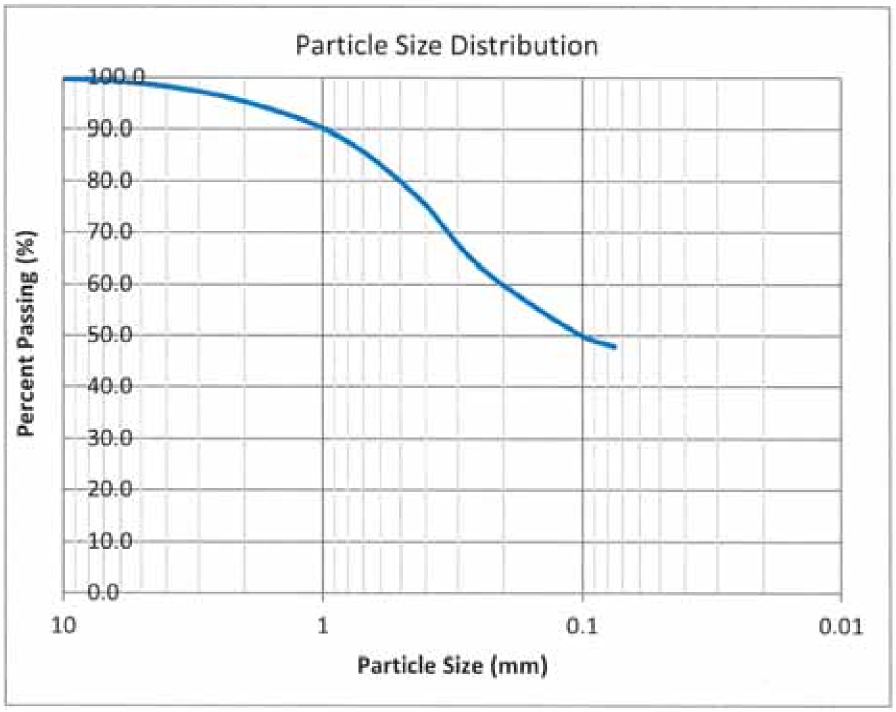
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B10-2 @ 10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	98.7
No. 10	2	95.3
No. 20	0.85	88.4
No. 40	0.425	76.7
No. 60	0.25	63.6
No. 140	0.106	50.6
No. 200	0.075	47.9
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



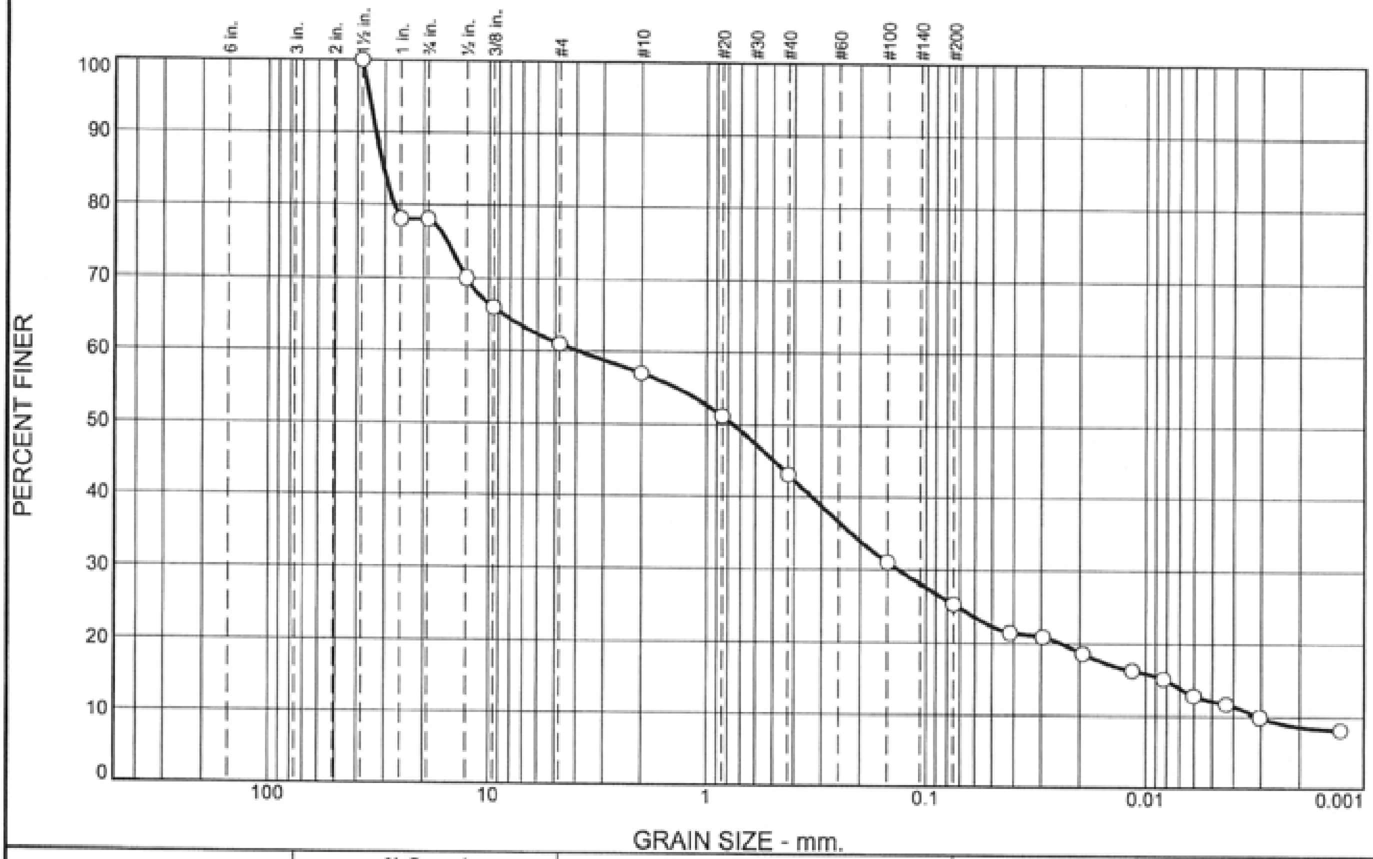
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	4.7
Medium Sand (<No. 10-No. 40):	18.6
Fine Sand (<No. 40-No. 200):	28.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	52.1	47.9

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	22.0	17.0	4.0	13.9	17.7	16.8	8.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	78.0		
0.75"	78.0		
0.5	70.0		
0.375"	66.0		
#4	61.0		
#10	57.0		
#20	51.2		
#40	43.1		
#100	31.0		
#200	25.4		

* (no specification provided)

Material Description

GC, (#30355)

Atterberg Limits

PL= 17.6 LL= 39.2 PI= 21.6

Coefficients

D_{90}= 33.0385 D_{85}= 30.4413 D_{60}= 3.8886
D_{50}= 0.7550 D_{30}= 0.1341 D_{15}= 0.0082
D_{10}= 0.0031 C_u= 1239.77 C_c= 1.47

Classification

USCS= GC AASHTO= A-2-6(1)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B11-1
Depth: 5'

Date: 2/7/15

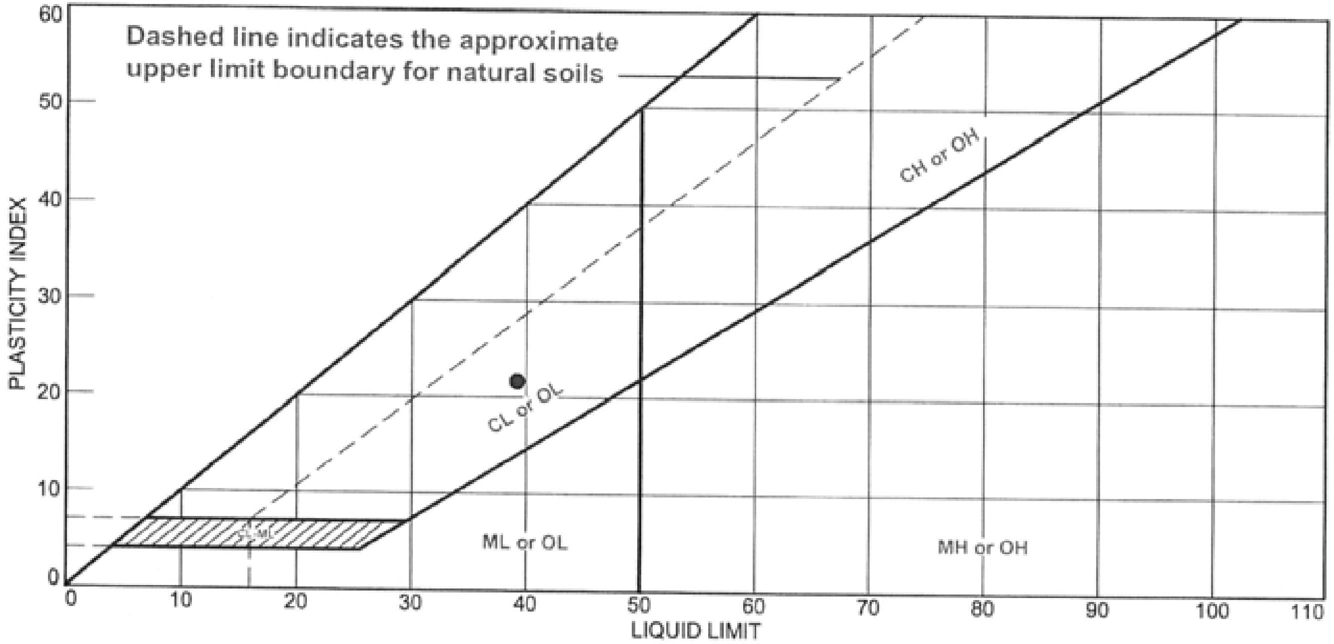
amec

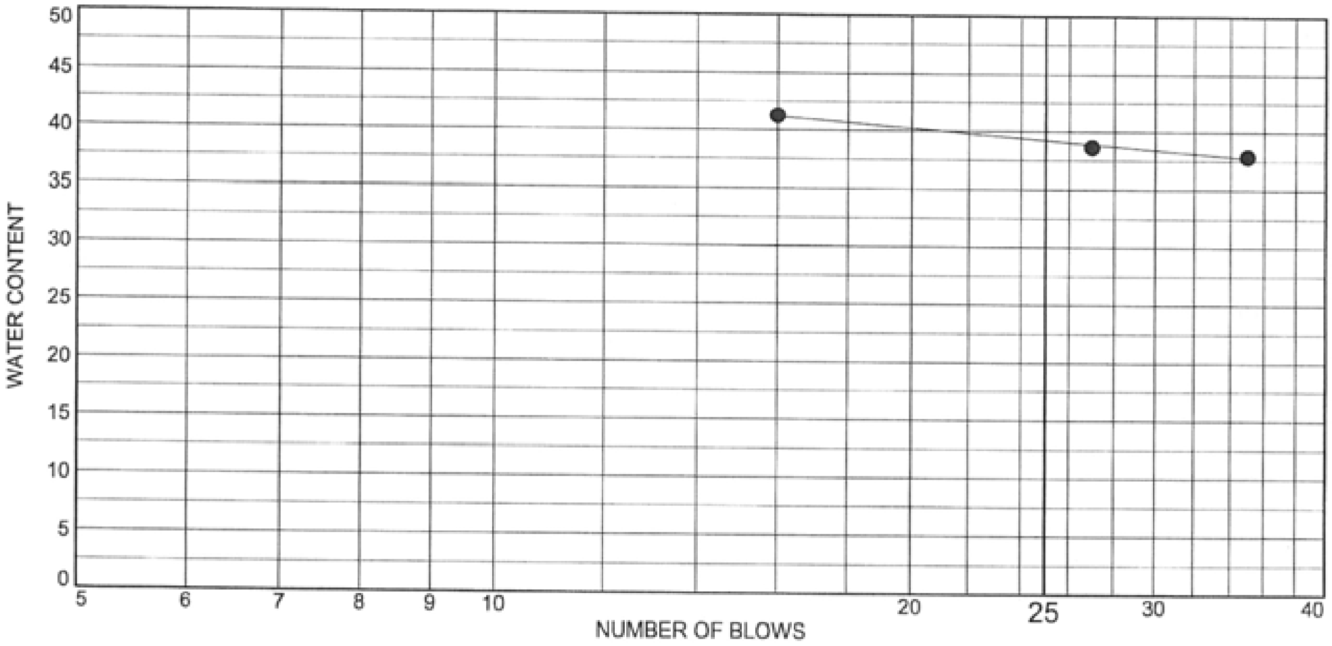
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30355

Tested By: Iacovera **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	GC, (#30355)	39.2	17.6	21.6	43.1	25.4	GC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B11-1
Depth: 5'

Remarks:

Figure #30355

Tested By: Iacovera **Checked By:** Collins

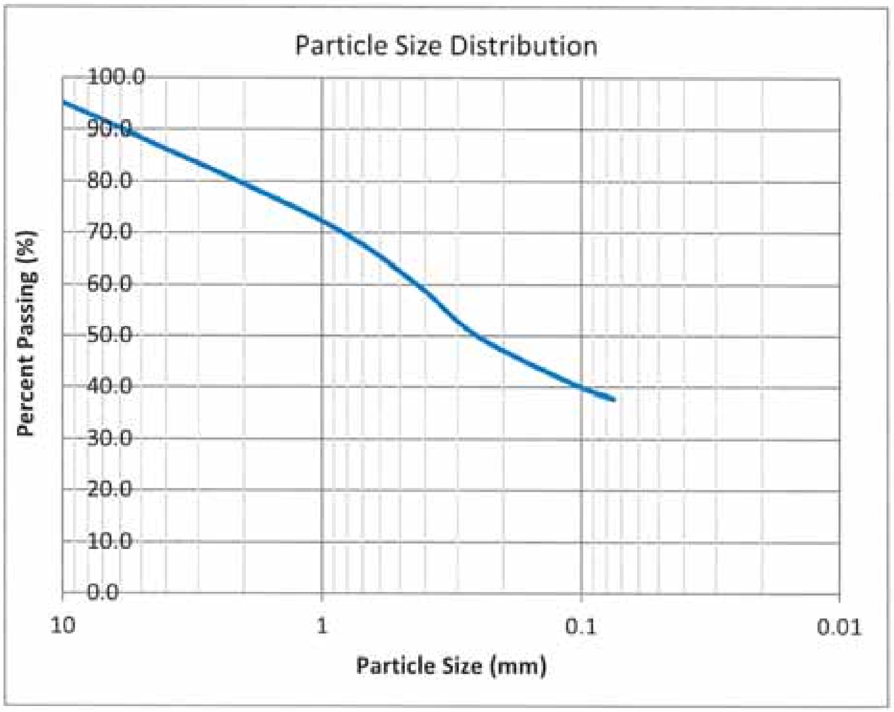
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B11-2 @ 8-10'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	97.3
No. 4	4.75	87.9
No. 10	2	79.5
No. 20	0.85	70.3
No. 40	0.425	59.9
No. 60	0.25	49.9
No. 140	0.106	40.5
No. 200	0.075	37.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



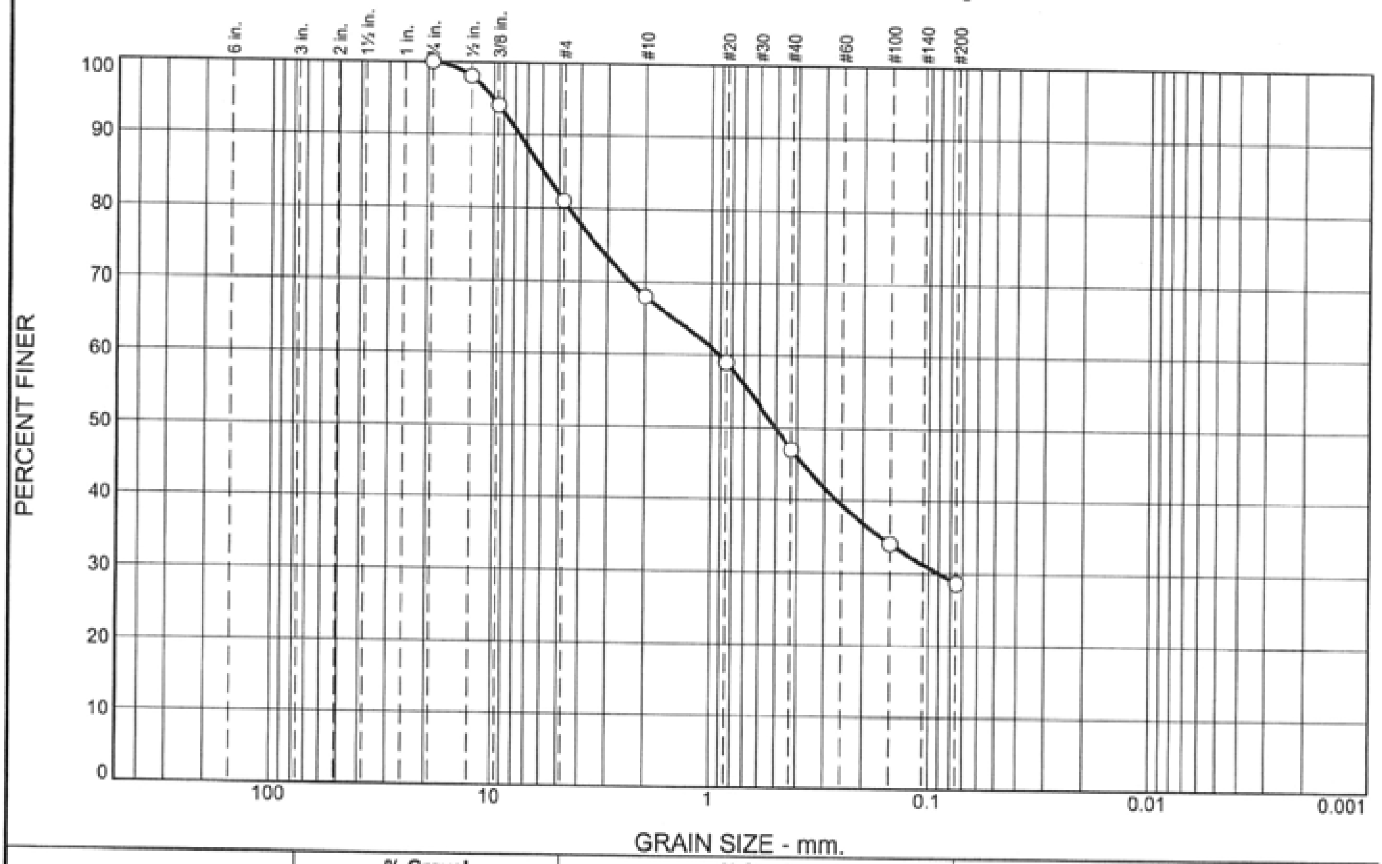
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	2.7
Course Sand (<No. 4-No.10):	17.8
Medium Sand (<No. 10-No. 40):	19.6
Fine Sand (<No. 40-No. 200):	22.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
2.7	59.5	37.8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.0	13.0	21.0	18.3	28.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	98.0		
0.375"	94.0		
#4	81.0		
#10	68.0		
#20	59.0		
#40	47.0		
#100	34.0		
#200	28.7		

* (no specification provided)

Material Description

(#30357)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.6121 D_{85}= 5.8632 D_{60}= 0.9148

D_{50}= 0.5034 D_{30}= 0.0901 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Location: B12-2
Depth: 7'-8'

Date: 2/2/17

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30357

Tested By: Woodard **Checked By:** Collins

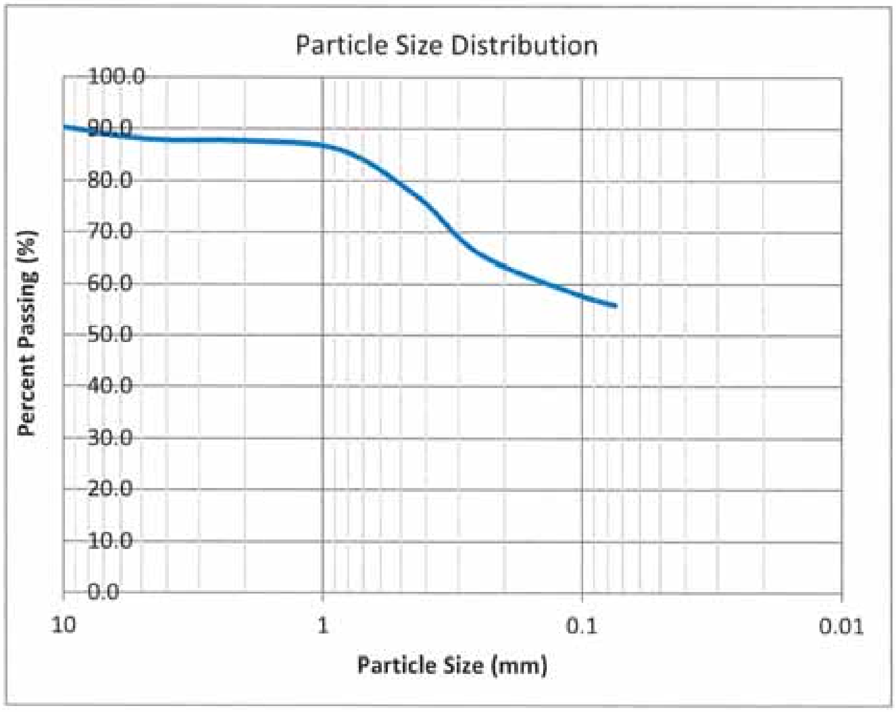
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B12-3 @ 10-11'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	90.9
No. 4	4.75	88.1
No. 10	2	87.8
No. 20	0.85	86.0
No. 40	0.425	76.7
No. 60	0.25	65.9
No. 140	0.106	58.1
No. 200	0.075	55.9
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	9.1
Course Sand (<No. 4-No.10):	3.1
Medium Sand (<No. 10-No. 40):	11.1
Fine Sand (<No. 40-No. 200):	20.9
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
9.1	35.0	55.9

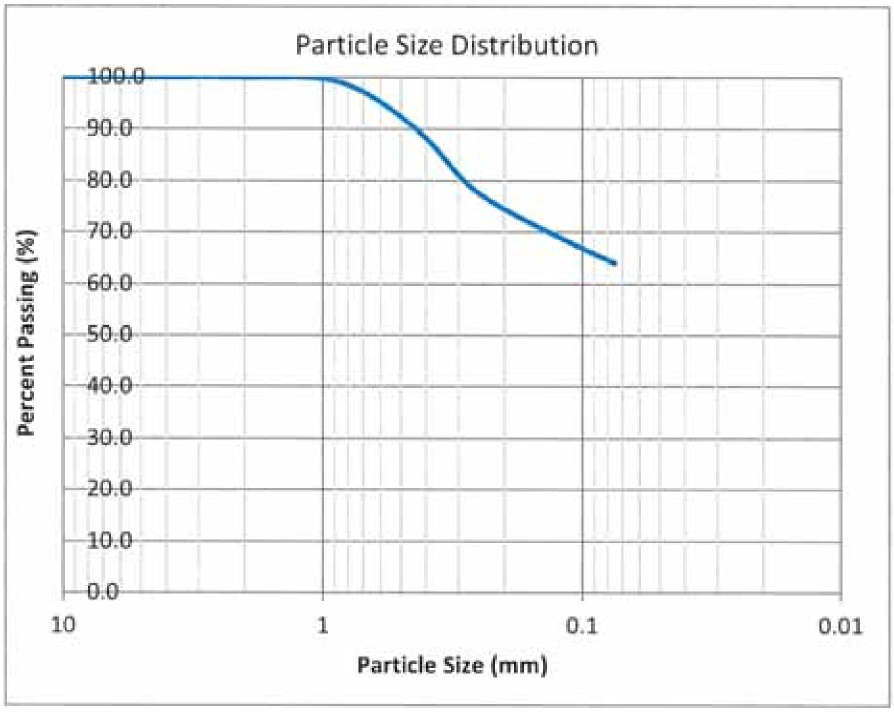
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B13-2 @ 7-8'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	100.0
No. 10	2	100.0
No. 20	0.85	99.0
No. 40	0.425	89.5
No. 60	0.25	77.5
No. 140	0.106	67.4
No. 200	0.075	64.1
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



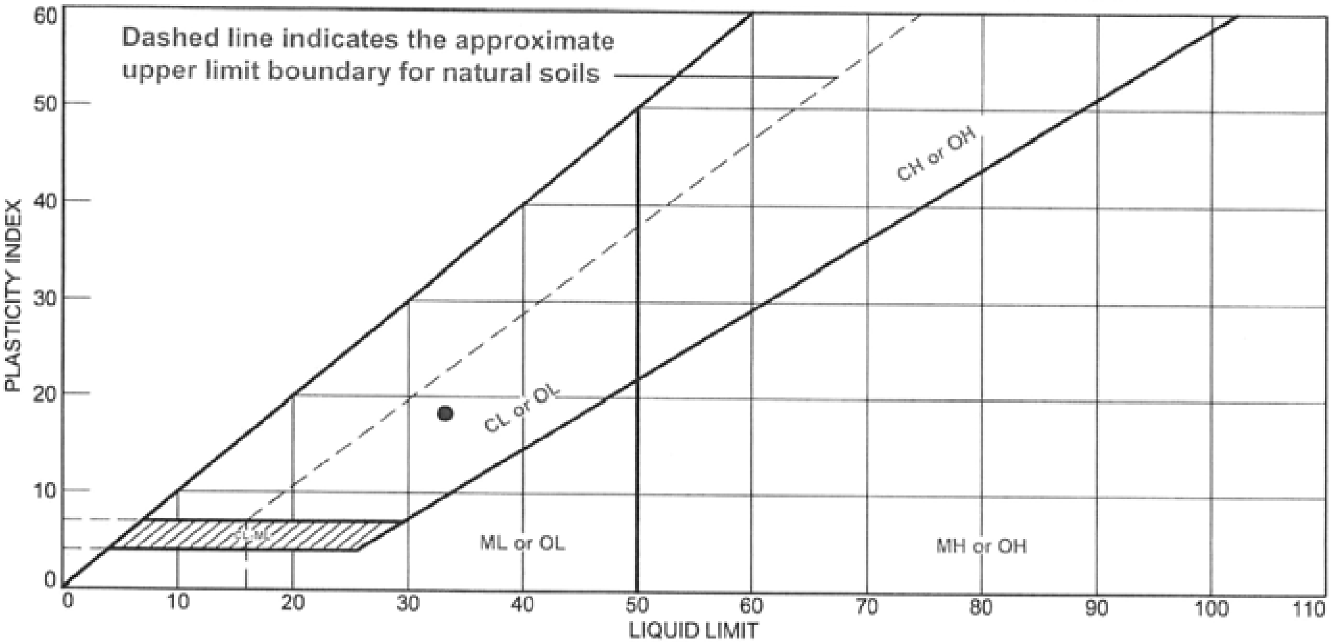
ASTM Composition of Total Sample per ASTM D422-63

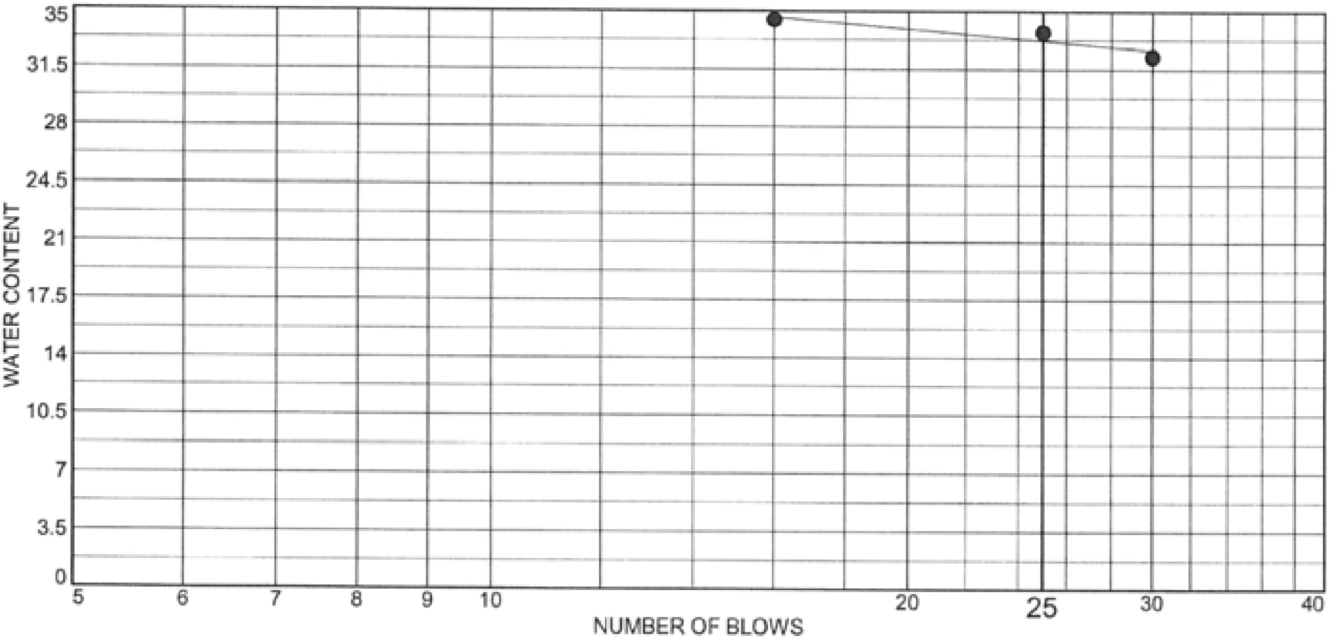
	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	0.0
Medium Sand (<No. 10-No. 40):	10.5
Fine Sand (<No. 40-No. 200):	25.4
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	35.9	64.1

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30359)	33.2	14.9	18.3			CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

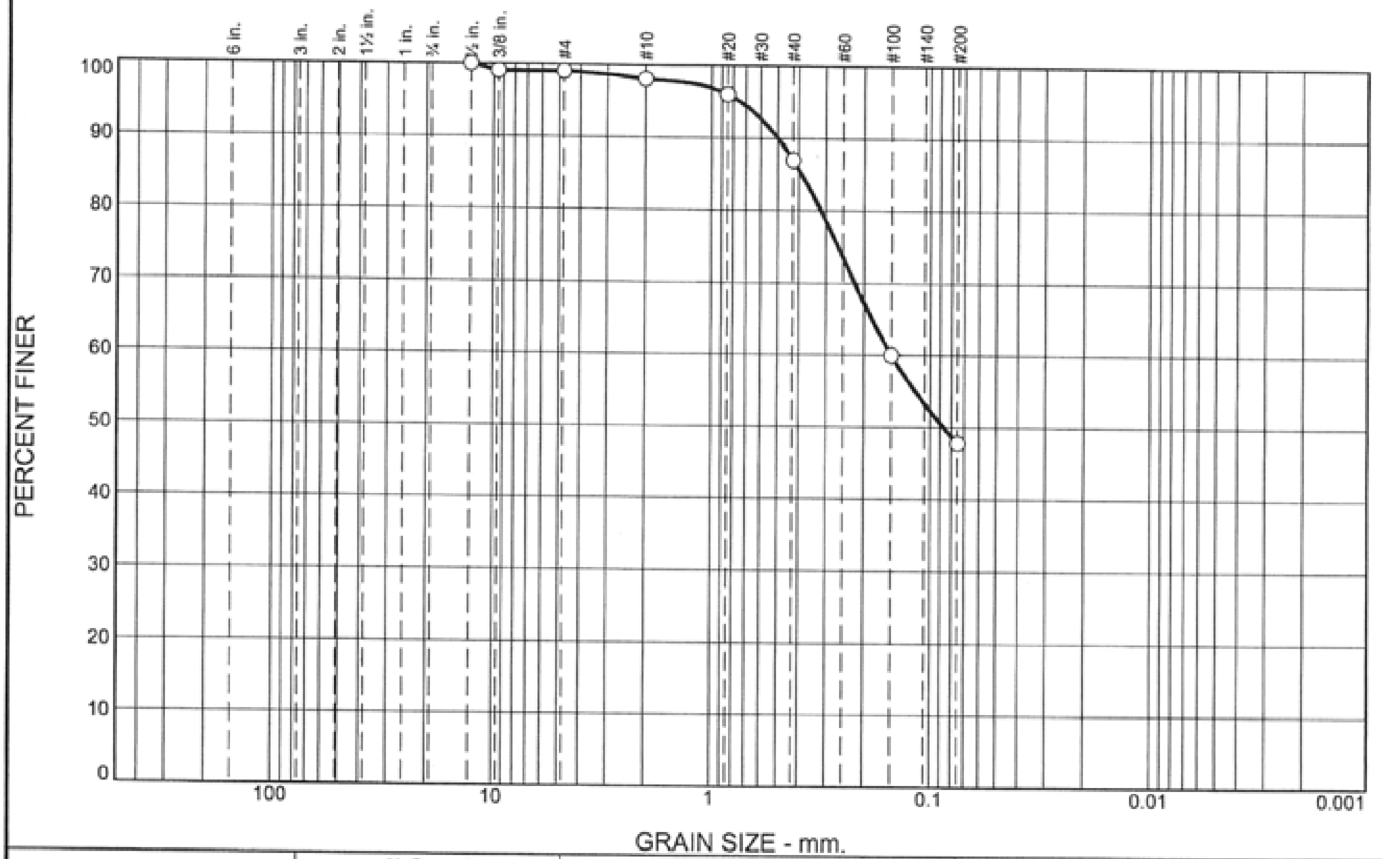
Location: B13-3
Depth: 10'

Remarks:

Figure #30359

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	1.0	11.0	39.2	47.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.5"	100.0		
0.375	99.0		
#4	99.0		
#10	98.0		
#20	96.0		
#40	87.0		
#100	60.0		
#200	47.8		

* (no specification provided)

Material Description

Clayey Sand, (#30360)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4986 D_{85}= 0.3879 D_{60}= 0.1500
D_{50}= 0.0867 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Location: B13-4
Depth: 13'

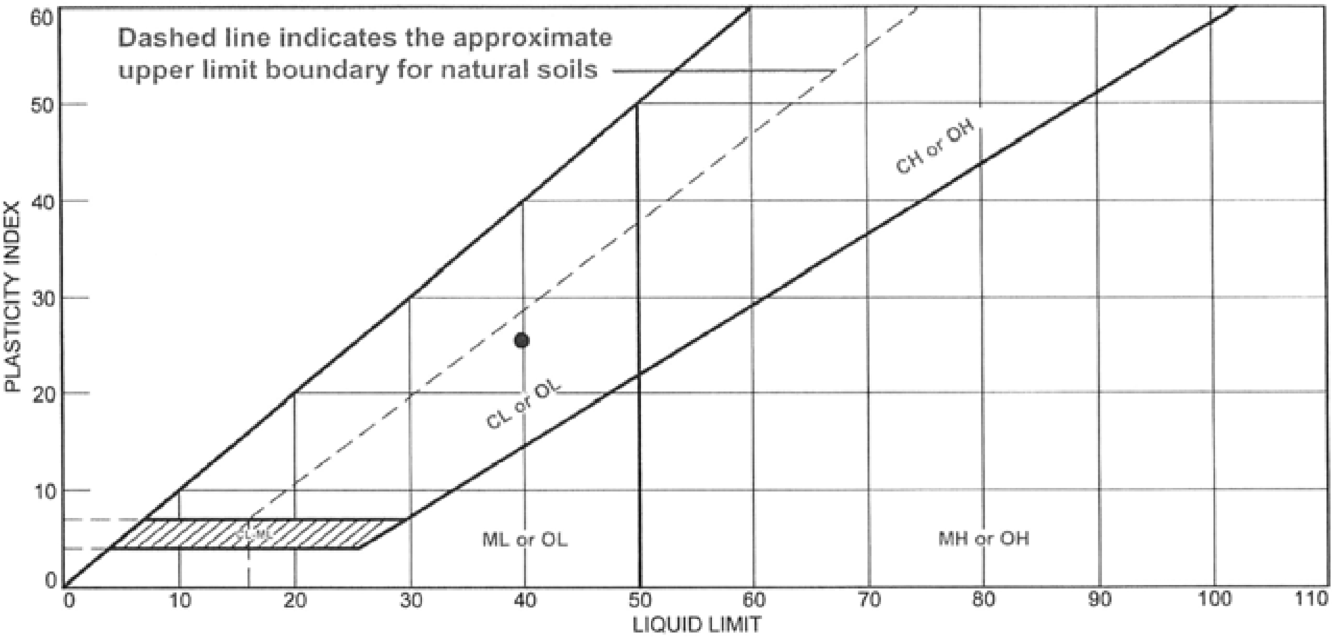
Date: 2/2/17

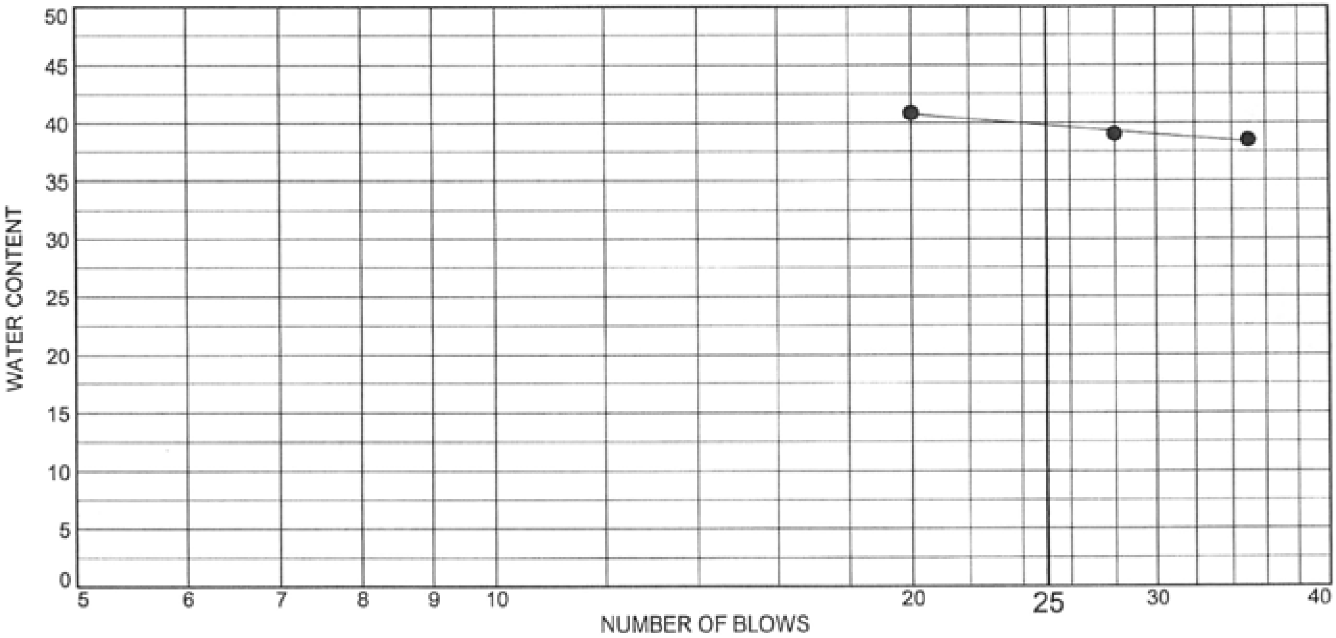
amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project No: 5015-15-0030.21 **Figure** #30360

Tested By: Gibson **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30361)	39.8	14.3	25.5			CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

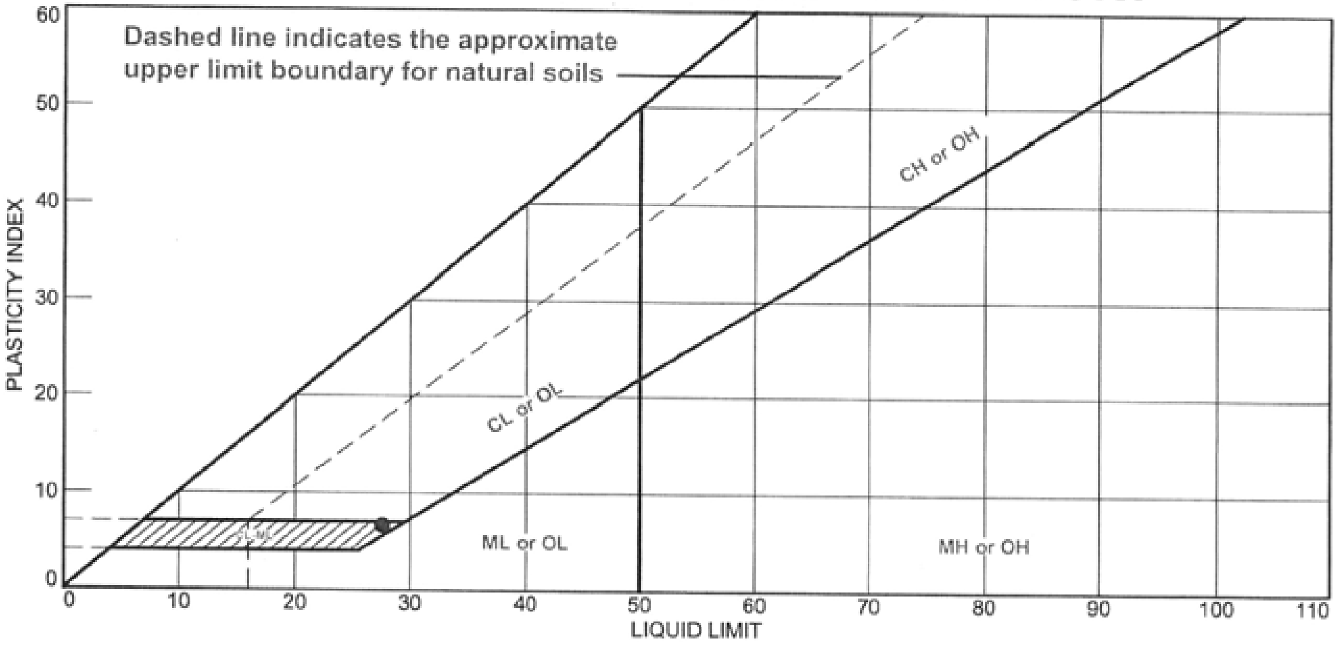
Location: TB2A-1
Depth: 5'

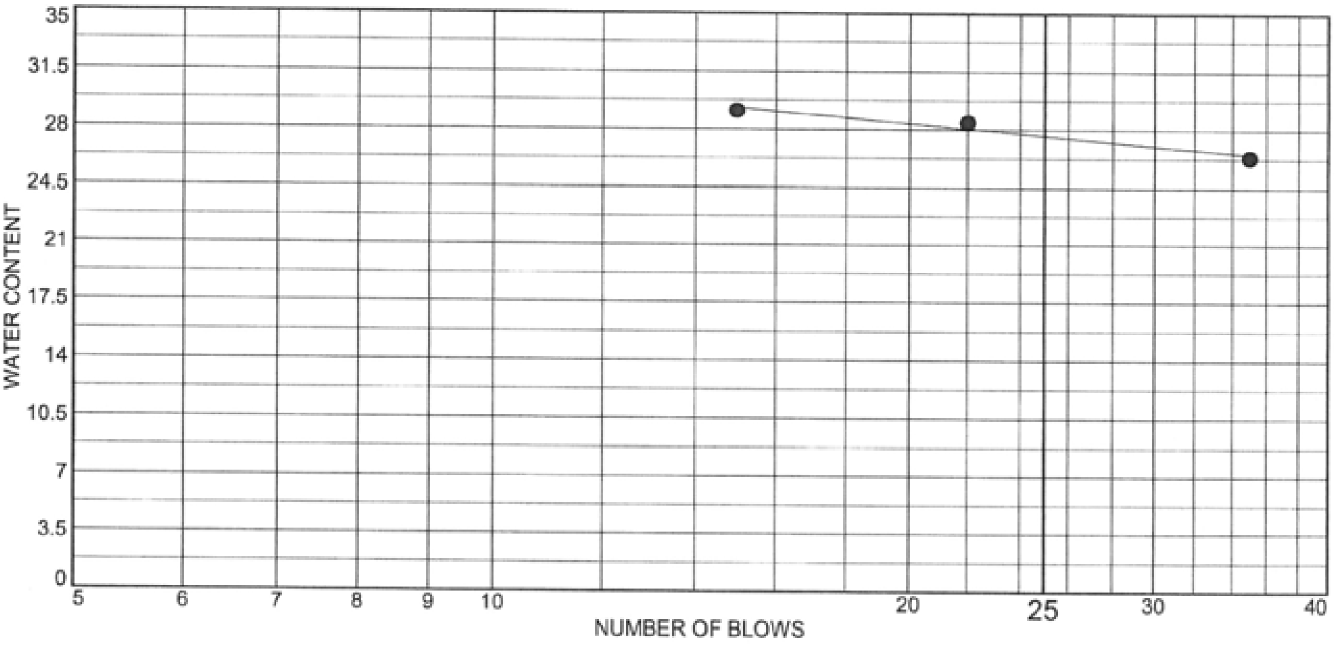
Remarks:

Figure #30361

Tested By: Gibson **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	CL, (#30363)	27.6	21.0	6.6			CL

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: TB2A-3
Depth: 15'

Remarks:

Figure #30363

Tested By: Iacovera **Checked By:** Collins

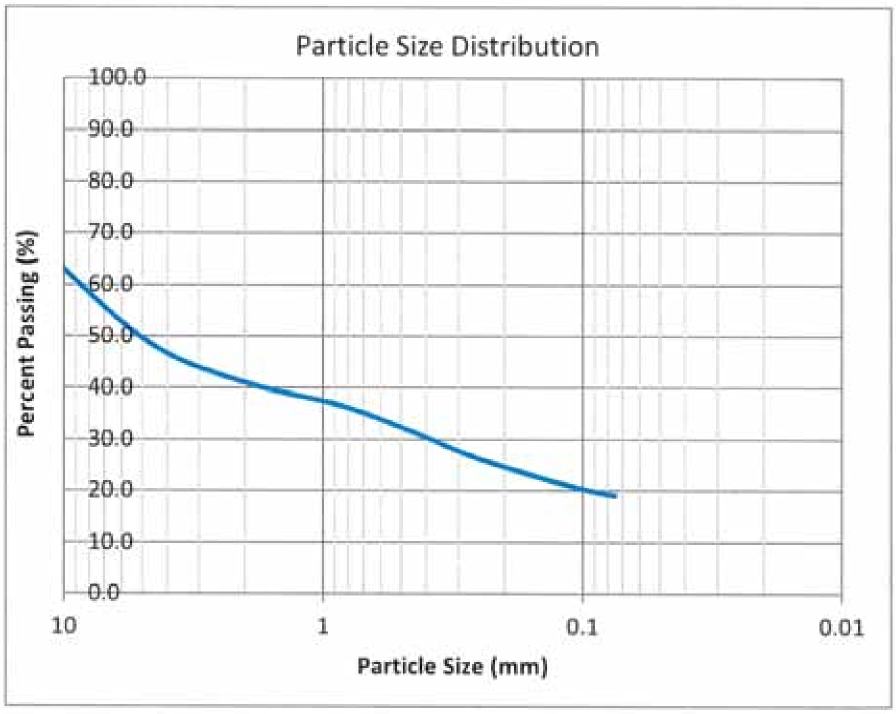
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

TB-2a-7 @ 32'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	67.6
No. 4	4.75	48.9
No. 10	2	41.0
No. 20	0.85	36.5
No. 40	0.425	31.0
No. 60	0.25	26.2
No. 140	0.106	20.6
No. 200	0.075	19.1
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	32.4
Course Sand (<No. 4-No.10):	26.6
Medium Sand (<No. 10-No. 40):	10.0
Fine Sand (<No. 40-No. 200):	11.9
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
32.4	48.5	19.1

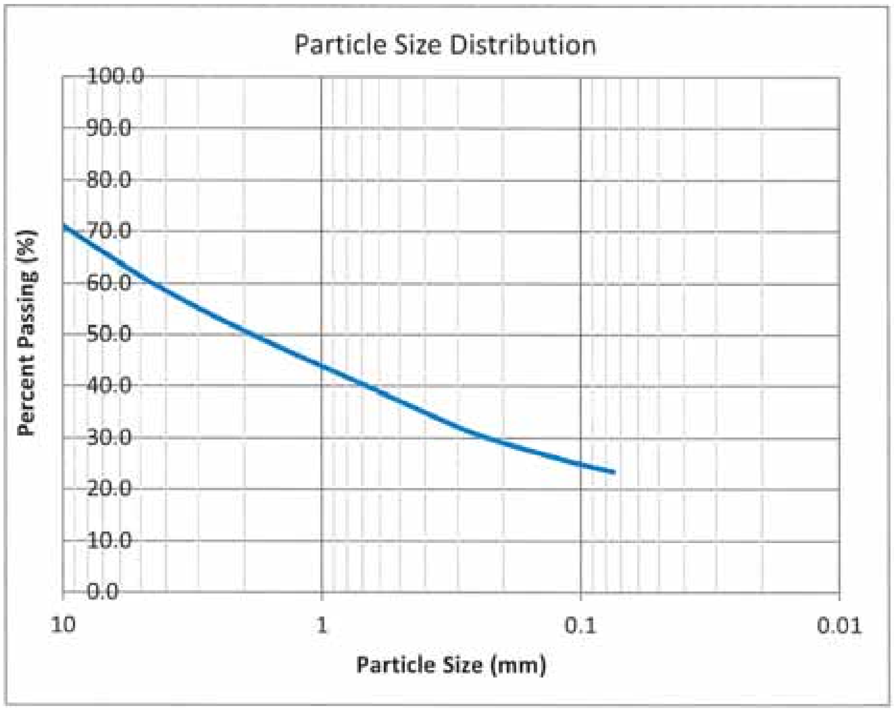
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B14-3 @ 13-14'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	74.2
No. 4	4.75	60.7
No. 10	2	50.8
No. 20	0.85	42.3
No. 40	0.425	35.7
No. 60	0.25	30.6
No. 140	0.106	25.2
No. 200	0.075	23.4
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



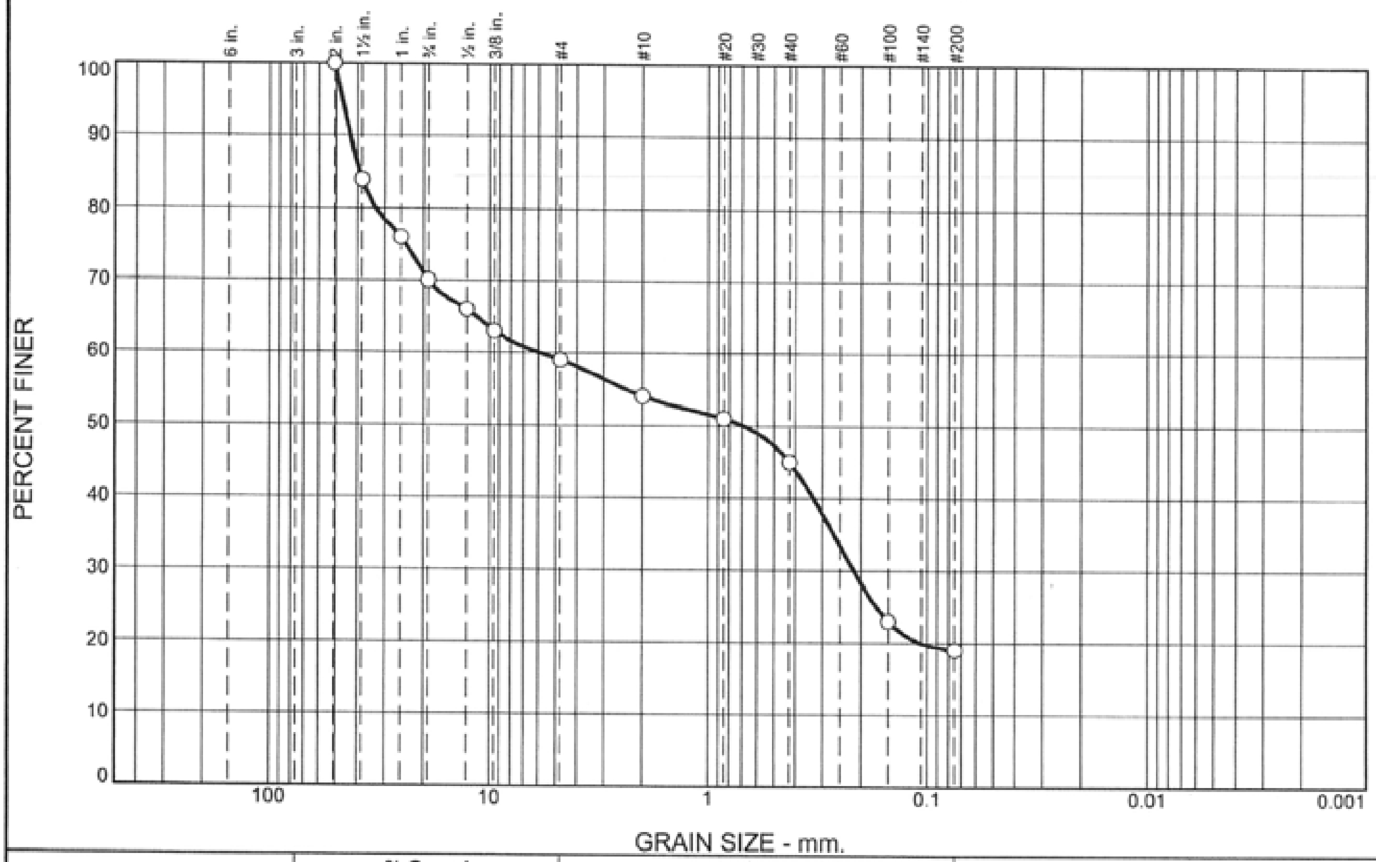
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	25.8
Course Sand (<No. 4-No.10):	23.5
Medium Sand (<No. 10-No. 40):	15.1
Fine Sand (<No. 40-No. 200):	12.2
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
25.8	50.8	23.4

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	30.0	11.0	5.0	9.0	26.0	19.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2"	100.0		
1.5"	84.0		
1"	76.0		
0.75"	70.0		
0.5"	66.0		
0.375"	63.0		
#4	59.0		
#10	54.0		
#20	51.0		
#40	45.0		
#100	23.0		
#200	19.0		

* (no specification provided)

Material Description

(#30365)

Atterberg Limits

PL=
LL=
PI=

Coefficients

D_{90}= 43.1591
D_{85}= 39.0447
D_{60}= 5.9401
D_{50}= 0.6875
D_{30}= 0.2160
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS=
AASHTO=

Remarks

Location: B16-1
Depth: 4'

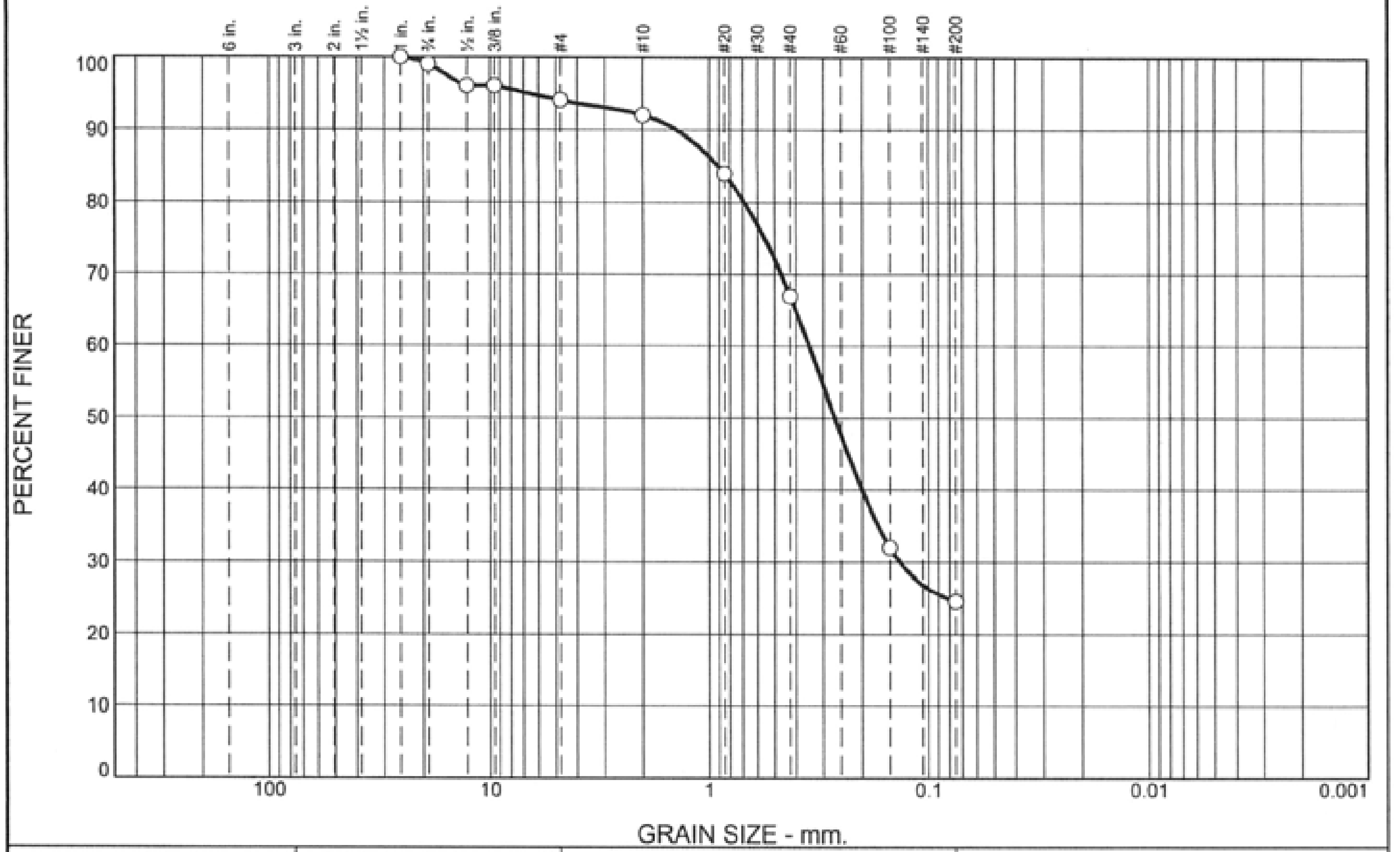
Date: 2/3/17

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21
Figure #30365

Tested By: Adame
Checked By: Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	1.0	5.0	2.0	25.0	42.4	24.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
0.75"	99.0		
0.5"	96.0		
0.375"	96.0		
#4	94.0		
#10	92.0		
#20	84.0		
#40	67.0		
#100	32.0		
#200	24.6		

* (no specification provided)

Material Description

(#30366)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 1.4047 D_{85}= 0.9048 D_{60}= 0.3480
D_{50}= 0.2663 D_{30}= 0.1352 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Location: B16-2
Depth: 5'

Date: 2/2/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30366

Tested By: Woodard **Checked By:** Collins

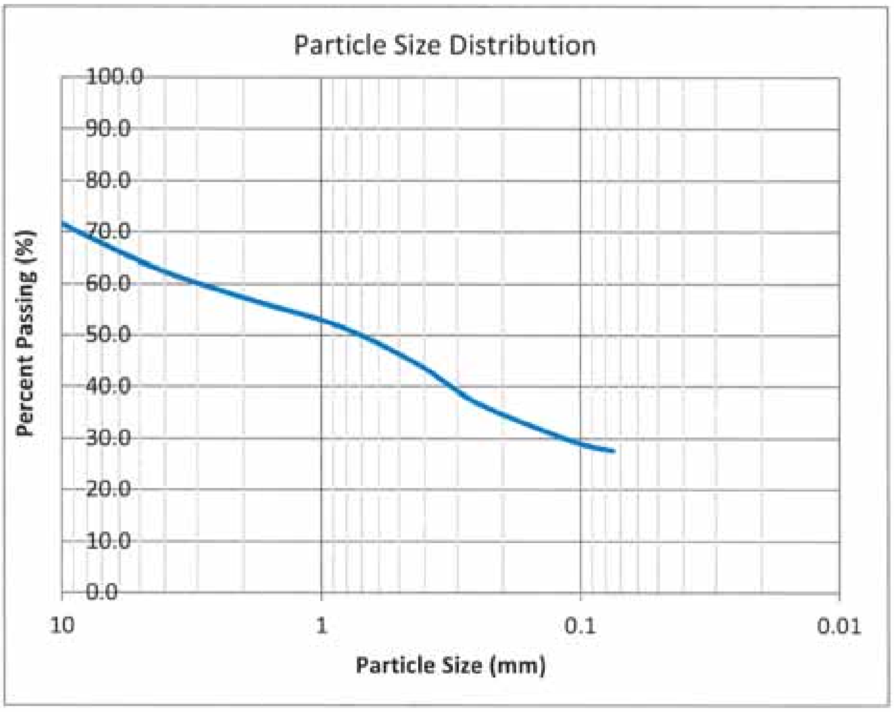
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B16-4 @ 15-16.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	74.0
No. 4	4.75	63.8
No. 10	2	57.3
No. 20	0.85	51.7
No. 40	0.425	44.3
No. 60	0.25	36.9
No. 140	0.106	29.4
No. 200	0.075	27.6
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



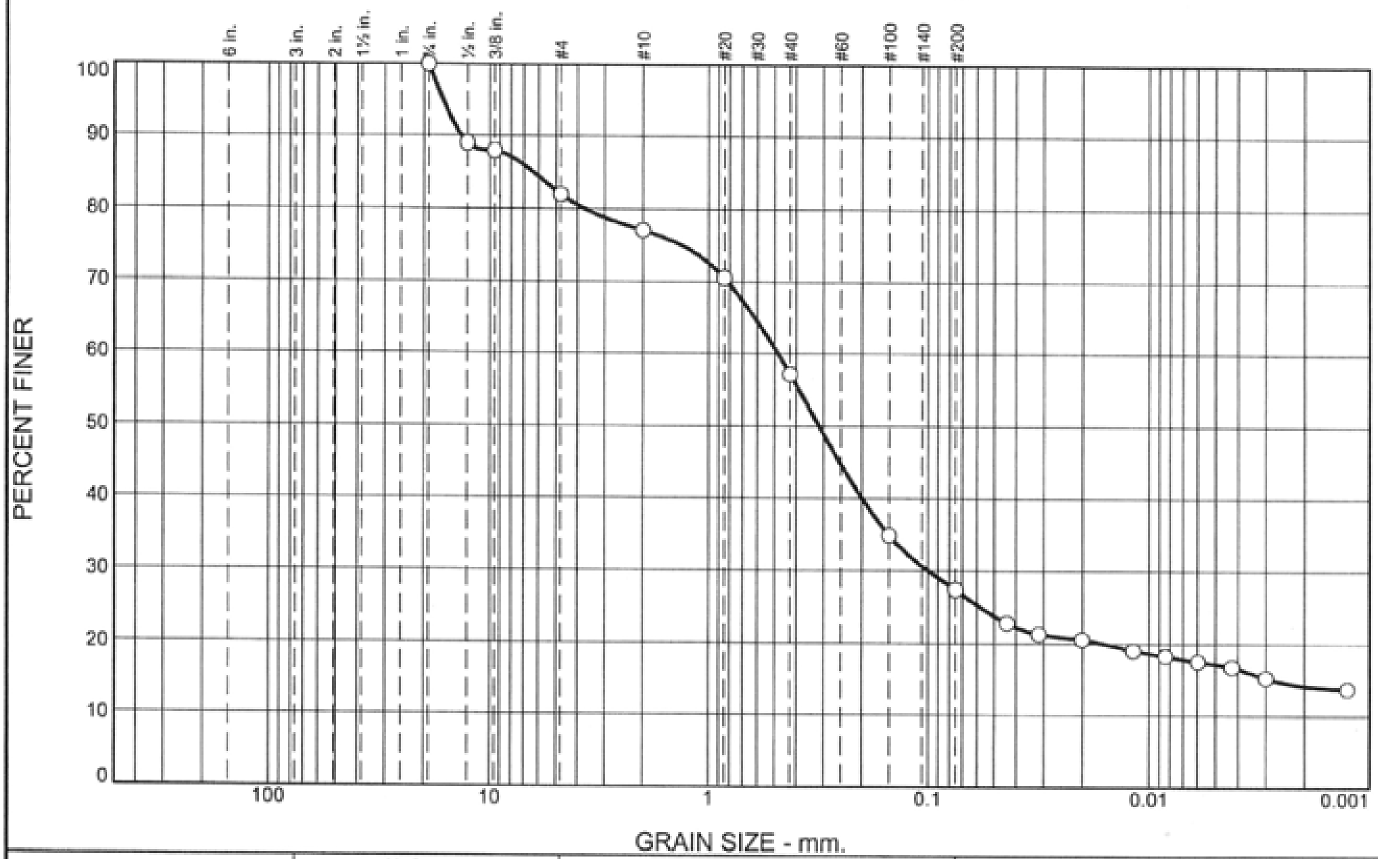
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	26.0
Course Sand (<No. 4-No.10):	16.7
Medium Sand (<No. 10-No. 40):	13.0
Fine Sand (<No. 40-No. 200):	16.7
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
26.0	46.4	27.6

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	18.0	5.0	19.9	29.7	13.3	14.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	89.0		
0.375"	88.0		
#4	82.0		
#10	77.0		
#20	70.5		
#40	57.1		
#100	34.9		
#200	27.4		

* (no specification provided)

Material Description

SC, (#30368)

Atterberg Limits

PL= 22.3 LL= 30.6 PI= 8.3

Coefficients

D_{90}= 13.5407 D_{85}= 6.2971 D_{60}= 0.4832
D_{50}= 0.3139 D_{30}= 0.0996 D_{15}= 0.0028
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO= A-2-4(0)

Remarks

Location: B17-1
Depth: 5'

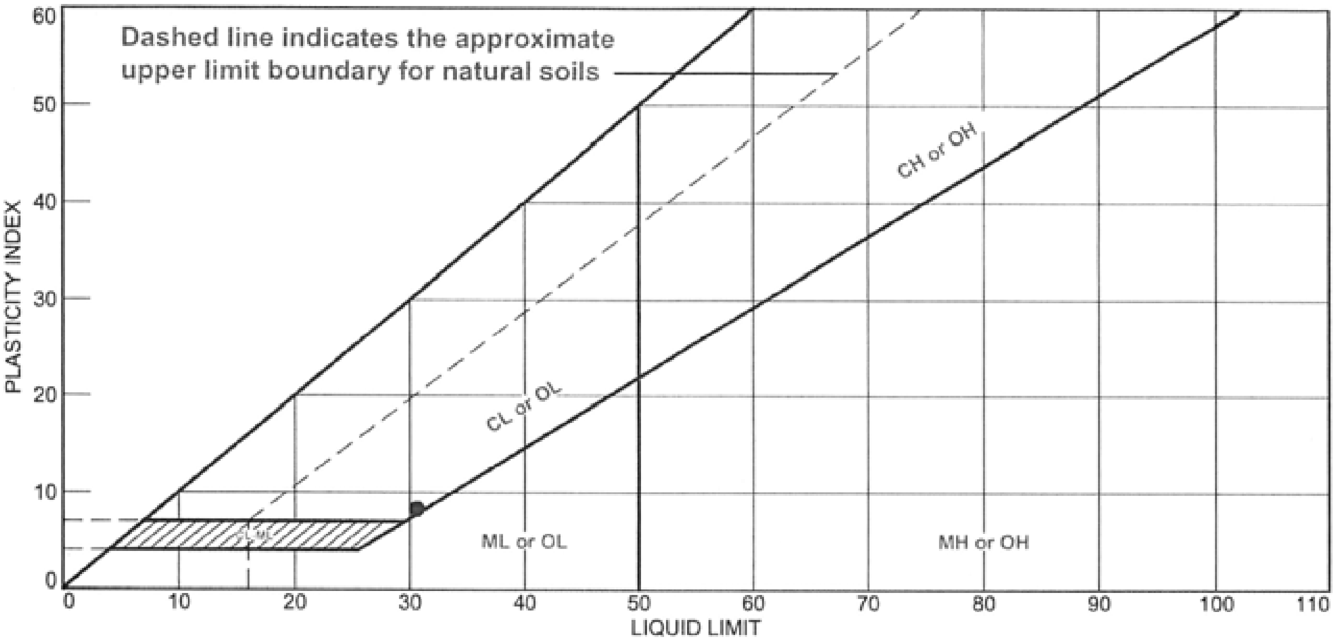
Date: 2/6/17

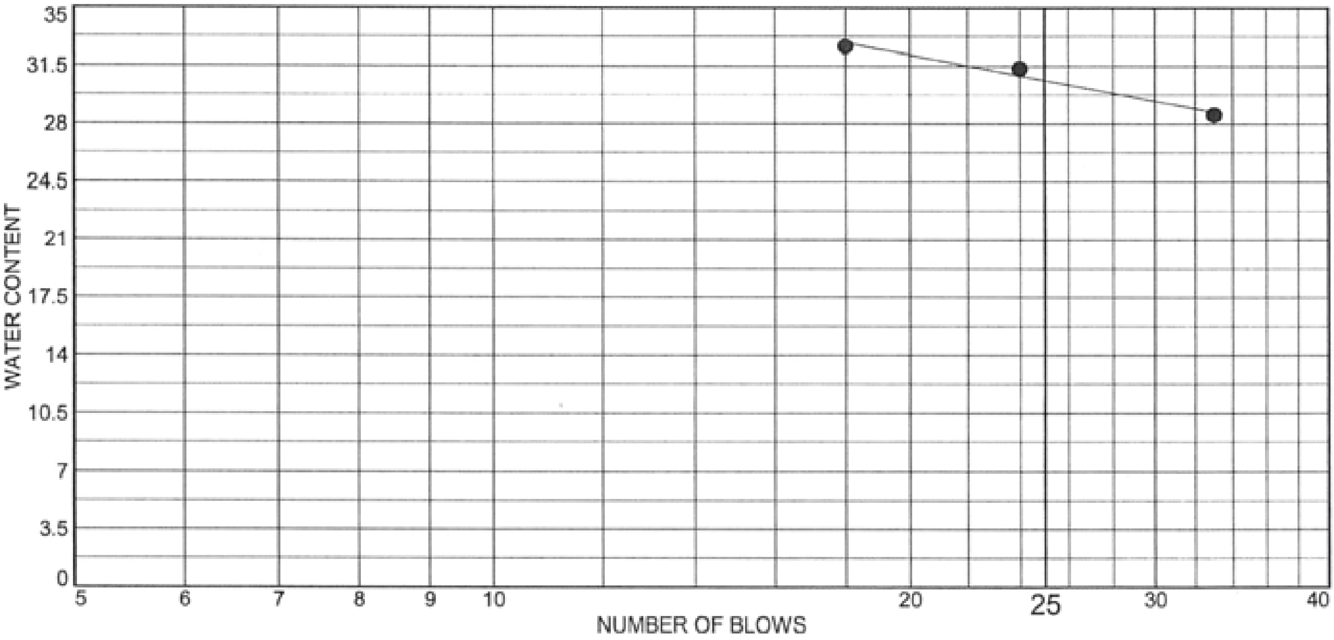
amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline
Project No: 5015-15-0030.21 **Figure** #30368

Tested By: Gibson **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SC, (#30368)	30.6	22.3	8.3	57.1	27.4	SC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

Location: B17-1
Depth: 5'

Remarks:

amec

Figure

Tested By: Iacovera **Checked By:** Collins

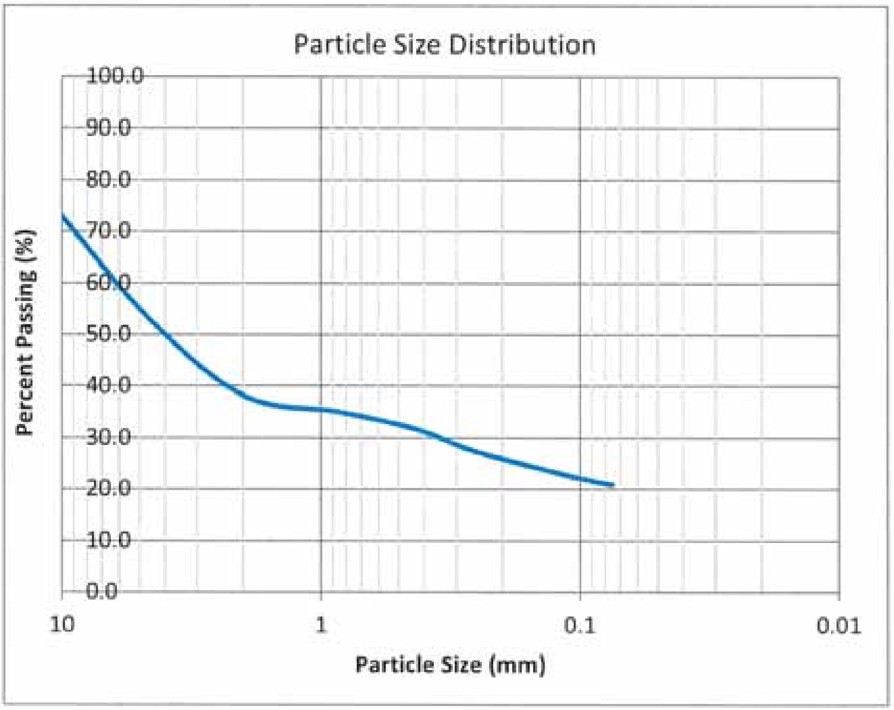
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B17-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	78.8
No. 4	4.75	53.8
No. 10	2	38.2
No. 20	0.85	35.0
No. 40	0.425	31.7
No. 60	0.25	27.3
No. 140	0.106	22.4
No. 200	0.075	20.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	21.2
Course Sand (<No. 4-No.10):	40.6
Medium Sand (<No. 10-No. 40):	6.5
Fine Sand (<No. 40-No. 200):	10.8
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
21.2	58.0	20.8

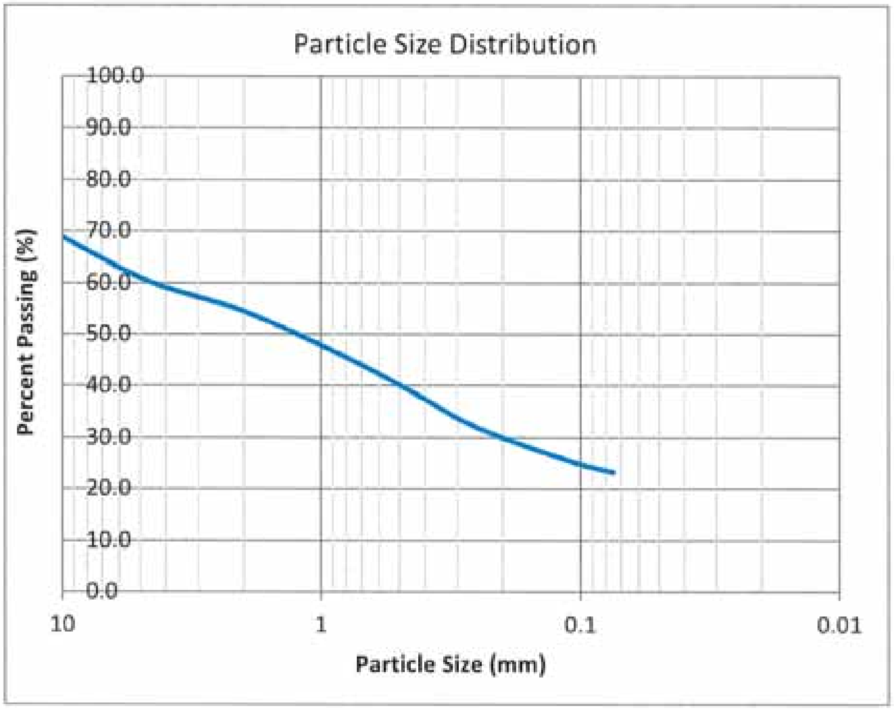
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B18-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	88.0
3/4"	19	77.4
3/8"	12.5	71.4
No. 4	4.75	60.5
No. 10	2	54.5
No. 20	0.85	46.1
No. 40	0.425	38.2
No. 60	0.25	31.9
No. 140	0.106	25.2
No. 200	0.075	23.2
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	22.6
Fine Gravel (<3/4"- No. 4):	5.9
Course Sand (<No. 4-No.10):	16.9
Medium Sand (<No. 10-No. 40):	16.3
Fine Sand (<No. 40-No. 200):	15.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
28.6	48.2	23.2

HDR

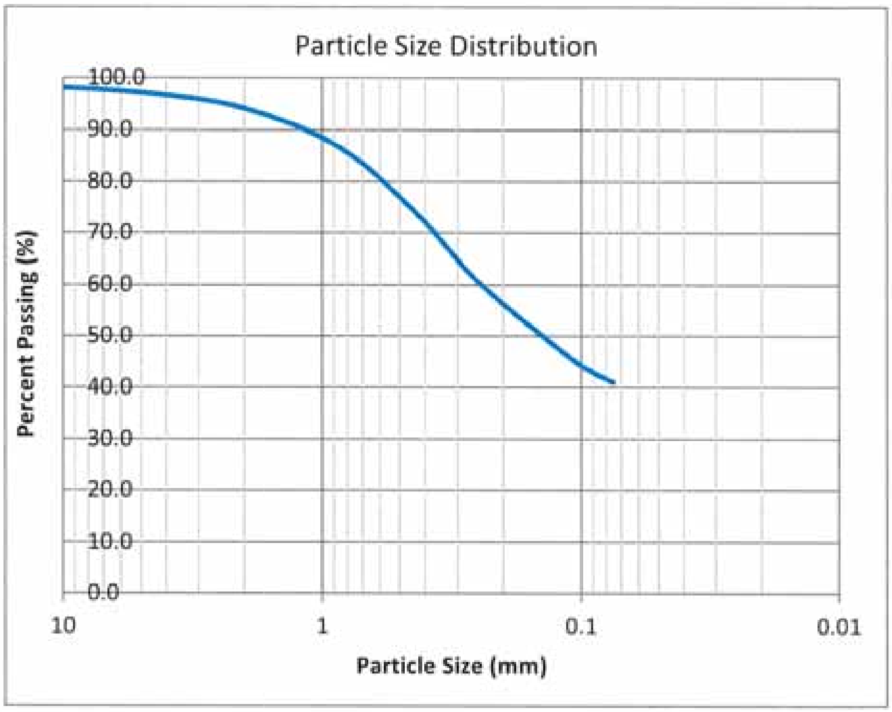
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B19-3 @ 14-15'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	98.4
No. 4	4.75	97.0
No. 10	2	94.1
No. 20	0.85	86.5
No. 40	0.425	73.4
No. 60	0.25	60.6
No. 140	0.106	45.1
No. 200	0.075	41.0
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



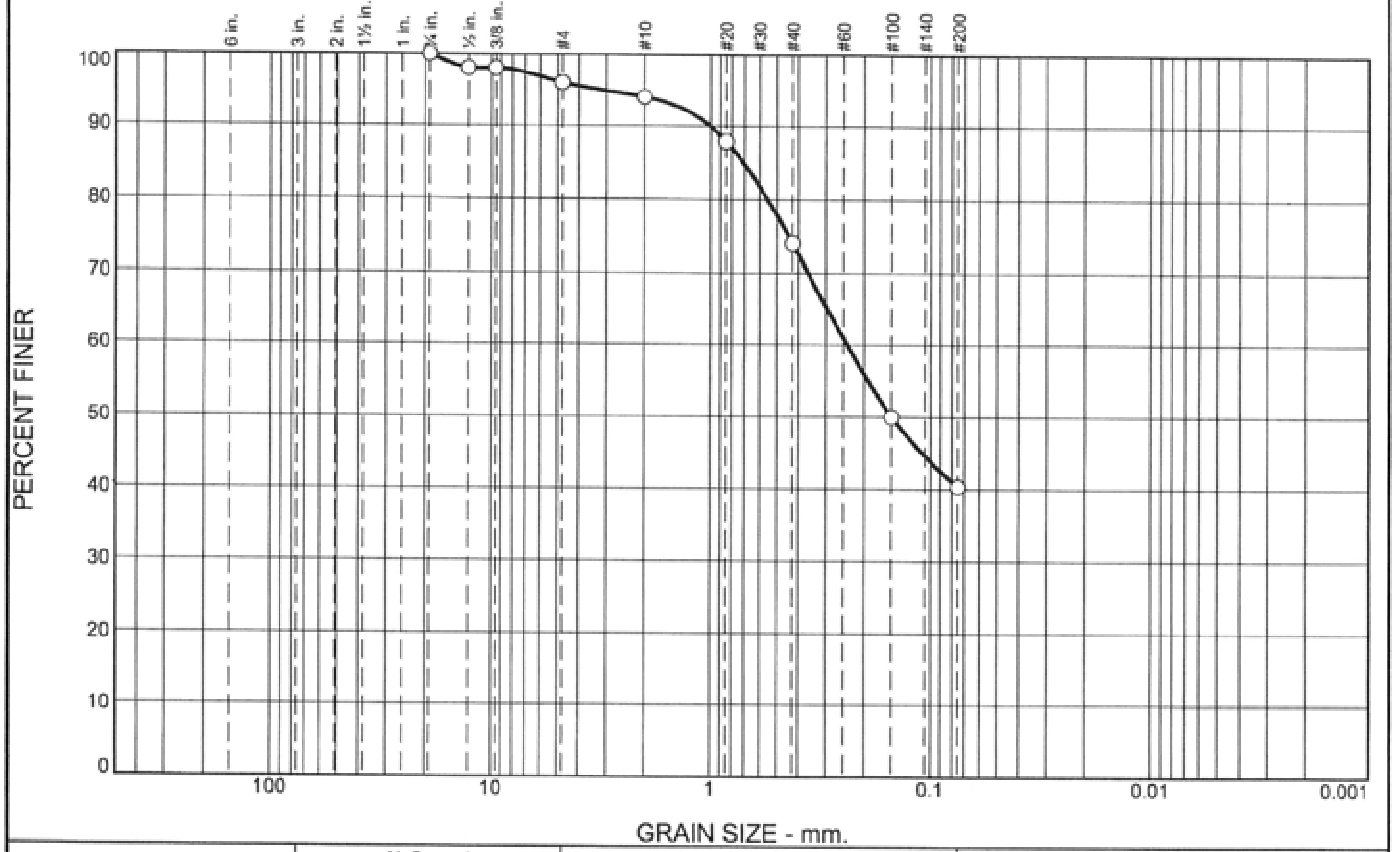
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	1.6
Course Sand (<No. 4-No.10):	4.3
Medium Sand (<No. 10-No. 40):	20.6
Fine Sand (<No. 40-No. 200):	32.5
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
1.6	57.4	41.0

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.0	2.0	20.0	33.7	40.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	98.0		
0.375"	98.0		
#4	96.0		
#10	94.0		
#20	88.0		
#40	74.0		
#100	50.0		
#200	40.3		

* (no specification provided)

Material Description

Clayey Sand, (#30373)

Atterberg Limits

PL=
LL=
PI=

Coefficients

D_{90}= 0.9987
D_{85}= 0.7063
D_{60}= 0.2401
D_{50}= 0.1500
D_{30}=
D_{15}=
D_{10}=
C_u=
C_c=

Classification

USCS=
AASHTO=

Remarks

Location: B19-4
Depth: 15'

Date: 2/2/17

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21
Figure #30373

Tested By: Woodard
Checked By: Collins

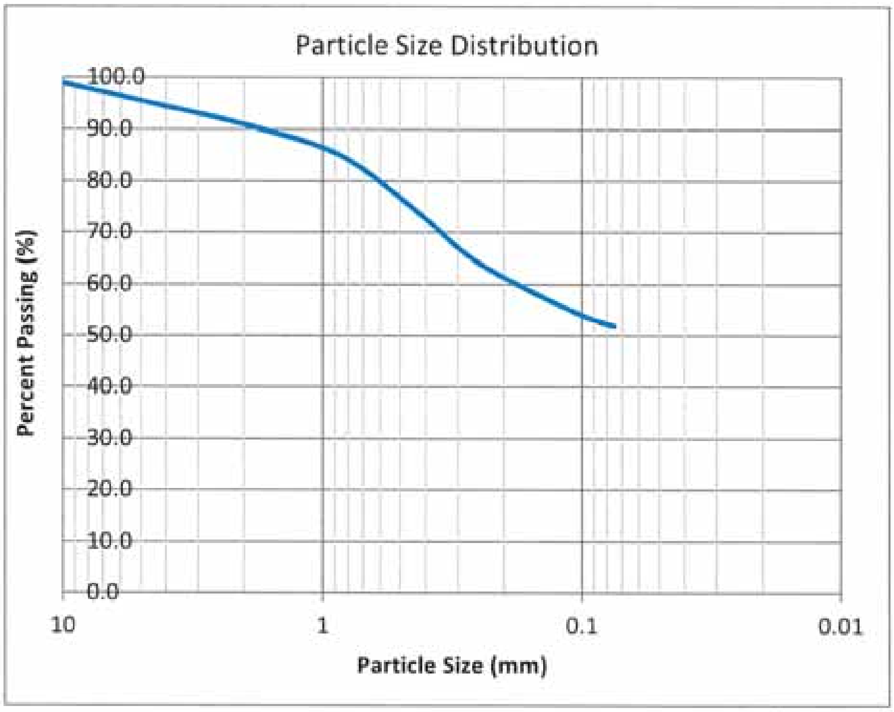
Table 2 - Particle-Size Analysis of Soils

TerraCosta Consulting Group
North City Conveyance System
17-0038LAB
20-Feb-17

Sample ID

B20-2 @ 10-11.5'

Sieve	Size (mm)	Percent Passing
3"	76.2	100.0
2"	50.8	100.0
1 1/2"	38.1	100.0
1"	25	100.0
3/4"	19	100.0
3/8"	12.5	100.0
No. 4	4.75	95.2
No. 10	2	90.9
No. 20	0.85	84.9
No. 40	0.425	73.8
No. 60	0.25	64.2
No. 140	0.106	54.5
No. 200	0.075	51.8
Hydro 1	NA	NA
Hydro 2	NA	NA
Hydro 3	NA	NA
Hydro 4	NA	NA
Hydro 5	NA	NA
Hydro 6	NA	NA
Hydro 7	NA	NA



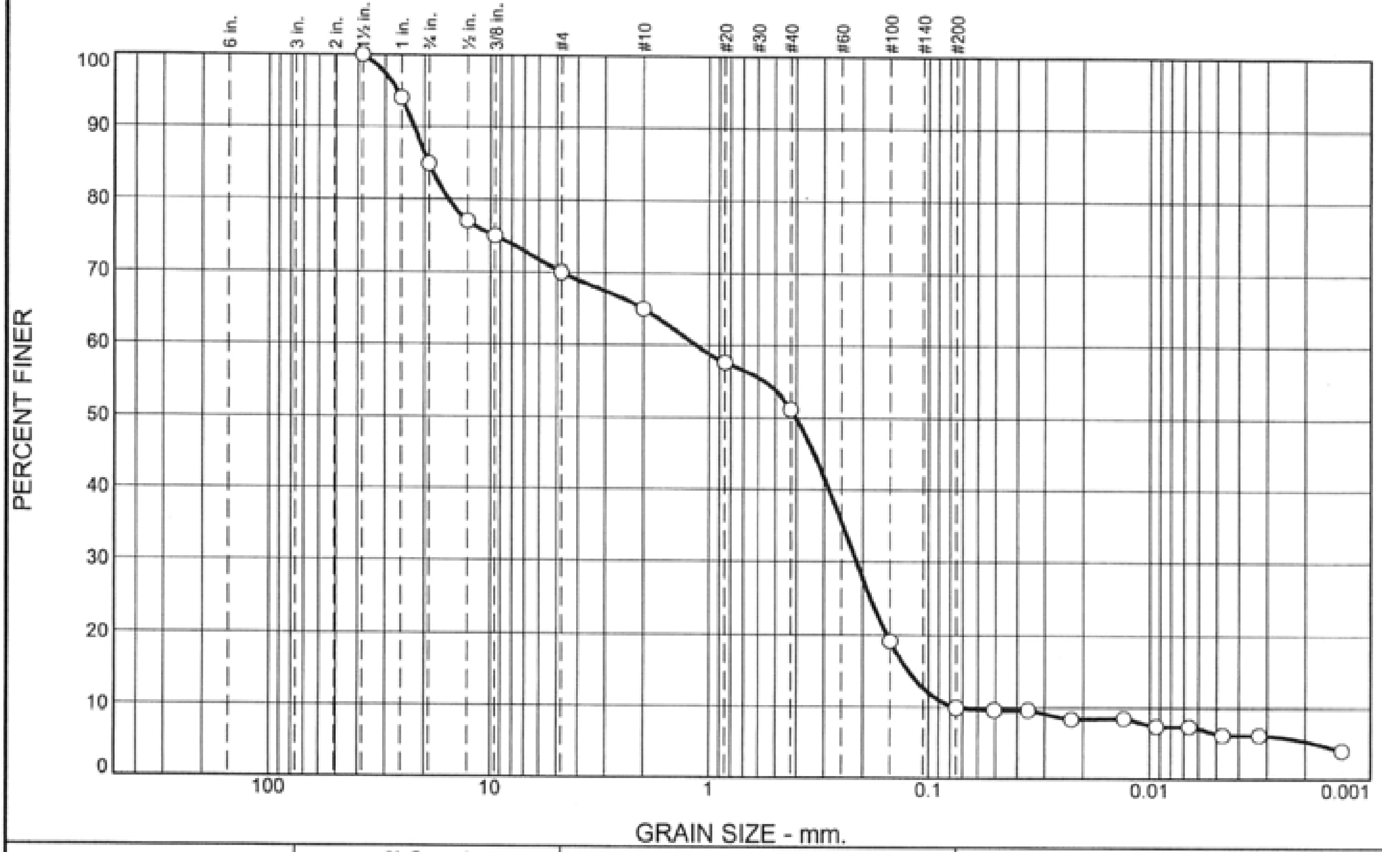
ASTM Composition of Total Sample per ASTM D422-63

	%
Course Gravel (3"-3/4"):	0.0
Fine Gravel (<3/4"- No. 4):	0.0
Course Sand (<No. 4-No.10):	9.1
Medium Sand (<No. 10-No. 40):	17.0
Fine Sand (<No. 40-No. 200):	22.0
Silt (<No. 200-0.005 mm):	NA
Clay (<0.005mm-0.001 mm):	NA

Particle Distribution Summary (%)

Gravel	Sand	Silt/Clay
0.0	48.2	51.8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	15.0	15.0	5.0	13.8	41.3	4.6	5.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1"	94.0		
0.75"	85.0		
0.5"	77.0		
0.375"	75.0		
#4	70.0		
#10	65.0		
#20	57.6		
#40	51.2		
#100	19.1		
#200	9.9		

* (no specification provided)

Material Description

SP-SC, (#30325)

Atterberg Limits

PL= 18.7 LL= 45.4 PI= 26.7

Coefficients

D_{90}= 22.2374 D_{85}= 19.0500 D_{60}= 1.1522
D_{50}= 0.4017 D_{30}= 0.2134 D_{15}= 0.1241
D_{10}= 0.0770 C_u= 14.97 C_c= 0.51

Classification

USCS= SP-SC AASHTO= A-2-7(0)

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: B20: a1@2', a2@5',a3@10',a4@15' & a5@18.5' combined
Depth: 2'-18.5' combined

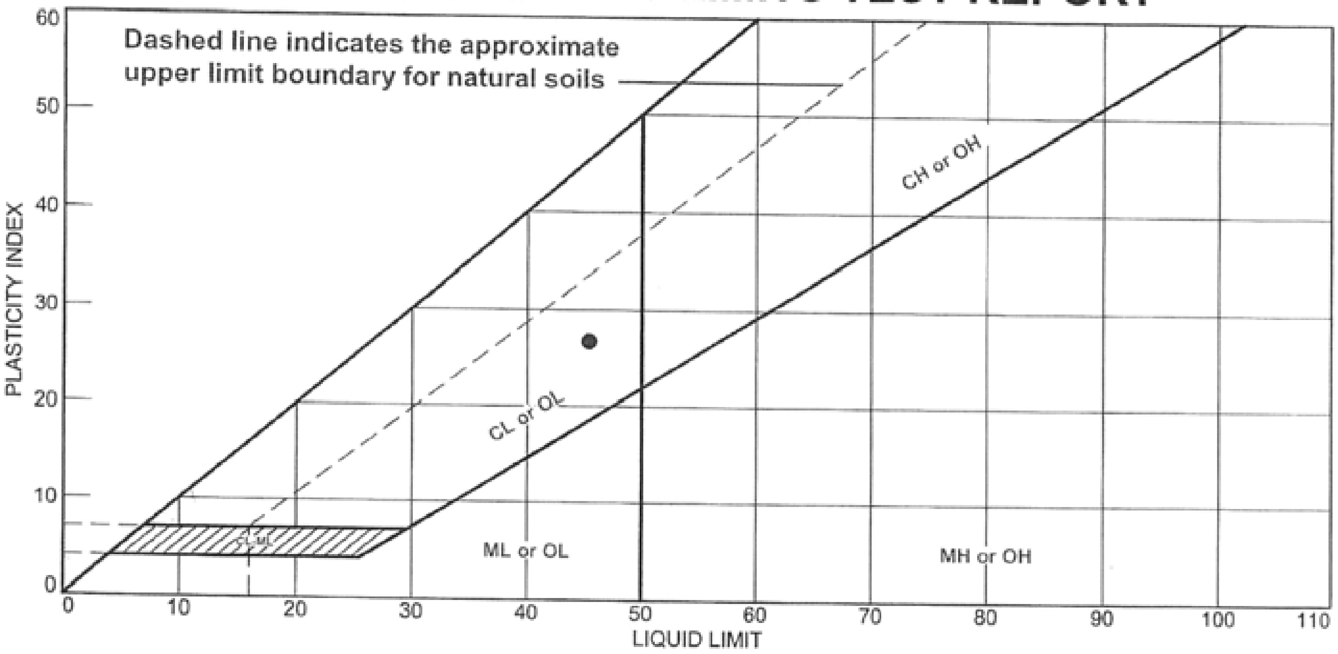
Date: 2/10/17

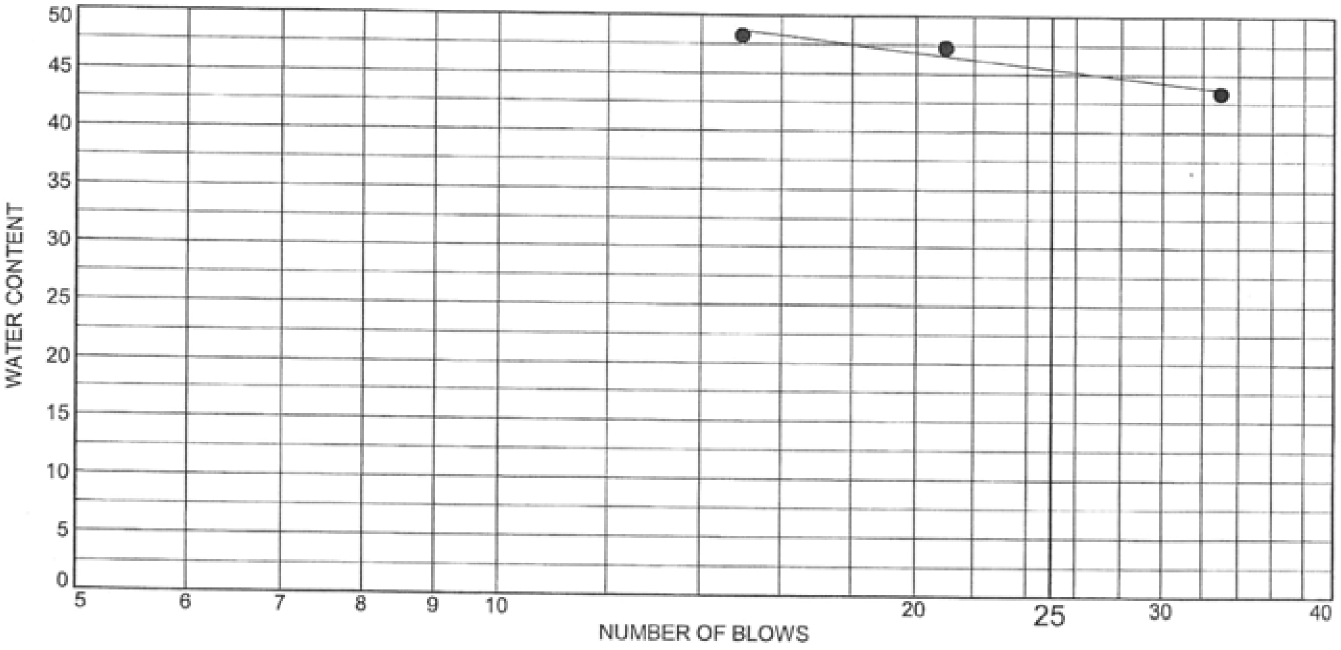
Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30325

Tested By: Gibson **Checked By:** Collins

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	SP-SC, (#30325)	45.4	18.7	26.7	51.2	9.9	SP-SC

Project No. 5015-15- **Client:** Terra Costa Consulting Group, Inc.

Project: #2934 NCCS Pipeline

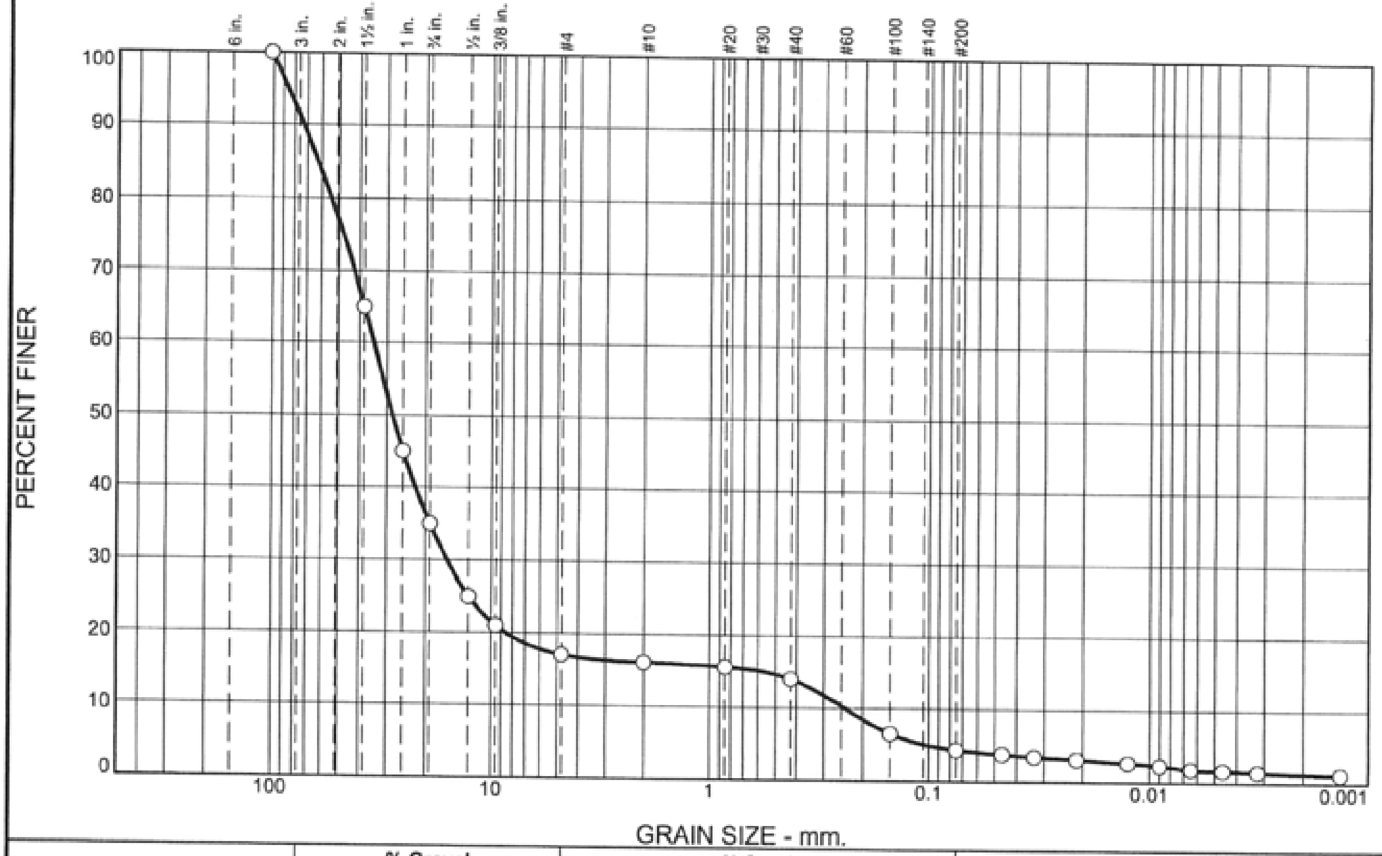
Location: B20: a1@2', a2@5',a3@10',a4@15' & a5@18.5' combined
Depth: 2'-18.5' combined

Remarks:

Figure #30325

Tested By: Iacovera **Checked By:** Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
8.6	56.4	18.0	1.0	2.1	9.5	3.1	1.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
4"	100.0		
1.5"	65.0		
1"	45.0		
0.75"	35.0		
0.5"	25.0		
0.375"	21.0		
#4	17.0		
#10	16.0		
#20	15.5		
#40	13.9		
#100	6.6		
#200	4.4		

* (no specification provided)

Material Description

Poorly Graded Gravel, GP (#30322), Core

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 72.7336 D_{85}= 62.4016 D_{60}= 34.4924
D_{50}= 28.3325 D_{30}= 15.9095 D_{15}= 0.5865
D_{10}= 0.2418 C_u= 142.67 C_c= 30.35

Classification

USCS= GP AASHTO=

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: TB-4b
Depth: 77'

Date: 2/2/2017

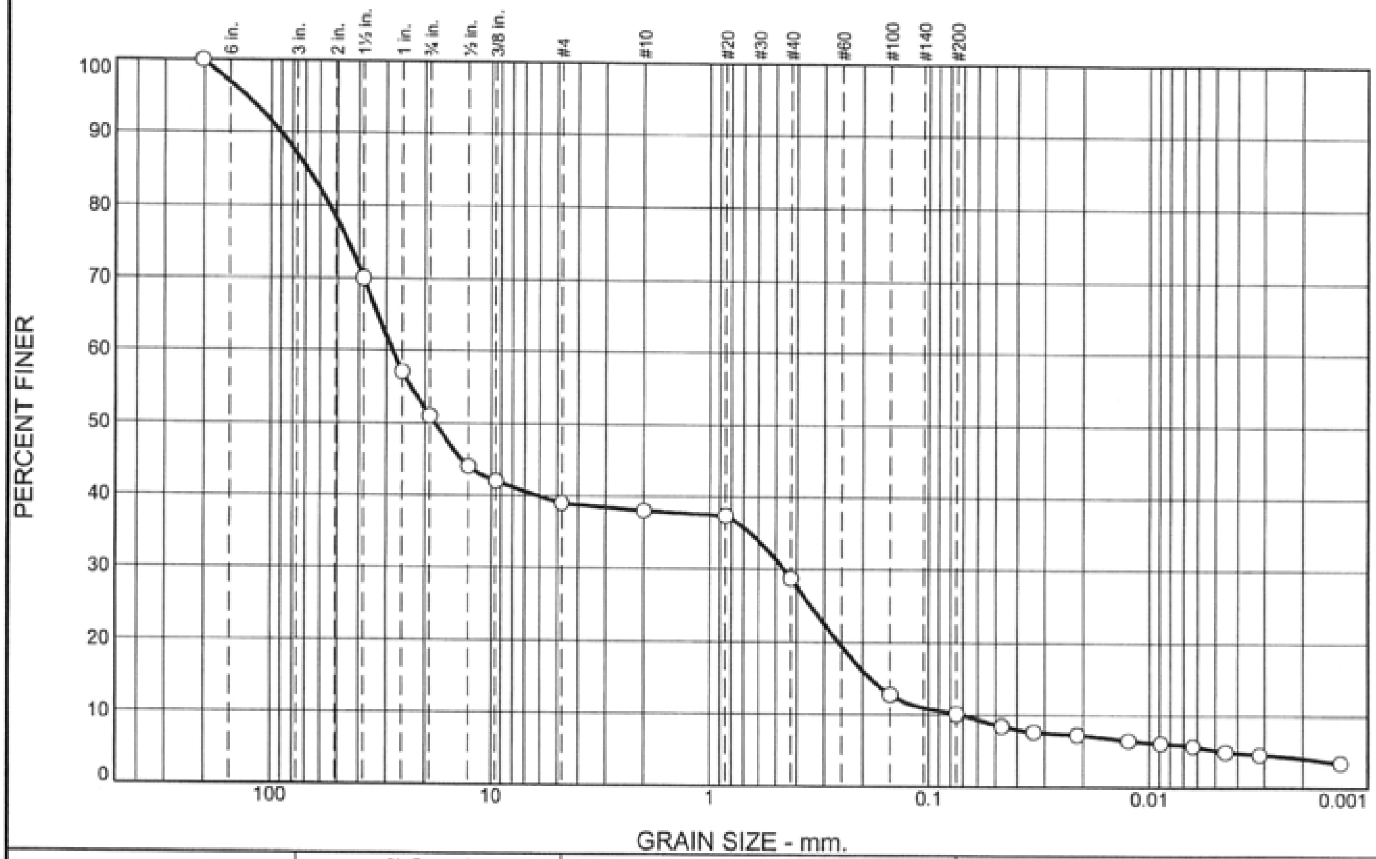
amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30322

Tested By: J.Iacovera/M.Gibson **Checked By:** L. Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
12.9	36.1	12.0	1.0	9.3	18.6	6.0	4.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
8"	100.0		
1.5"	70.0		
1"	57.0		
0.75"	51.0		
0.5"	44.0		
0.375"	42.0		
#4	39.0		
#10	38.0		
#20	37.4		
#40	28.7		
#100	12.7		
#200	10.1		

* (no specification provided)

Material Description

Poorly Graded Gravel with Silt, GP-GM (#30323), (Core)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 90.4037 D_{85}= 68.1988 D_{60}= 28.2072
D_{50}= 18.0872 D_{30}= 0.4596 D_{15}= 0.1859
D_{10}= 0.0724 C_u= 389.63 C_c= 0.10

Classification

USCS= GP-GM AASHTO=

Remarks

Assumed specific gravity of 2.65 used for hydrometer calculations and soil particles smaller than 0.002mm have been classified as clay.

Location: TB-4c
Depth: 86'

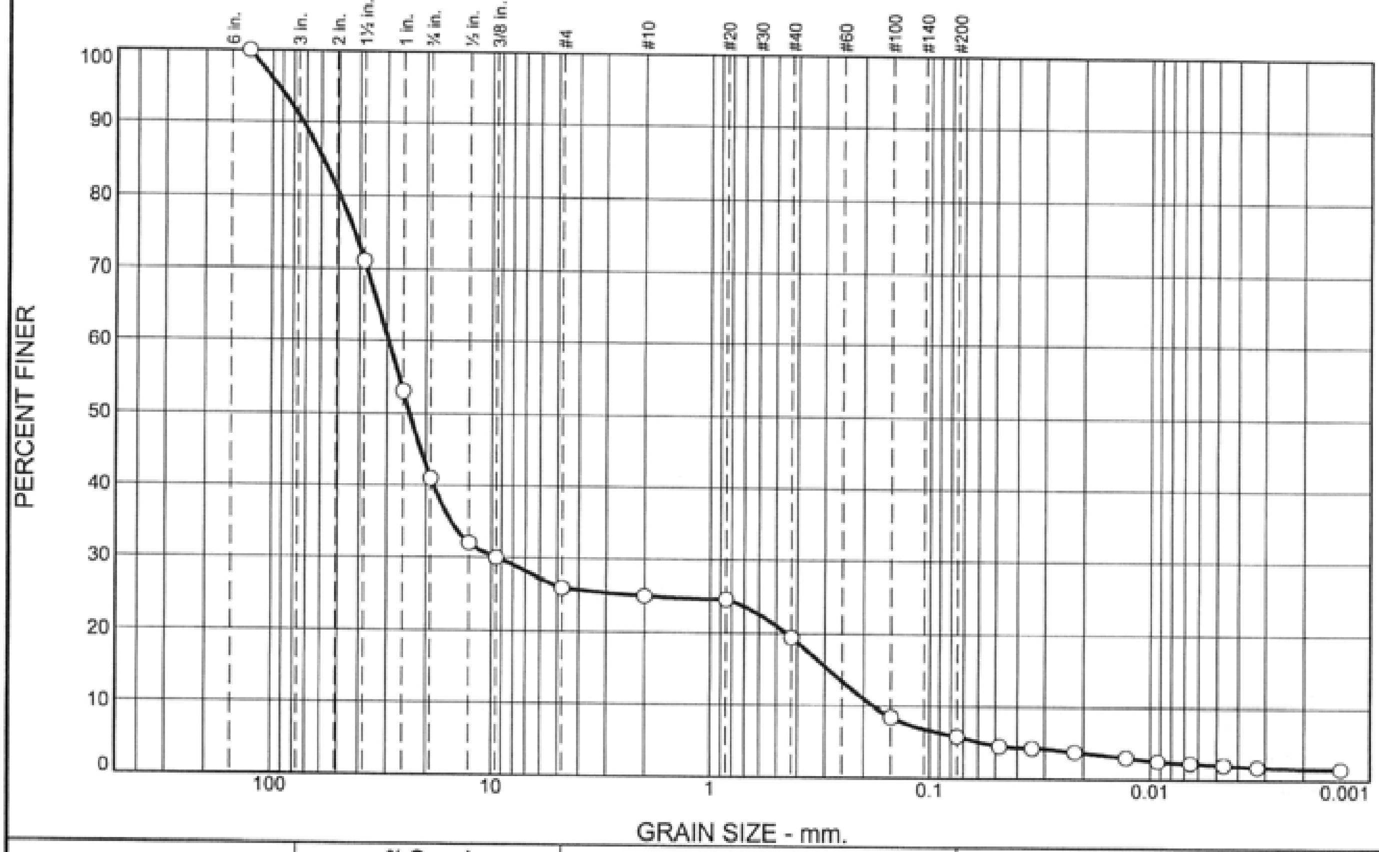
Date: 2/2/2017

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30323

Tested By: J.Iacovera/M.Gibson **Checked By:** L. Collins

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
9.0	50.0	15.0	1.0	5.6	13.5	4.3	1.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
5"	100.0		
1.5"	71.0		
1"	53.0		
0.75"	41.0		
0.5"	32.0		
0.375"	30.0		
#4	26.0		
#10	25.0		
#20	24.6		
#40	19.4		
#100	8.5		
#200	5.9		

* (no specification provided)

Material Description

Poorly Graded Gravel, GP (#30324), (Core)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 72.7442 D_{85}= 58.7810 D_{60}= 29.5692
D_{50}= 23.7761 D_{30}= 9.5250 D_{15}= 0.2890
D_{10}= 0.1815 C_u= 162.91 C_c= 16.90

Classification

USCS= GP AASHTO=

Remarks

Location: TB-4d
Depth: 93'

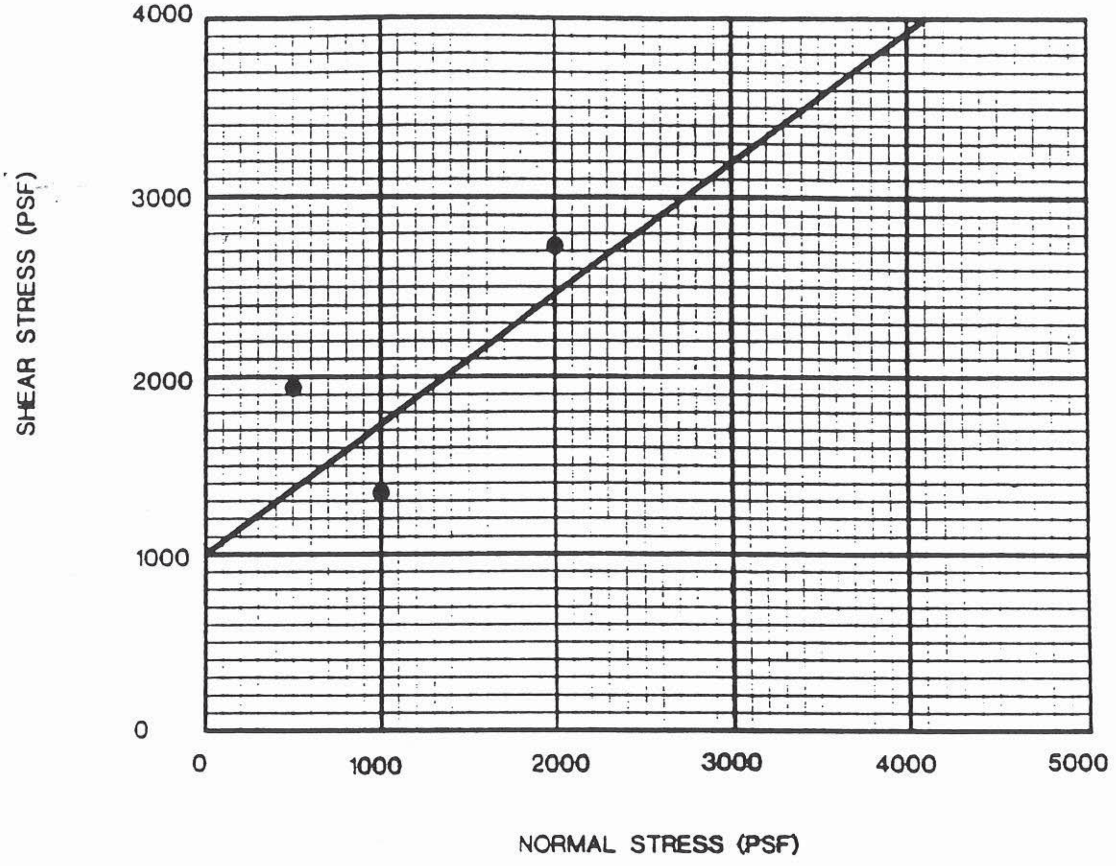
Date: 2/2/2017

amec

Client: Terra Costa Consulting Group, Inc.
Project: #2934 NCCS Pipeline

Project No: 5015-15-0030.21 **Figure** #30324

Tested By: J.Iacovera/M.Gibson **Checked By:** L. Collins



Description	Symbol	Boring Number	Shear Strength	Depth (Feet)	Cohesion (PSF)	Friction Angle	Soil Type
Clayey fine-grained SANDSTONE	●	B-11	PEAK	5.0-6.5	1000	37 °	FORMATION

9 274674

Ninyo & Moore

DIRECT SHEAR TEST RESULTS

MIRAMAR ROAD SUBSYSTEM EXTENSION
SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
102746-01	2/95	B-7

SAMPLE LOCATION	SAMPLE DEPTH (FT.)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN.)	EXPANSION INDEX	EXPANSION POTENTIAL
B-11	2.5-4.5	10.0	108.8	20.5	0.0376	38	Low

B:2746FS

PERFORMED IN ACCORDANCE WITH UBC STANDARD 29-2.

Ninyo & Moore

EXPANSION INDEX TEST RESULTS

MIRAMAR ROAD SUBSYSTEM EXTENSION
SAN DIEGO, CALIFORNIA

PROJECT NO.	DATE	FIGURE
102746-01	2/95	B-8

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* DATE: *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-3	5-5.5	9										GM
	9-10.0											GM, Tested by others
	10-11.5	10										GM
	15-16.5	15										Ss
	17-18.0	9	107.9									Ss
	19-20.5	17										Ss
B-4	3-4.0	11	103.2									SC (Fill)

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* DATE: *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-4	5-5.1	13										GM
	9-10.0	---										GM, Poor Sample
	10-10.5	12										GM
	15-16.5	13										Ss
	18-19.0	11										Ss
B-5	3-4.0	7										CL
	5-5.5	5										GM
	5-10.0	---					26		57	17		GM
	10-10.5	4										GM
	10-11.0	---										GM, Tested by others
	15-16.0	8	107.4								DS, C	Ss
	20-21.5	14										Ss
	25-26.5	9										Ss
	30-31.0	9										Ss
	35-36.0	8										Ss
B-6	0-0.5	15										SP/SC (Talus/Fill)
	1-1.5	14										SP/SC (Talus/Fill)
	2-3.5	10					22		78	0		Ss
	5-.60	10	104.4								C	Ss
	6-7.0	11										Ss

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* **DATE:** *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-7	0-0.5	8					5		43	54*	*Mixed 0 - 1.5 feet	SP (Talus/Fill)
	1-1.5	11										SP (Talus/Fill)
	2-3.5	9										Ss
	4-5.0	8	108.0								DS	Ss
	5.5-6.5	10										Ss
	7-8.0	9										Ss
B-8	5-5.5	14										CL (Fill)
	10-10.5	9										GM
	11-11.5	11	96.6									GM
	15-15.5	10										GM
	16.5-17.5	19	96.6									Ss
	20-20.5	9										Ss
	25-25.5	15										Ss
B-9	2-5.0	---					23		48	29		Mixed GP/SC
	6.5-6.8	8										GP
	10-11.0	---										GP, Tested by others
	10-10.3	7										GP
	12-13.0	4	108.6								DS	GP
	15-16.5	13										GP
	19.0	---										GP, Tested by others
	21-21.5	15										Ss

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* DATE: *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-9	22.5-23.0	10										Ss, Poor Sample
	26-26.2	15										Ss
B-10	2-2.5	7										SC
	5-5.2	11										GM
	9-9.3	7										GM
	10-10.2	6										GM
	15-16.0	7										Ss, Tested by others
	19-20.0	7	102.3									Ss
	20-20.5	---										GP
	25-25.5	11										GP
B-11	1-2.0	8										SP/SM (Fill)
	5-6.0	---										GP, Poor Sample
	6-6.3	4										GP
	7-7.5	4										GP
	10-10.2	7										GP
	12.5-13.5	8	90.5									Ss
	15-15.5	12										Ss
	17-18.0	11	105.8									Ss
	20-20.5	11										Ss
	25-25.3	5										Ss
	20-30.3	5										Ss

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* DATE: *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-12	2-2.5	5										SM (Fill)
	10-10.2	7										GM
	15-15.2	6										GM
	18-19.0	7										Ss
	21-21.2	10										Ss
	25-25.2	10										Ss
	30-30.2	5										Ss
	35-35.1	4										Ss
B-13	2-2.5	15										SC (Fill)
	5-5.2	11										GP
	10-10.2	7										GP
	15-15.5	7										GP
	20-20.5	6										GP, Poor Sample
B-14	2-3.5	15										CL
	5-5.5	10										GP
	9-10.0	---										Ss, tested by others
	10.5-11.0	11										Ss
	15-15.2	9										Ss
	20-20.3	7										Ss
B-15	2-3.0	14										SC (Fill)

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* **DATE:** *July 1994*

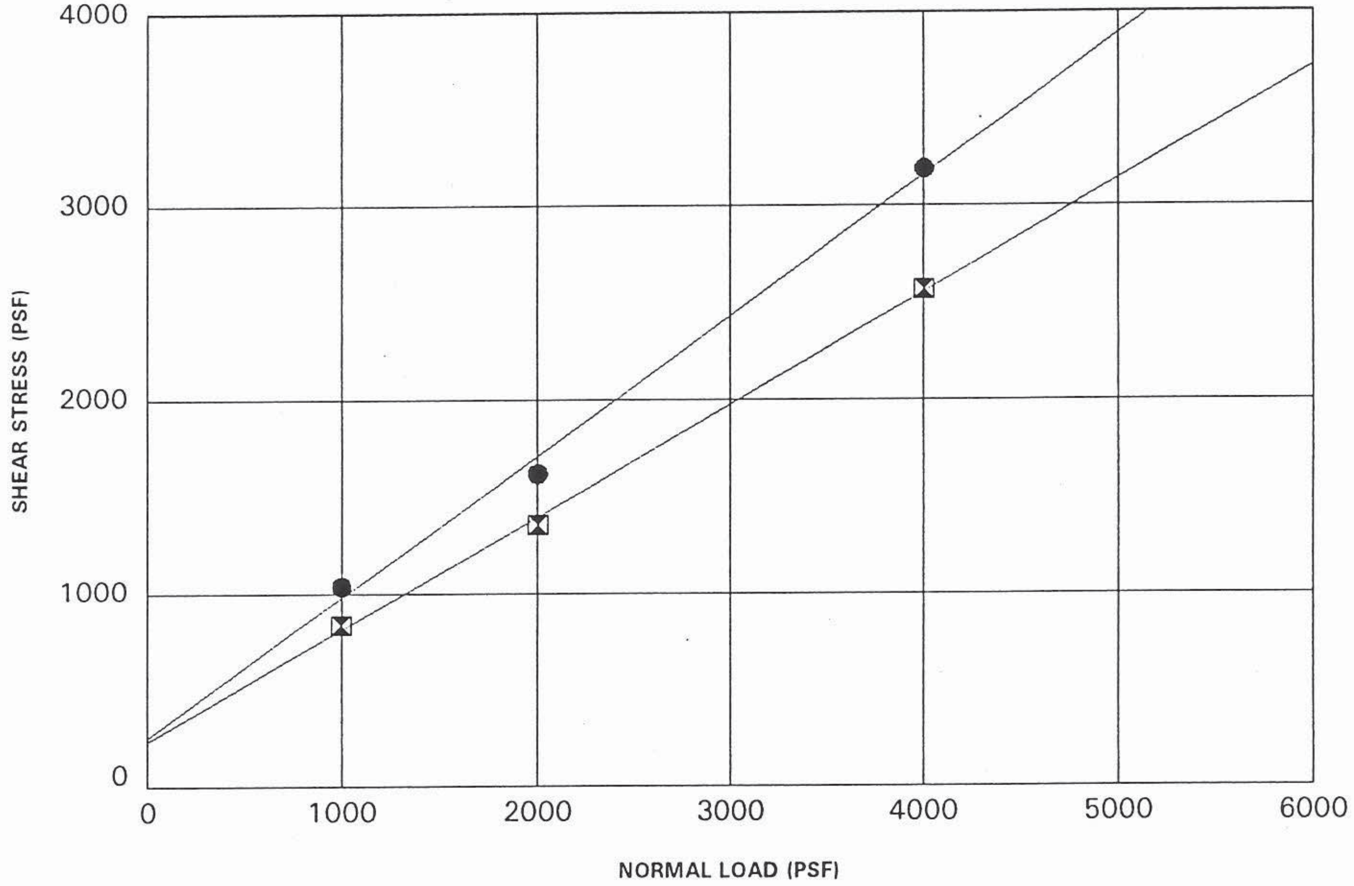
BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-15	2-5.0	---					25		47	28		SC (Fill)
	4-5.0	---										SC (Fill), Tested by others
	5-5.2	8										GP
	7-8.0	---										GP, Tested by others
	8-8.5	---										GP, Tested by others
	10-10.5	5										Ss
	12-13.0	12	94.1								C	Ss
	15-15.5	17										Ss
	19-20.0	20										Ss
B-16	2-3.5	11										CL/GP
	5-5.1	6										GP

GEOBASE

SUMMARY OF LABORATORY TEST RESULTS

PROJECT: *PENASQUITOS TRUNK SEWER RELIEF PROJECT, SAN DIEGO, CALIFORNIA* **PROJECT NO:** *P.155.04.01* DATE: *July 1994*

BORING	DEPTH (ft.)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS			PARTICLE SIZE DISTRIBUTION				OTHER TESTS	DESCRIPTION AND REMARKS
				LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAVEL (%)		
B-40	4-5.0	8										SP (Fill)
B-41	1-5.0	---										GM
	4-5.0	---										GM, Tested by others
	5-5.5	4										GM
	9-10.0	---										GM, Tested by others
	10-10.5	---										GM
	12-13.0	6										Ss
	15-16.0	---										Ss



SAMPLE DESCRIPTION: SANDSTONE (Ss)
BORING NO.: B-5
DEPTH INTERVAL (ft): 15.0 - 16.0
NOTES:

HEIGHT (in): 1.0
AREA (sq in): 4.58
STRAIN RATE (in/min): 0.005

DRY DENSITY (pcf): 107.4
INITIAL MOISTURE (%): 8
FINAL MOISTURE (%): 19

	PEAK ●	ULTIMATE ☒
COHESION (psf)	250	230
FRICTION ANGLE (deg)	36	30

GEOBASE

DIRECT SHEAR TEST RESULTS

Penasquitos Trunk Sewer Relief Project
San Diego, California

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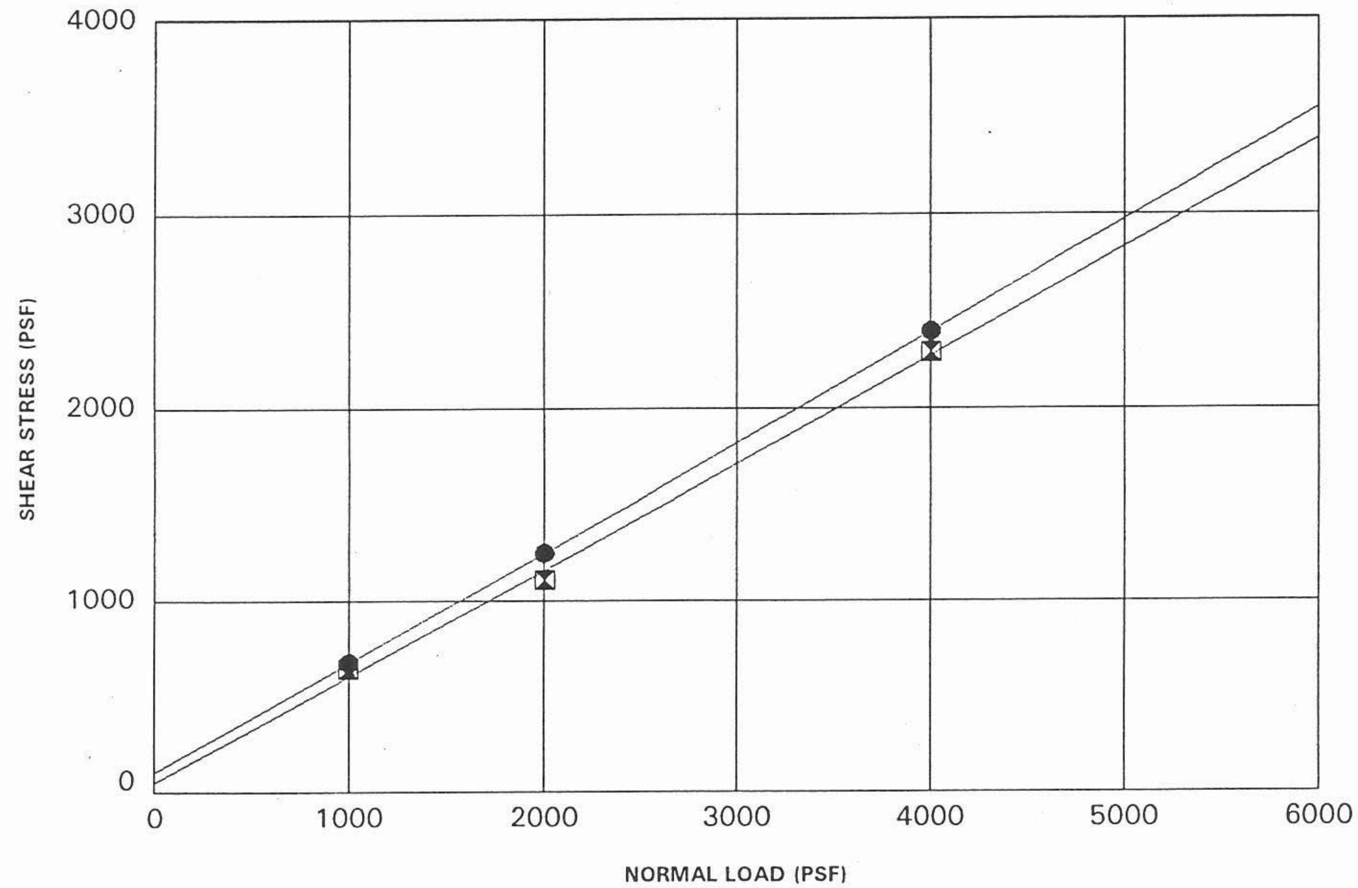
FIGURE C-2



SAMPLE DESCRIPTION: SANDSTONE (Ss)
BORING NO.: B-7
DEPTH INTERVAL (ft): 4.0 - 5.0
NOTES:

HEIGHT (in): 1.0
AREA (sq in): 4.56
STRAIN RATE (in/min): 0.005

DRY DENSITY (pcf): 108.0
INITIAL MOISTURE (%): 8
FINAL MOISTURE (%): 20

	PEAK ●	ULTIMATE ☒
COHESION (psf)	100	50
FRICTION ANGLE (deg)	35	29

GEOBASE

DIRECT SHEAR TEST RESULTS

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San Diego, California

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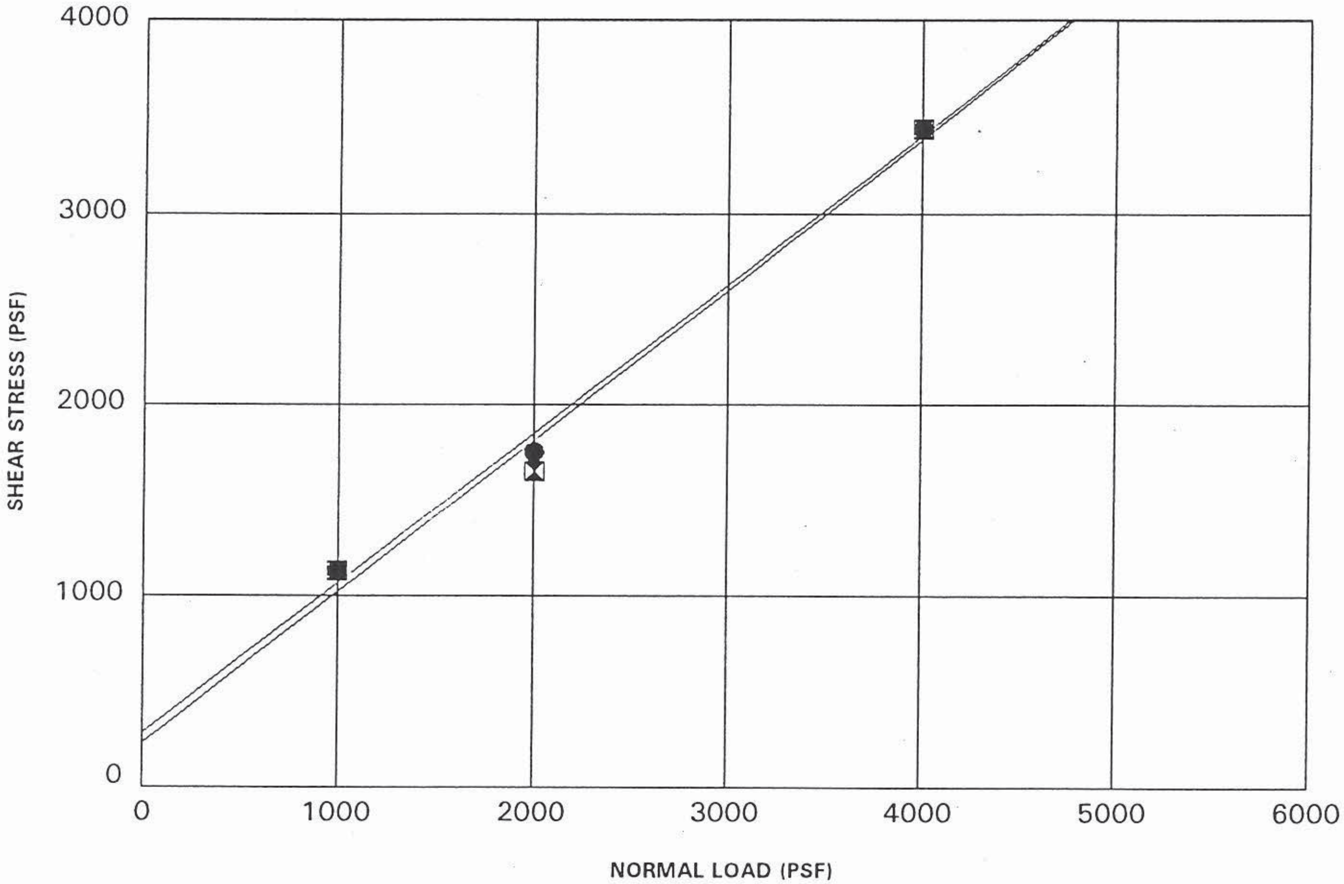
FIGURE C-3



						PEAK ●	ULTIMATE ☒
SAMPLE DESCRIPTION:	GRAVEL, brown, sand matrix (GP)	HEIGHT (in):	1.0	DRY DENSITY (pcf):	108.6		
BORING NO.:	B-9	AREA (sq in):	4.56	INITIAL MOISTURE (%):	4	COHESION (psf) 280	230
DEPTH INTERVAL (ft):	12.0 - 13.0	STRAIN RATE (in/min):	0.005	FINAL MOISTURE (%):	18	FRICTION ANGLE (deg) 38	38
NOTES:							

GEOBASE

DIRECT SHEAR TEST RESULTS

Penasquitos Trunk Sewer Relief Project
San Diego, California

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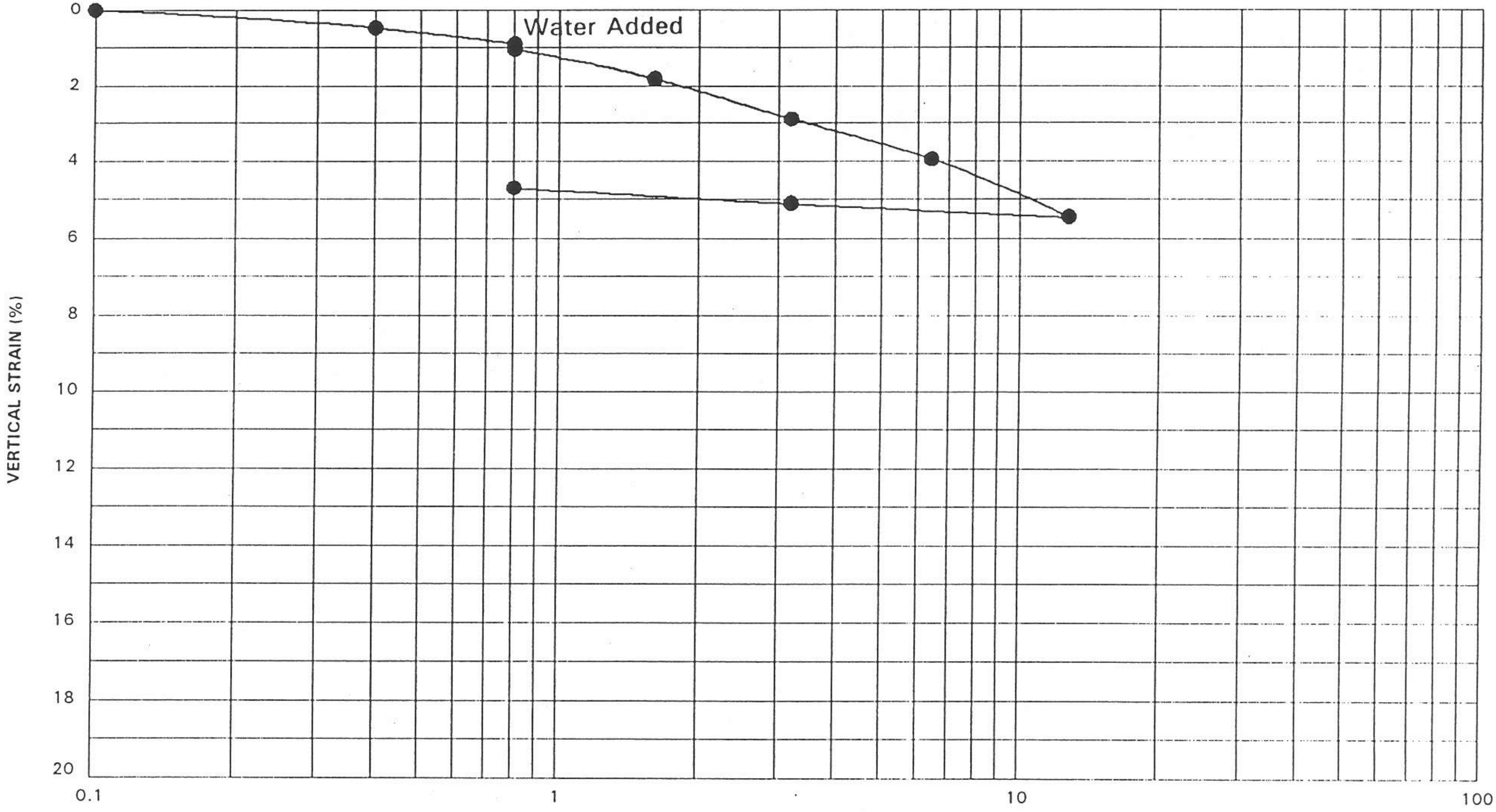
FIGURE C-4



BORING NO.: B-5

DEPTH (ft): 15.5

SAMPLE DESCRIPTION: SANDSTONE (Ss)

GEOBASE

CONSOLIDATION TEST RESULTS

Penasquitos Trunk Sewer Relief Project
San Diego, California

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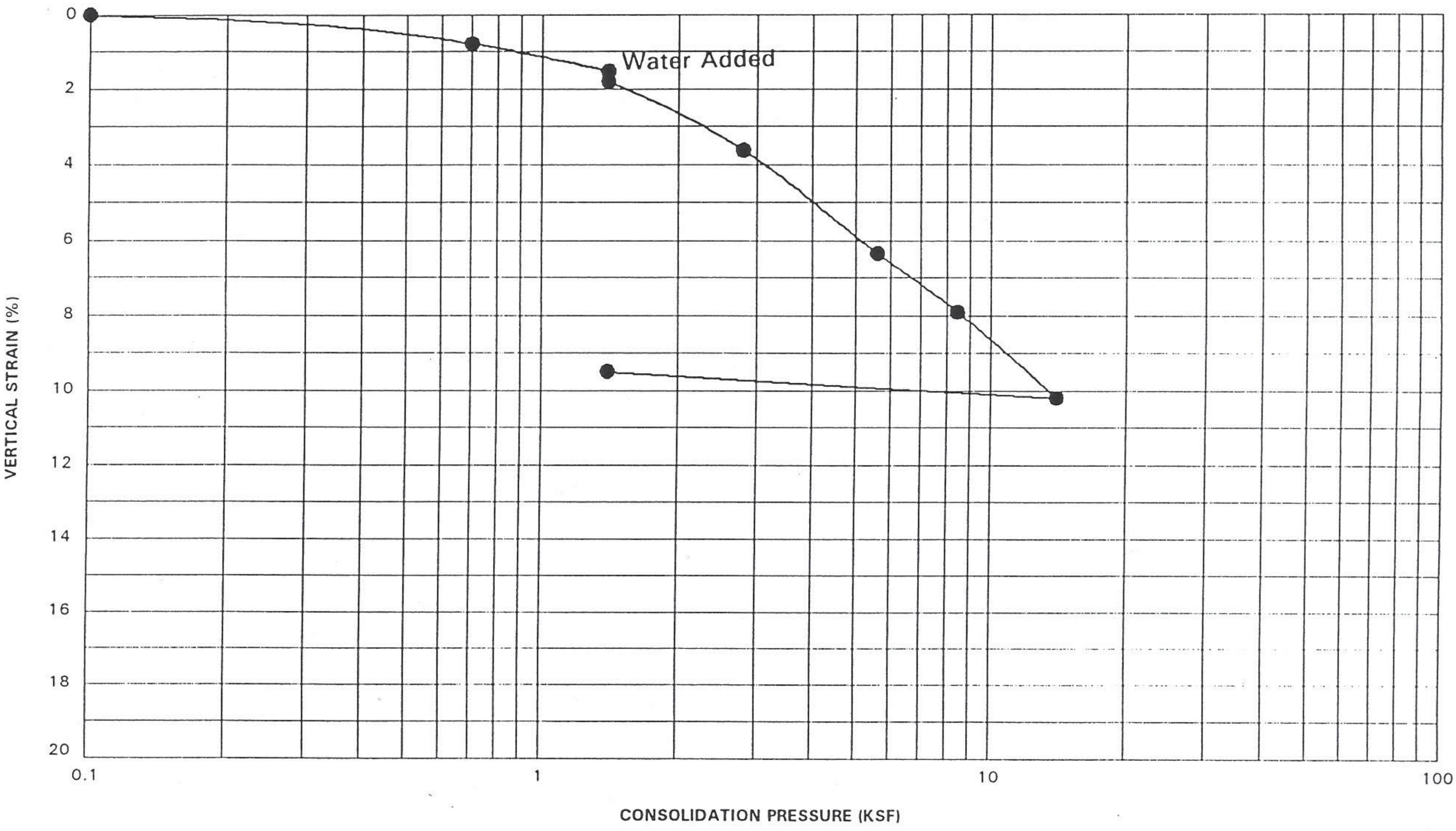
FIGURE C-10



BORING NO.: B-15
DEPTH (ft): 12.5
SAMPLE DESCRIPTION: SANDSTONE (Ss)

GEOBASE

CONSOLIDATION TEST RESULTS

Penasquitos Trunk Sewer Relief Project
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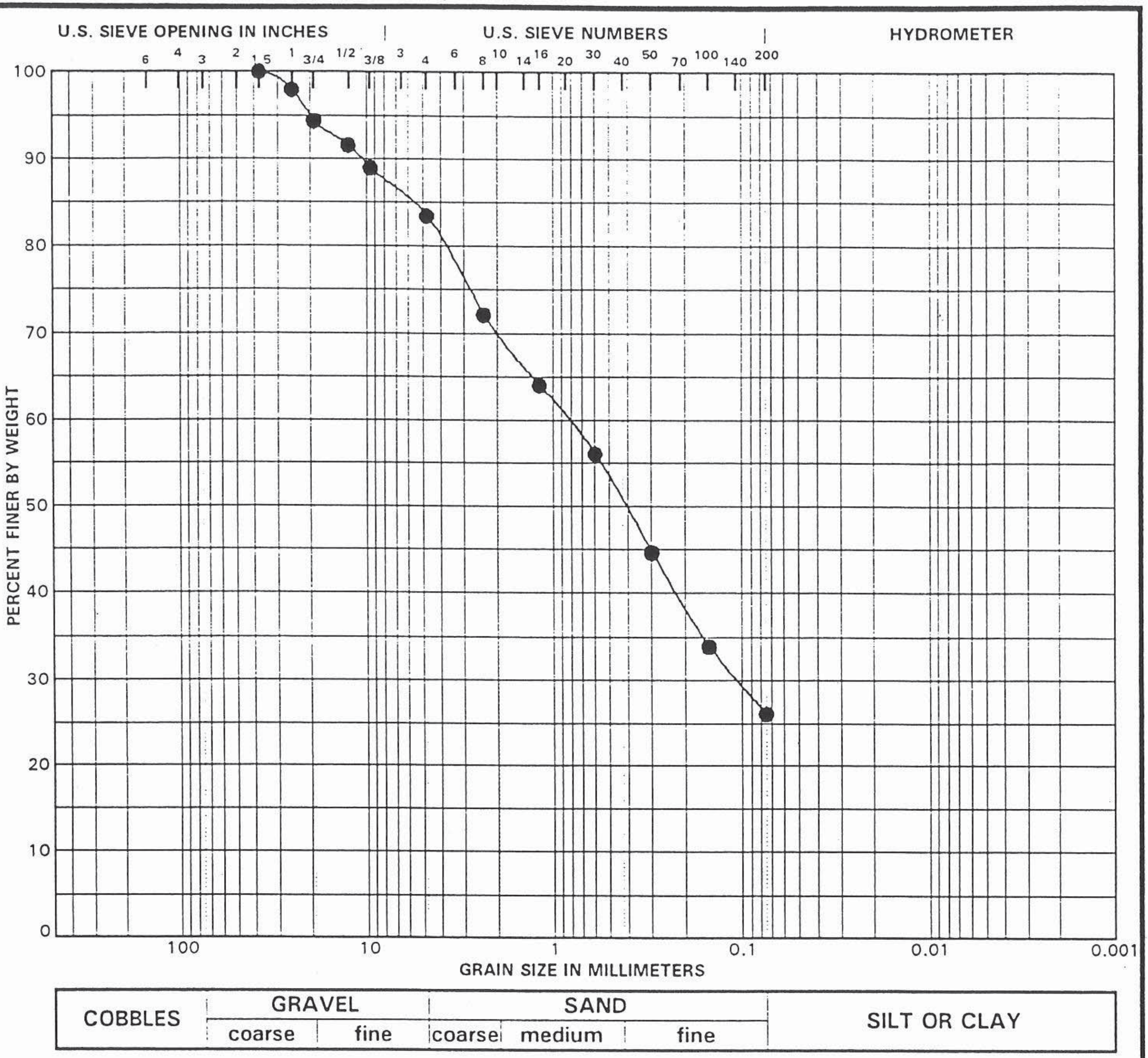
FIGURE C-11



BORING NO.: B-5
DEPTH (ft): 5-10

GRAVEL: 17 %
SAND: 57 %
FINES: 26 %

D_{10}= mm
D_{30}= 0.11 mm
D_{60}= 0.84 mm

Cu =
Cc =

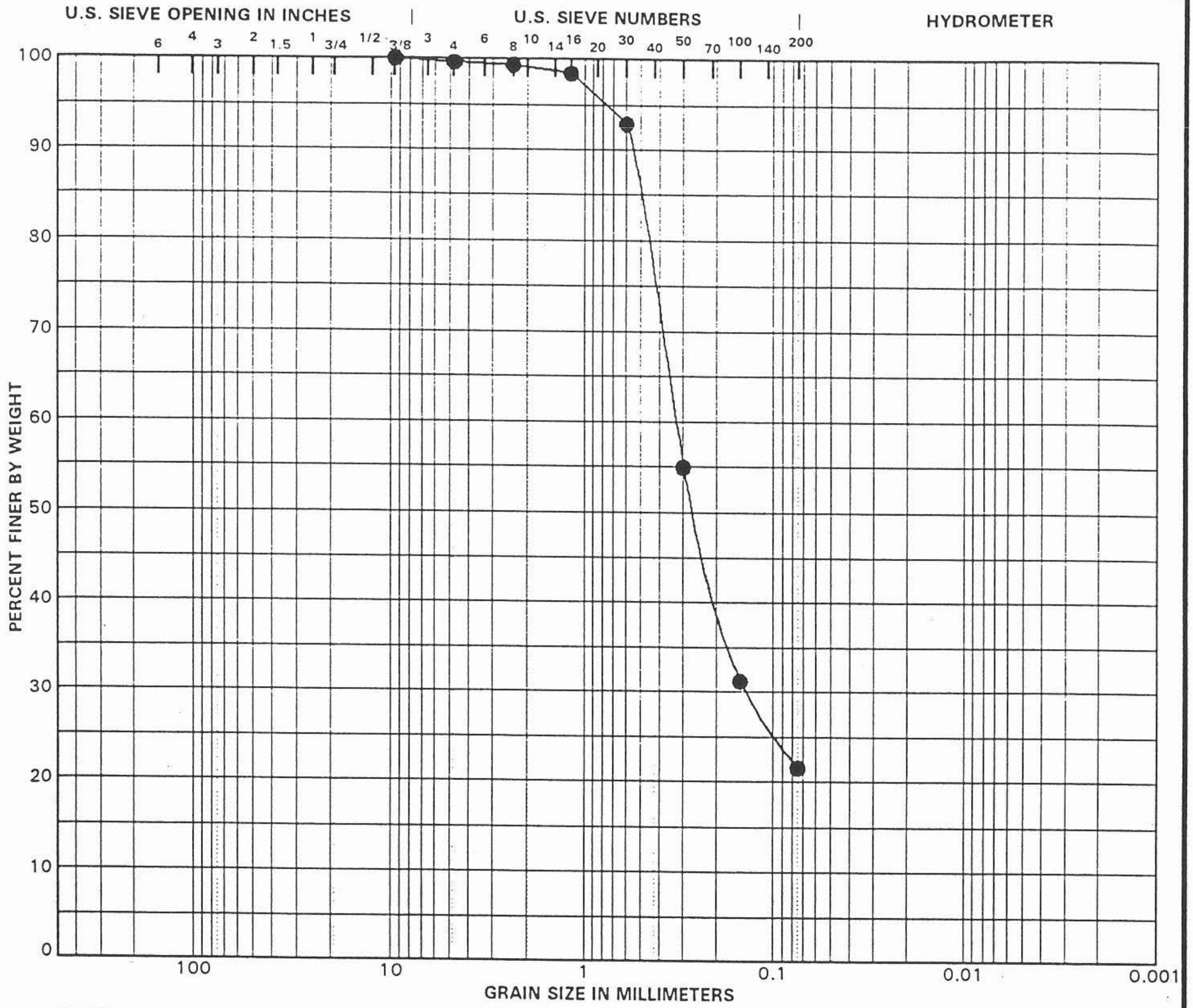
GEOBASE

PARTICLE SIZE DISTRIBUTION

Penasquitos Trunk Sewer Relief Project
San Diego, California

P.155.04

FIGURE C-14



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BORING NO.: B-6
DEPTH (ft): 2-3.5

GRAVEL: 0 %
SAND: 78 %
FINES: 22 %

D_{10}= mm
D_{30}= 0.14 mm
D_{60}= 0.33 mm

Cu =
Cc =

GEOBASE

PARTICLE SIZE DISTRIBUTION

Penasquitos Trunk Sewer Relief Project
San Diego, California

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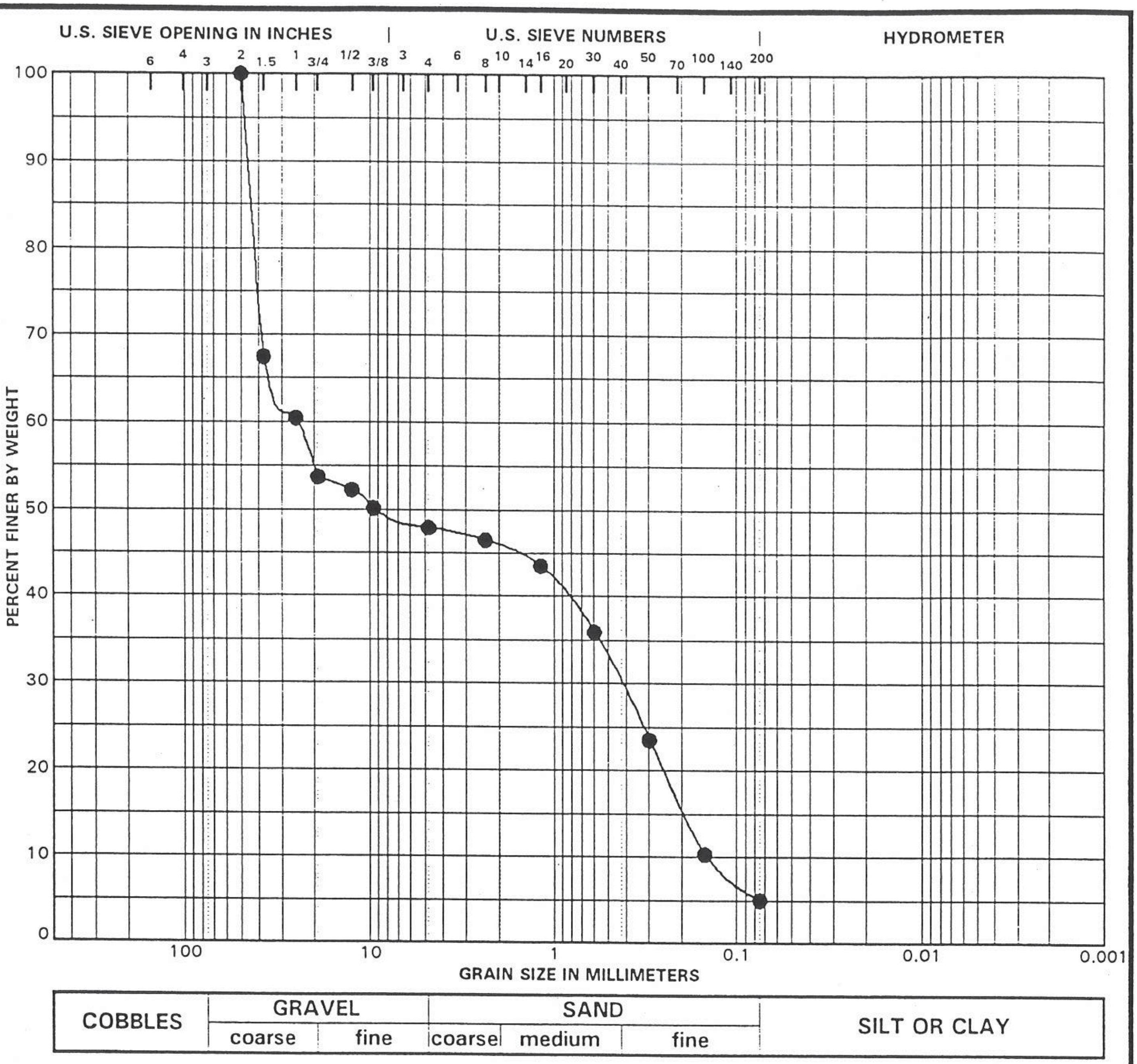
FIGURE C-15



BORING NO.: B-7
DEPTH (ft): 0-1.5

GRAVEL: 52 %
SAND: 43 %
FINES: 5 %

D_{10}= 0.14 mm
D_{30}= 0.43 mm
D_{60}= 24.59mm

Cu = 170.51
Cc = 0.05

GEOBASE

PARTICLE SIZE DISTRIBUTION

Penasquitos Trunk Sewer Relief Project
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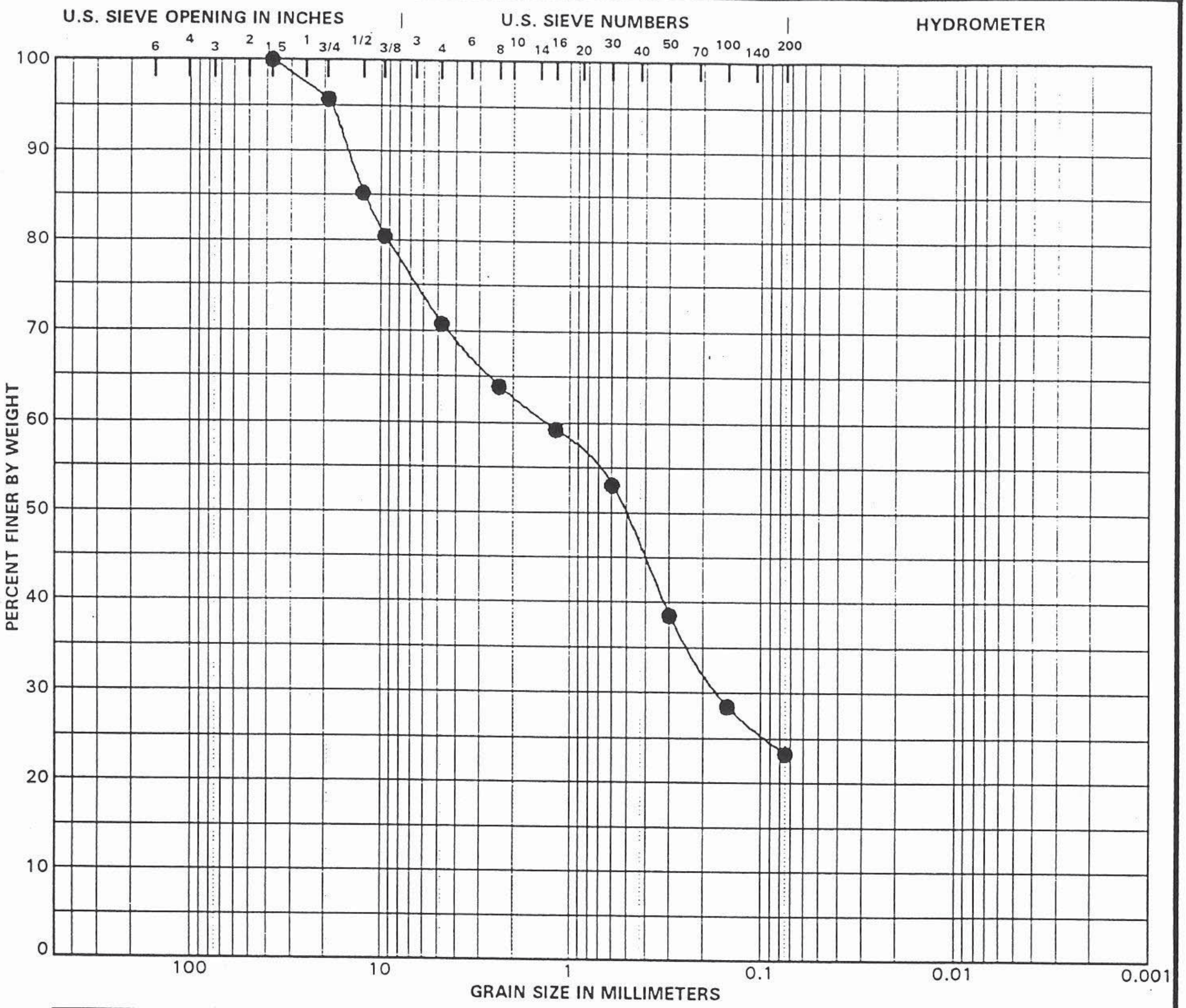
FIGURE C-16



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

BORING NO.: B-9
DEPTH (ft): 2-5

GRAVEL: 29 %
SAND: 48 %
FINES: 23 %

D_{10}= mm
D_{30}= 0.17 mm
D_{60}= 1.33 mm

Cu =
Cc =

GEOBASE

PARTICLE SIZE DISTRIBUTION

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San Diego, California

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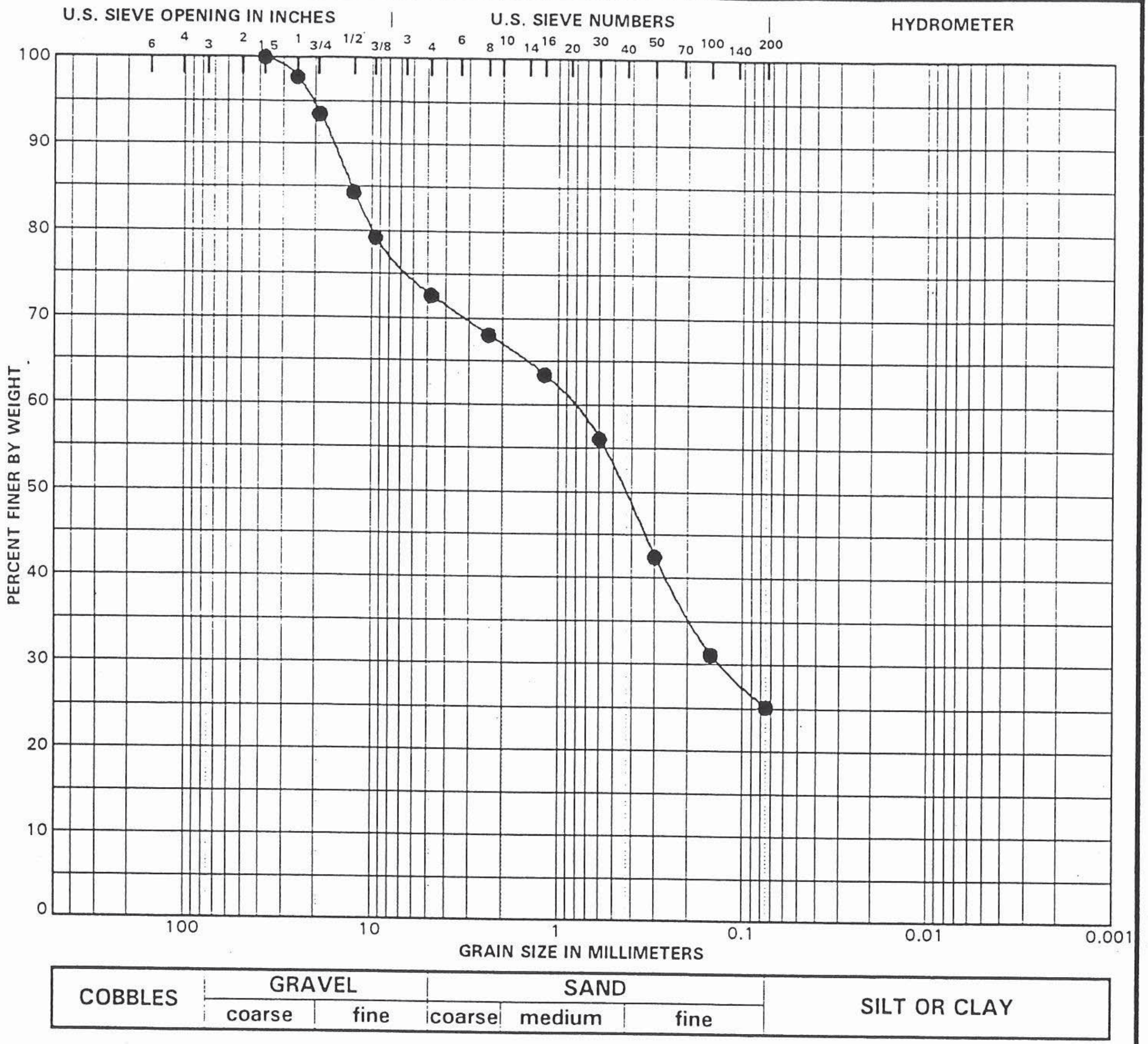
FIGURE C-17



BORING NO.: B-15	GRAVEL: 28 %	D_{10}= mm	Cu=
DEPTH (ft): 2-5	SAND: 47 %	D_{30}= 0.13 mm	Cc=
	FINES: 25 %	D_{60}= 0.86 mm	

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PARTICLE SIZE DISTRIBUTION

Penasquitos Trunk Sewer Relief Project
San Diego, California

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FIGURE C-18

APPENDIX D

WIRE REINFORCEMENT INSTITUTE METHOD FOR DESIGN OF SLABS ON EXPANSIVE SOILS

TF 700-R-03
Update

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DESIGN OF SLAB-ON-GROUND FOUNDATIONS

An Update

A Design, Construction & Inspection Aid For Consulting Engineers

March, 1996

Prepared for:
Copyright, Wire Reinforcement Institute
Wire Reinforcement Institute
942 Main Street
Hartford, CT 06103
Phone (800) 552-4WRI [4974]
Fax (860) 808-3009

Authored By:
Walter L. Snowden, P.E.
2613 Passion Flower Pass
Cedar Park, TX 78613

Phone: 512-331-6159
Fax: 512-331-6002
Email: wlspe@sbcglobal.net

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INTRODUCTION

In 1981 “DESIGN OF SLAB-ON-GROUND FOUNDATIONS, A Design, Construction & Inspection Aid for Consulting Engineers” was first published. The design procedure set forth in that publication had at that time been in use by the author for about 15 years. After this publication, it was subsequently adopted by the Uniform Building Code (UBC) as Standard 29-4(I). Copies of this work have been distributed by WRI for 22 years to consultants all across the nation. Feedback has been most favorable with no comments of design inadequacy. In a few cases there have been suggestions that this procedure produced extra conservative designs, but this guide is intended to always produce a safe, serviceable foundation. Engineers who care to are free to exercise their judgement and to adjust the results in either direction.

SOILS INVESTIGATIONS

It is still mandatory that soils investigation be made on any site to set out the necessary conditions for design. The original recommendation of a minimum of one boring for each isolated site is still valid, but many insuring agencies have specified at least two borings in areas where expansive clay is found. Large sites and subdivisions will need a specific planned program utilizing several borings. Subdivisions will usually average about one boring for every 3 or 4 contiguous lots. Borings should be a minimum of 15 feet deep in most cases, and in some instances will need to be deeper. The soils Engineer should be sure to obtain adequate information to cover any grading changes which can be anticipated. Fill should be identified and noted. Uncompacted fill placed on a site, and improper drainage have been found to be the largest contributors to unsatisfactory foundation performance. Either one or both are guarantees of foundation problems.

During the last 22 years, many alternatives to an adequate on-site investigation have been proposed; soils maps, adjacent data, guesses, and something called a “max design”. A “max design” is supposedly a design for the maximum soil condition in the area. How is that known unless an on-site investigation has been done? That is another name for a guess.

What remains true is that the performance of the slab is influenced primarily by the underlying soil. If the severity of the soil is underestimated, the foundation will not be satisfactory. It is therefore essential to know what type soil conditions exist, and that can only be known through an adequate site investigation.

LOADING CONDITIONS

For one, two, and even three story wood frame construction such as homes and small commercial buildings, the assumption of uniform load works well with the design equations. If there are large concentrated loads or numerous columns, attention must be paid to the location of stiffening beams or thickened areas of the slab so that the load can be spread out. Buildings which are carried totally on columns need a different analysis from a uniform loading assumption.

DESIGN ASSUMPTIONS

The design procedure presented originally by The Building Research Advisory Board (B.R.A.B.) in their Report 33, assumed a loss of support at the edges (Fig 1a) and a loss of support at the center (Fig 1b).

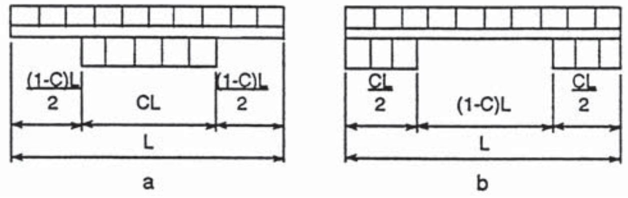
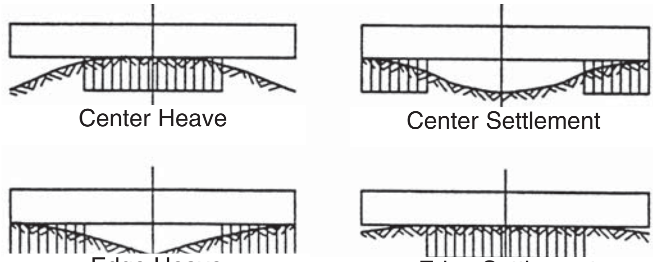


Figure 1



Center Heave

Center Settlement

Edge Heave

Edge Settlement

Figure 2

These conditions approximated the conditions of center heave or edge settlement and center settlement or edge heave as shown in Figure 2.

By making some simplifying assumptions it was possible to analyze the foundation slab by applying the loading conditions in both the long and short directions (Figure 3).

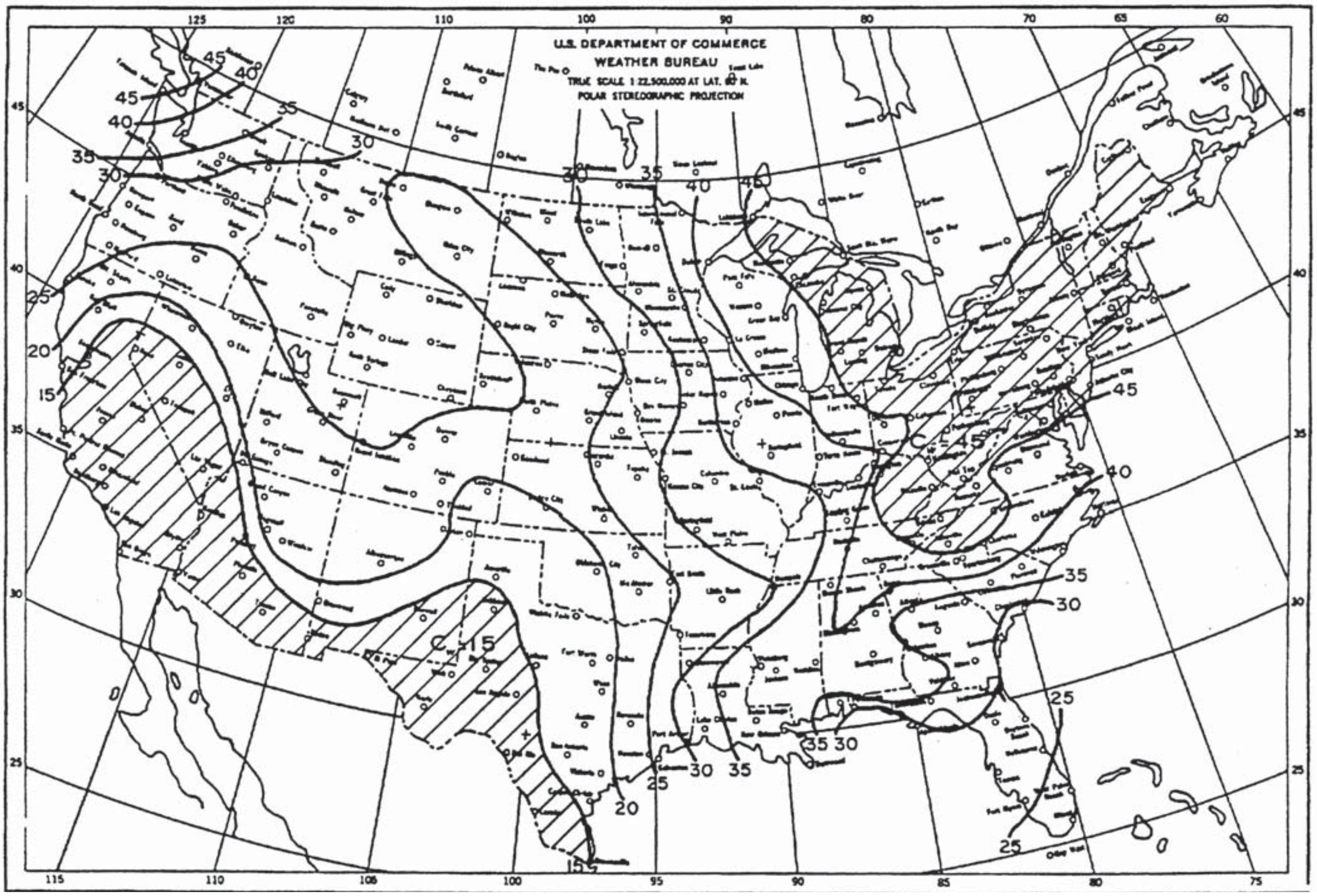
a. Cantilever

b. Simple Span Beam

Figure 3

GEOGRAPHIC CONSIDERATIONS

BRAB utilized the Climatic rating (see Figure 4) of the locality to reflect the stability of the moisture content in an expansive soil. While there are other methods of accounting for the seasonal moisture change potential, this system has seemed to work well.



Climatic Ratings (C_W) for Continental United States
Figure 4

DESIGN LENGTH

Looking at the various loading conditions above and slabs in the field, it became apparent that the foundations were very sensitive to the changes at the edges. It was decided that a cantilever distance, (I_C) would be used as a basis for this design procedure to replace the L(1-C) utilized by BRAB. Figure 5 gives a cantilever design length for a given soil condition (PI) in a given climatic rating (C_W).

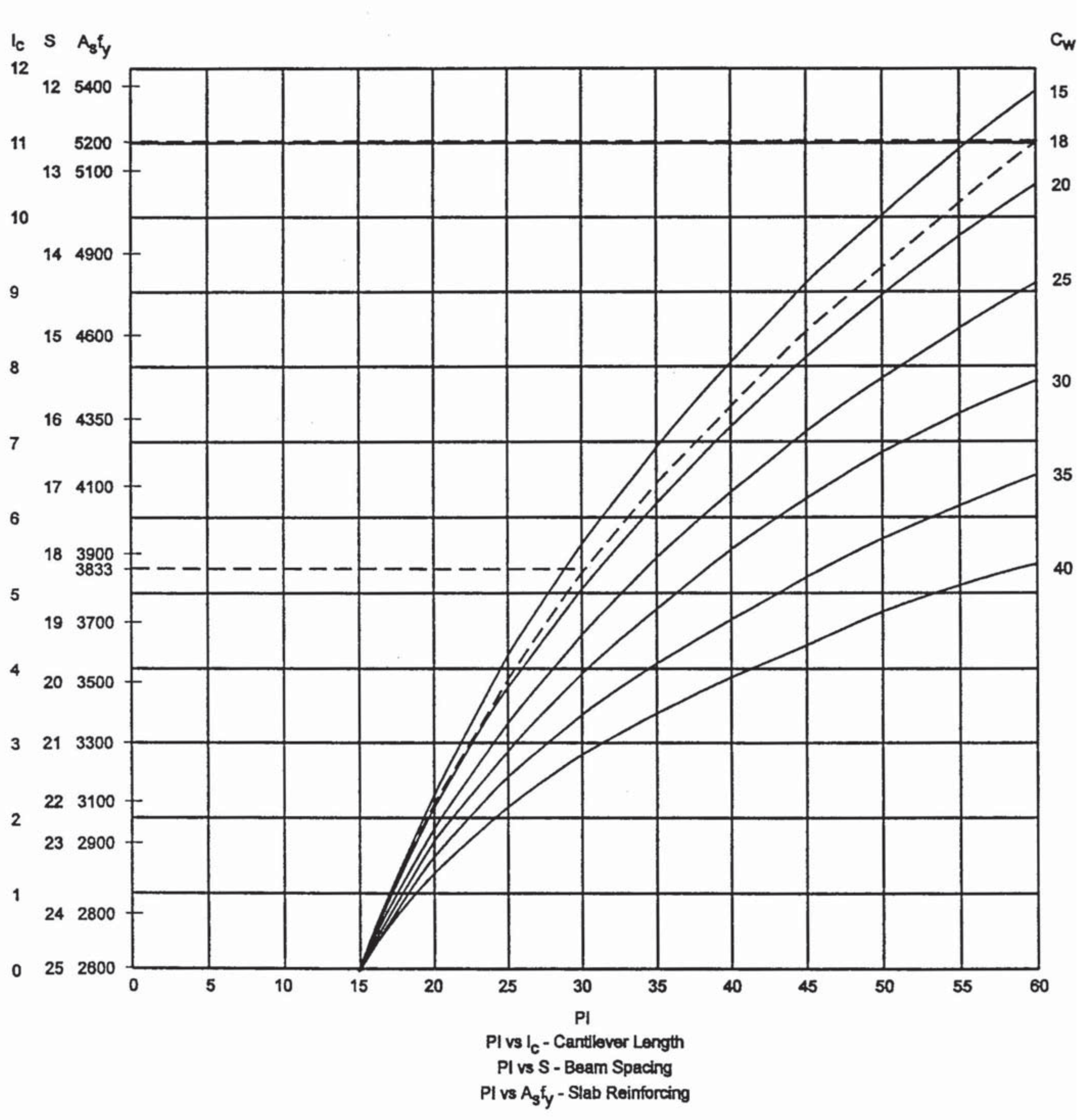


Figure 5

It seems apparent that the size of the foundation must also be considered. The values given in Figure 5 for the cantilever length are for large slabs. Figure 6 gives a modification coefficient which will adjust the cantilever length for smaller slabs depending on the slab size.

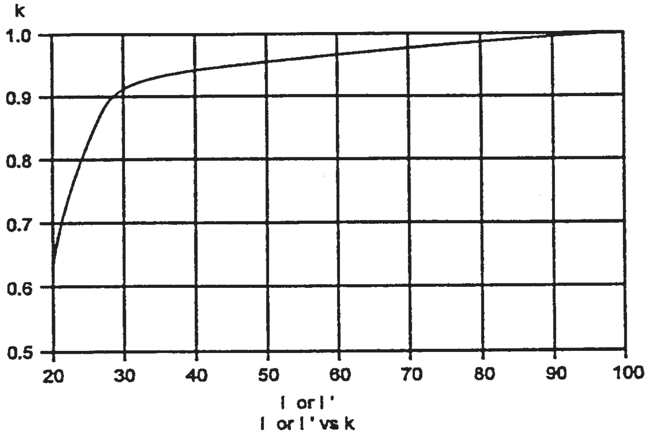
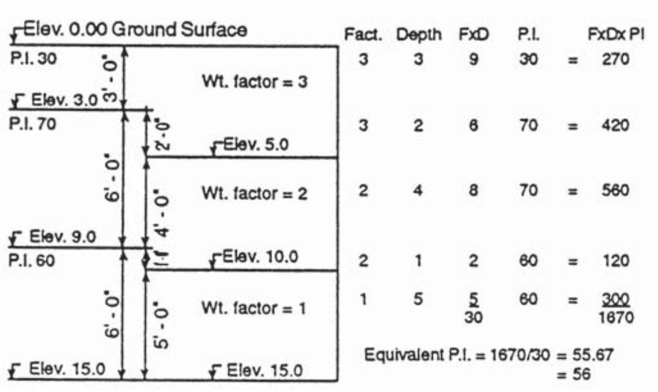


Figure 6

SOIL CONDITIONS

The design procedure shown in this report is based on the use of the "effective P.I." (PI_O). It has long been known that the Plasticity Index (PI) of the soil can be used as an indicator of the Potential Volumetric Activity of a given soil. It has the added advantage of being a test which is familiar and inexpensive to perform.

Obviously, different soils have different PIs, and the PI may change with depth at any one location. To account for this, the design procedure first calculates an "equivalent" or "weighted" PI. It is necessary to use the weighing system shown in Figure 7 to be compatible with this design procedure. This weighing method gives more attention to the upper soils where the soil would have the opportunity for more activity, and reduces the activity potential with depth due to confining pressure and protection from seasonal moisture changes, etc. This is not the only way to weight this effect, but it has proved to be very satisfactory, and must be used for this procedure.



Fact.	Depth	FxD	P.I.		FxDxPI
3	3	9	30	=	270
3	2	6	70	=	420
2	4	8	70	=	560
2	1	2	60	=	120
1	5	5	60	=	300
		30			1670

Equivalent P.I. = 1670/30 = 55.67
= 56

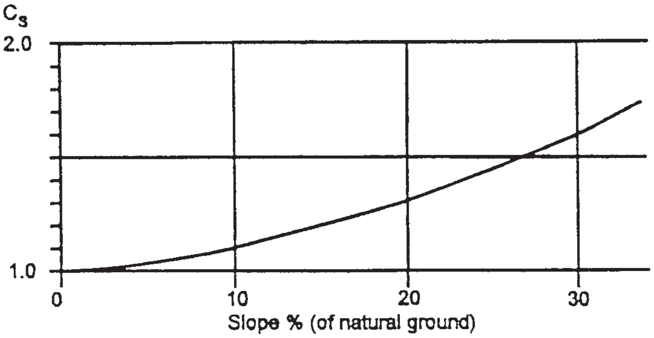
Figure 7

There are instances where this weighing system might give unconservative results. One would be where the underlying formations might contain sand stringers or are overlaid by porous sand which would provide quick, easy routes for water to reach any underlying or interbedded expansive clays.

A second case would be where highly expansive clays overlaid a rock formation. Using a zero (0) PI. for these rock layers can reduce the equivalent P.I. excessively, making it appear to be a very stable site. It is recommended that to eliminate this problem, a minimum P.I. of 15 be used for any layers which have little or no P.I.

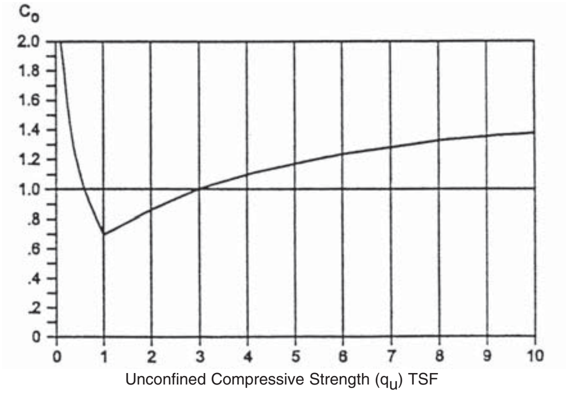
OTHER PARAMETERS OF CONCERN

Other factors to be considered are slope and degree of consolidation. Figures 8 and 9 present modification coefficients to be used with the "equivalent" PI to obtain the "effective" PI.



Slope of natural ground vs. Slope Correction Coefficient

Figure 8



Unconfined Compressive Strength vs. Consolidation Correction Coefficient

Figure 9

The effective PI then is:

$$PI_o = \text{equivalent PI} \times C_5 \times C_o$$

Where: C_5 is the slope correction coefficient
C_o is the consolidation correction coefficient

As an example: assume -

Equivalent (or weighted) PI = 30
10% ground slope C_5 (Fig. 8) = 1.1
6 TSF Unconfined C_o (Fig. 9) = 1.2
$PI_o = 30 \times 1.1 \times 1.2 = 39.6$

Use an Effective Plasticity Index of 40 for design purposes

HOUSE GEOMETRY AND LOADS

It is best to calculate the total weight of house and foundation, but in lieu of that, or as a starting point it is possible to use the following for most conventional wood frame houses with no unusual features (tile roofs, floors, high masonry loads, etc).

1 story - 200 lb/sq.ft.
2 story - 275 lbs/sq.ft.
3 story - 350 lbs/sq.ft.

Most houses can be subdivided into several rectangles and each section then be analyzed and then overlaid as shown in Figure 10.

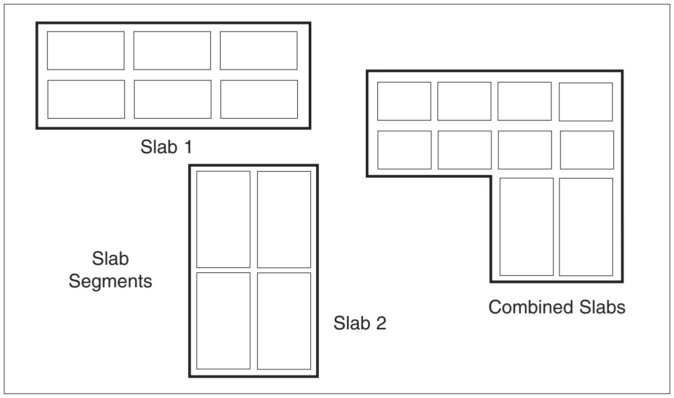


Figure 10

To begin the analysis the number of beams must be determined. Sometimes the geometry of the house will dictate the number of beams (N) required, sometimes the following equation will be used.

$$N = \frac{L'}{S} + 1$$

Where: S = Spacing ft (m) from Fig. 5
L' = width of slab, ft (m)

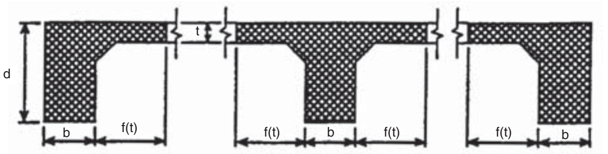
$$d = \sqrt[3]{\frac{664\, MI_C}{B}}$$

Where: d = Beam depth, in (mm)
B = Sum of all widths, in (mm)
M = Moment, kip-ft (N-m)
I_C = Cantilever length, ft (m)

Once N is known, a very good first approximation of the depth of the beams can be determined by the equation:

Using these equations yields a starting point with N number of beams, b inches wide and d inches deep which will give a Moment of Inertia (Iin^4) adequate to limit deflection to the order of magnitude of 1/480. This deflection ratio is greater than the usual 1/360, but it usually furnishes beam depths which allow the reinforcing requirement to be two or three bars of moderate size top and bottom. Of course, if the reinforcing requirement is still extremely large, try deepening all or some of the beams to lessen the reinforcing required.

In calculating the actual I of the slab, the sections shown in Figure 11 should be used. As can be seen, the exterior beams can be deepened, or all beams can be deepened. It is felt that deeper exterior beams are more effective, but as long as the slab is kept symmetrical it does not seem to matter.



Effective Width of "T" Beams

Figure 11

DESIGN CALCULATIONS

Now that the conditions have been defined, the following formulas can be used to calculate the moment, deflection and shear.

$$M = \frac{wL' (l_c)^2}{2}$$

$$\Delta = \frac{w (l_c)^4 L'}{4E_c I}$$

$$V = wL' \, l_c$$

Where: M = Moment + or -, kip-ft (N.m)
Δ = Deflection, in (mm)
V = Total shear, lbs (kg)
w = Unit weight, psf (kg/m^2)
L' = Width of slab, ft (m)
1_c = Cantilever, (l_c k) ft (m)
E_c = Creep Modulus of Elasticity of concrete, psi (MPa)
I = Moment of Inertia, in^4 (mm^4)

Naturally, these calculations will be performed in both the long and short directions.

TEMPERATURE AND SHRINKAGE REINFORCEMENT FOR CRACK CONTROL

The greatest number or reported complaints comes in the form of "cracked slabs". Of course all concrete will crack. Shrinkage crack prevention has spawned a plethora of papers, documents and books. The engineering community understands shrinkage cracking for the most part, but the general public sees each crack as a "structural failure". It is therefore very important to properly address the subject of minimum reinforcing to minimize shrinkage cracking and control crack widths.

The amount of reinforcing needed to control crack formation and width has been found to increase with the expansive potential of the site. Over the years greater need has developed to provide crack control to alleviate homeowners worries. When the beam spacings are near those shown in Figure 5, the minimum reinforcing shown also in Figure 5 is usually adequate. While this will not prevent shrinkage cracking, it will provide adequate reinforcing to hold cracks to a minimum width during deflection. In the field, actual deflection is a function of the expansive nature of the soil, and the stiffness of the slab, so the soil and the beam spacing together influence the deflection. Since the beam spacing is based on the soil (PI) and climate (C_w), the minimum slab reinforcement can also be based on the same factors.

HIGH STRENGTH WELDED WIRE REINFORCEMENT

The use of welded wire reinforcement in concrete has a long history. For this procedure it is strongly recommended that sheets of welded wire, plain or deformed be used. This will provide positive placement in the slab. Welded wire reinforcement sheets can be placed with the same degree of accuracy as tied reinforcing bars. Sheets with larger wires and wider spacing are more readily available, and are easily positioned. The use of high strength welded wire has been accepted by code and some real economies can now be realized, not only in material costs, but in placement costs.

Use of WWR actually provides the engineer a large number of choices as can be seen by the comparison below. Assuming a moderate soil condition and climatic conditions noted, the reinforcing in Chart 1 would be acceptable.

On higher PI soils, it would seem advisable to go to heavier slab reinforcing, even though the stiffness of the slab should be such that cracks would not tend to open any more than at lower PIs. To see how that would look for a higher PI soil, compare Chart 1 to Chart 2.

COMPARISON OF REINFORCING (1)

PI=60 $C_w = 18$ $A_8f_y = 3833$

Yield Stress f_y	A_8	Size (W -D)	Spacing**	Style
60000	.064	W6.4	12"O.C.	12x12-W6.4xW6.4
65000	.059	W5.9	12"O.C.	12x12-W5.9xW5.9
70000	.055	W5.5	12"O.C.	12x12-W5.5xW5.5
75000	.051	W5.1	12"O.C.	12x12-W5.1xW5.1
80000	.048	W4.8	12"O.C.	12x12-W4.8xW4.8

Chart 1

COMPARISON OF REINFORCING (2)

PI=60 $C_w = 18$ $A_8f_y = 5200$

Yield Stress f_y		Size* A_8	Spacing** (W-D)	Style
60000	.086	W8.6	12"O.C.	12x12-W8.6xW8.6
65000	.080	W8.0	12"O.C.	12x12-W8.0xW8.0
70000	.074	W7.4	12"O.C.	12x12-W7.4xW7.4
75000	.069	W6.9	12"O.C.	12x12-W69xW6.9
80000	.065	W6.5	12"O.C.	12x12-W6.5xW6.5

Chart 2

These values will approximate requirements of ACI 318, which allows for designs with yield strength up to 80,000 psi.

Use of the higher yield strengths will result in savings due to steel weight. Further savings can be realized by utilizing small edge wires closely spaced as shown in Figure 12. Savings will vary with specific areas, but some studies have shown that for each 5000 psi increase in $f_{y,}$ about 8% in steel weight is reduced. The use of small edge wires closely spaced can save an additional 3% or more. Perhaps the greatest saving will be in placing where costs have been reported to be reduced 50% and more over other conventional steel reinforcing.

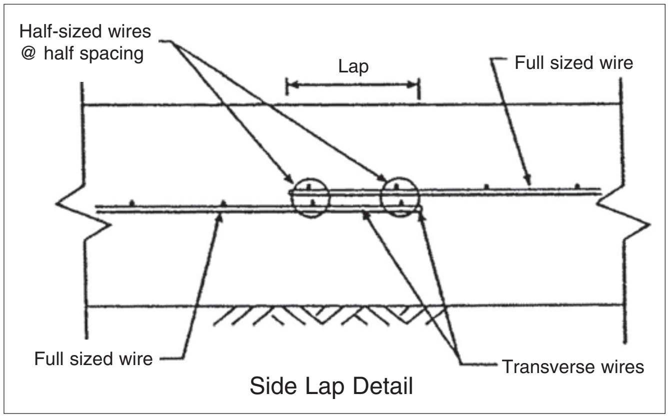


Figure 12

A DESIGN EXAMPLE

This design example utilizes welded wire reinforcement for slab-on-ground foundations over soils with high PI values:

Given: PI = 60
C_w=18

A_8f_y = 5200 lbs (f_y = 75,000psi)
Slab Thickness = 4"

Then: $A_8 = \frac{0.0018 \times 60{,}000}{75{,}000} \times (4 \times 12) = 0.069$ in.²/ft of concrete cross section

Check strength level required: $A_8f_y = 75.000 \times 0.069 = 5175 = 5200$ OK

CONCLUSIONS

This design procedure, which has been in use about 37 years at this time, has produced satisfactory foundations for single family housing and small commercial applications. This update is meant to make it easier for the consultant to use by combining several tables into one (Fig 5). The Effective PI, and the Climatic Rating are all that need be known to obtain a cantilever length for design.

This paper is a condensation of more detailed work. Engineers may obtain copies of the original work by contacting the WRI. Copyright, Wire Reinforcement Institute Wire Reinforcement Institute 942 Main Street, Suite 300, Hartford, CT 06103
Phone: 800 552-4WRI(4974) • Fax: 860 808-3009
The Author Walter L. Snowden, P.E. Cedar Park, Texas

Phone: 512-331-6159

Fax: 512-331-6002

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* W = plain wire, also can be prefix D for deformed wire.

** Wire spacings are available in 2" to 18" in either or both longitudinal and traverse directions. Contact individual welded wire producers for specific styles and spacings of WWR

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DESIGN OF SLAB-ON-GROUND FOUNDATIONS

A Design, Construction & Inspection Aid For Consulting Engineers

August 1981

Prepared for:
Copyright, Wire Reinforcement Institute
Wire Reinforcement Institute
942 Main Street
Hartford, CT 06103
Phone (800) 552-4WRI [4974]
Fax (860) 808-3009

Authored By:
Walter L. Snowden, P.E.
2613 Passion Flower Pass
Cedar Park, TX 78613

Phone: 512-331-6159
Fax: 512-331-6002
Email: wlspe@sbcglobal.net

This procedure was developed by Walter L. Snowden, P. E., Consulting Engineer, of Austin, Texas, over a period of some 15 years. It is empirically derived by observing slab performance and writing or modifying equations to give results which approximate the foundations which had been found to give satisfactory results.

In addition, Mr. Snowden, has served on the Pre-Stress Concrete Institute Ad Hoc Committee for the development of " Tentative Recommendations for Pre-Stressed Slabs-on-Ground" and as a Consultant to the Building Research Advisory Board Committee on Residential Slabs-On-Ground.

Designs done by this method should be economical yet give quite satisfactory results with a minimum of deflection and resulting superstructure distress.

While this publication deals only with foundations reinforced with reinforcing bars and/or welded wire reinforcement, the procedure has been developed to be independent of the type of reinforcing used.

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INTRODUCTION

Within the last several years there has been a lot of interest in a design procedure for the design of light foundations, particularly for use under single family residences. Reports and recommendations have been undertaken and prepared by several study groups for the purposes of developing design criteria or extending the Criteria for Selection and Design of Residential Slabs-on-Ground, BRAB Report #33. The recommendations derived from these and other studies vary from extremely light to extremely heavy.

It was actually the widespread use of the "post-tensioned" slab-on-ground which induced this interest in design procedure and in many studies of reported slab failures. Such reports have; perhaps, created an over-cautious climate concerning any moves to lighten the design requirements set forth in BRAB Report #33. Many theoretical analyses show that no lessening of the requirements is possible, while other studies and actual field installation indicate that considerable variances are permissible in many areas.

In the design procedure to be presented herein, adjustments are made to the BRAB procedure which allow the use of this simple procedure with larger slabs and further simplify the design engineer's problem of designing an adequate foundation at a reasonable cost, both in terms of the engineer's time, and cost of the installation itself.

The intent of this handbook is to provide a design procedure which could be used in any Consulting engineer's office to give adequate designs for economical construction without the use of large computers, or the necessity for site investigations so extensive as to make the use of engineered foundations economically prohibitive. The following procedure, with modifications, has been used for the last 15 years in designing foundations in the southwest with excellent results.

EARLY DEVELOPMENTS

In the early 1950's the use of the monolithic reinforced slab foundation become widespread in the south central portion of the United States. For the most part there were no consistent standards, and many different versions of this foundation were to be found throughout the area. Each office of the Federal Housing Administration had a different version being used in its area, and the differences in cross-section and reinforcing were great. Engineers did not have a generally accepted procedure to analyze the slab, and, therefore, the problem was mostly ignored.

In 1955 the Federal Housing Administration together with the National Academy of Science organized a group of nationally eminent authorities and began a several year research project to develop guidelines for design of slab-on-ground foundations.

The final report, Building Research Advisory Board (BRAB) Report #33 en-titled Criteria for Selection and Design of Residential slab-on-ground, was issued in 1968 and was widely discussed by builders. First designs to follow the BRAB Report required foundations heavier even than the San Antonio FHA office standard LAS-22 (Fig. 1). LAS-22 was thought to be the heaviest design ever needed, but a local study showed it was inadequate perhaps 30% of the time. There was naturally, great resistance to the added costs of design and construction required by the BRAB Report.

The next important contribution also occurred in 1968 when a full scale post-tensioned slab was built and tested to destruction. A subsequent report established the feasibility of using post-tensioning in slab-on-ground construction and verified many of the BRAB assumptions.

In 1965 the writer developed a complete, overall design system, later modified to conform, in format, to BRAB Report #33 and further influenced by the work done by H. Platt Thompson, P.E. This system gained wide use in both Austin and San Antonio because of the lower cost which the post-tensioned slab enjoyed compared to the heavier F.H.A. San Antonio "Standard Slab".

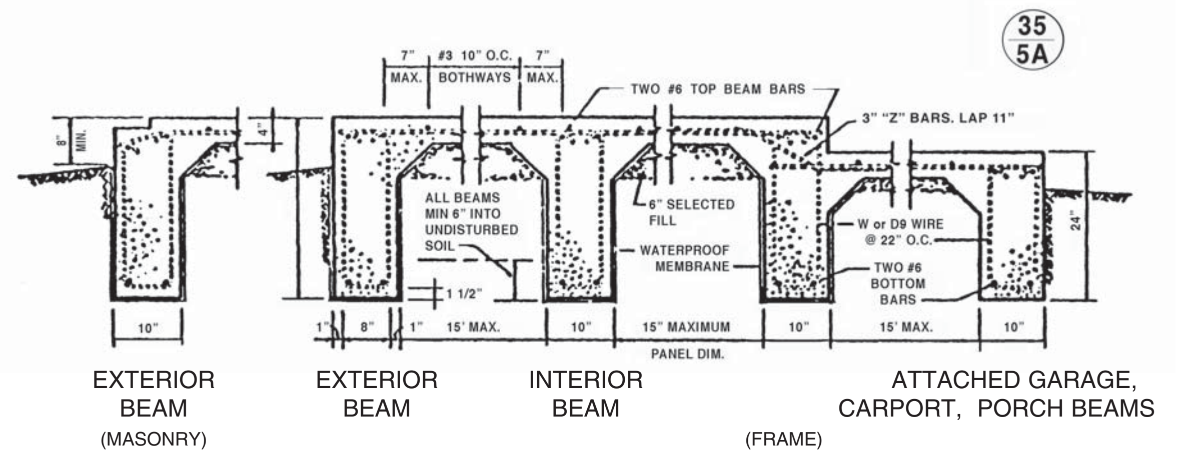
Variations from the BRAB Report #33 were developed to maintain a reasonable ratio between cost of the slab-on-ground and the value of the house it supported. The variations presented later in this paper have been derived empirically.

SOIL INVESTIGATIONS

It is considered imperative that a soils investigation be made on any site on which a design is to prepared.

RESIDENTIAL SLAB-ON-GROUND CONSTRUCTION

FEDERAL HOUSING ADMINISTRATION SAN ANTONIO~ TEXAS INSURING OFFICE



ALL RESIDENTIAL SLAB-ON-GROUND CONSTRUCTION SHALL COMPLY WITH THESE MINIMUMS. VARIATIONS ARE ACCEPTABLE WHERE SOIL INVESTIGATION OF THE BUILDING SITE, CLIMATIC RATINGS, AND ENGINEERING ANALYSIS INDICATE A SLAB OF LIGHTER OR HEAVIER DESIGN IS SUITABLE.

CONCRETE: 2500 PSI MIN. COMPRESSIVE STRENGTH. LAPS OR SPLICES: MINIMUM 30 DIAMETERS.

SLAB: 4" MINIMUM THICKNESS WITH W OR D9 WIRE 10" O.C. BOTH WAYS. MAXIMUM CLEAR PANEL BETWEEN BEAMS IS 15 FEET.BEAMS: 10" WIDE BY 30" DEEP. (24" DEEP FOR ATTACHED GARAGE. CARPORT, OR PORCH BEAMS) REINFORCE WITH TWO #6 BARS TOP AND TWO #6 BARS BOTTOM, CONTINUOUS. SPACE ALL STIRRUPS 22" O.C. ALL BEAMS SHALL PENETRATE MINIMUM 6" INTO UNDISTURBED SOIL.

CORNER BARS: PROVIDE #6 CORNER BARS IN ALL CORNERS OF THE PERIMETER OR EXTERIOR BEAMS. INSTALL ONE AT TOP, OUTSIDE. AND ONE AT BOTTOM, OUTSIDE.

FIGURE 1

For a small site with one structure, the minimum is obviously one test boring, which should be made where the worst soil condition is anticipated; ie, where fill is located, or where the worst clay is suspected. If it is not obvious, then more than one test hole is indicated. In no case should a design be attempted without an adequate soils investigation of the site.

For large sites with large structures or more than one structure, several test holes must be used. In planning the investigation, plan for the worst. It is always possible to omit borings in the field, based on data as it develops.

For a subdivision, there can be no fixed minimum number of borings. The work done should be that which is required to get the answer. In general, locating holes about one to every four or Five lots, if the subdivision is reasonably uniform, will be adequate. Should different materials be encountered, additional borings must be placed to provide more complete information of the underlying soils. In some cases it is necessary to drill each lot. When a contact between a high P.I. soil and limestone is discovered, for instance, each lot which the contact crosses must be designed as though the entire lot were the worst soil condition.

As drilling progresses, samples should be taken at 2’ intervals and at each different soil strata encountered, to a depth of at least 15’ If it is likely that some soil will be cut from the lot, borings should be deepened appropriately. Perhaps all borings should be 20 feet deep to allow for any cut, but at present, 15’ borings are considered sufficient. Undisturbed samples should be taken, where possible, to allow evaluation of unconfined fined strengths of the various strata. As unconfined strength of 1 ton is usually sufficient for single story frame houses such as those under consideration. For commercial and multi-story, 2 tons is usually adequate to insure against bearing capacity failure.

During field investigation it is important to make notes of existing fill, trees, thickets, old fence lines, roads, slope of each lot, topography, seeps, sinks, rock outcrops, and any area which may require fill to bring it up to grade before construction. Grading and drainage plans, when available, may be helpful in identifying some of these significant features. Note these fill lots or even suspected fill lots in the report so that proper care may be exercised by the insuring agency, city officials, design engineers, et. al. Uncompacted fill under the beams of an engineered slab will almost certainly create problems. Specify that all fill be acceptable material, properly compacted. H.U.D. projects and subdivisions are supposed to require that fill be placed in accordance with “Data Sheet 79-G”. [15]

LABORATORY TESTING

After the proper field investigations have been made, it is necessary to run laboratory tests on samples from the various strata taken in the field. It is important that all strata be correctly identified and tested. Identification should be in accordance with the unified soil classifications chart shown in Fig. 2. Such terms as “caliche,” “fat clays,” “loam” and other colloquialisms should be avoided or used only as extra comment. Plotting liquid limits and plasticity indices on the classification chart will confirm field evaluations. If proper testing and Identification are done, some degree of uniformity can be applied to Slab-on-Ground designs.

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve*	**GRAVELS** 50% or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse fraction passes No.4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravel-sand mixtures, little or no fines
		SANDS WITH FINES	SM	Silty sands, and-silt mixtures
			SC	Clayey sands, sand-silt mixtures
FINE GRAINED SOILS 50% more passes No. 200 sieve*	**SILTS AND CLAYS** Liquid limit 50% or Less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			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 AND CLAYS Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous, fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

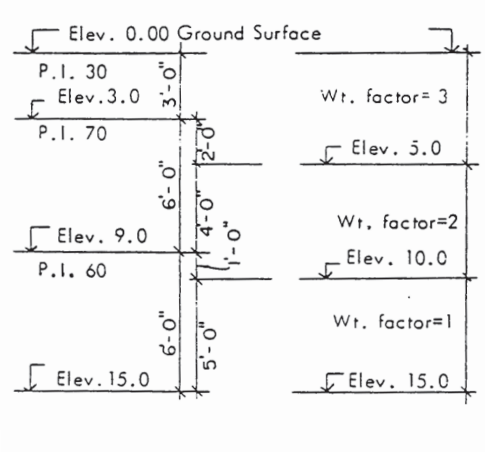
* Based on the material passing the 3 -in. (75-mm) sieve.

Figure 2

DETERMINING THE "EFFECTIVE P.I."

The BRAB report bases its design procedure on the soil plasticity index (P.I.). This design procedure also uses the P.I. because it is a relatively simple-ple test which is routinely performed in all testing laboratories.

Since the soil is not always constant with depth, it is necessary to find the "effective P.I." of the underlying 15 Feet. BRAB Report #33 suggests a weighing system (Fig.3).



Fact.	Depth	FxD	P.I.		FxDxPI
3	3	9	30	=	270
3	2	6	70	=	420
2	4	8	70	=	560
2	1	2	60	=	120
1	5	5	60	=	300
		30			1670

Effective P.I. = 1670/30 = 55.67
= 56

Figure 3

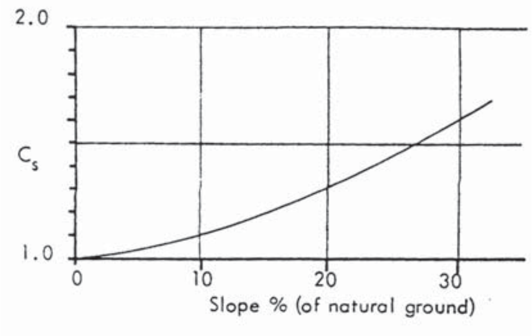
This seems as valid as any weighting method, as McDowell's[16,17] procedure for calculating potential vertical rise also indicates that the upper few feet is the most active. The activity then decreases with depth due to confining pressure and protection from seasonal moisture change, etc. Any system that gives more attention to the surface soils is probably satisfactory. One place where this system might give erroneous results would be in formations which contain sand stringers or are overlaid by porous sand which would provide quick, easy routes for water to reach underlying or interbedded CH clays.

Another case would be high P.I. clays overlaying rock. Using a zero (0) P.I. for these rock layers can reduce the "effective P.I." excessively making it appear to be a very innocuous site, It is probably best never to use zero for a P.I. Since BRAB recognizes 15 as a breaking point for Type III slabs, some minimum value such as 15 should always be used for those layers with little or no P.I. BRAB recognized the problem by utilizing the P.I. immediately below the slab if it was higher than the P.I. of the lower layers. This very conservative approach will always yeld good, safe designs, considerably overdesigned.

OTHER PARAMETERS

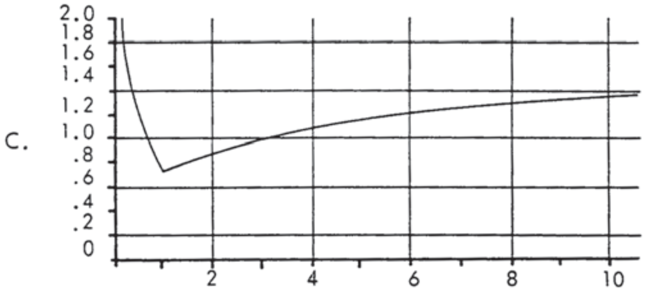
Once the "effective P.I.'s" for each boring are calculated, they need to be modified by some other parameters. The slope of the lot should be used to increase the "effective P.I." Figure 4 can be used to determine coefficients based on slope.

The degree of over-consolidation of the natural material can be estimated from the unconfined compressive strengths. By using Fig. 5 a coefficient for over-consolidation can be determined.



Slope of natural ground vs. Slope Correction Coefficient

Figure 4



Unconfined Compressive Strength (q_u) TSF
Figure 5

Other factors are known to require consideration; moisture condition at time of construction, geologic formation, percentage of soil passing #40 sieve, percentage passing #200
sieve, all of these affect the potential volume change of the underlying soil. The correct value of "effective P.I." is that from the equation:

Eff. P.I. ( ) = Effective PI x C_s x C_o x C_y x C_z ••••• C_n

Much work needs to done in this area.

The ultimate performance of a slab reflects how well the soil analysis was done. Slab design is only as good as the soil data on which it is based. Some engineers say they do not need soil data to do a design. They are either deceiving themselves or are over-designing their slabs in which case they delude their clients and ultimately, the purchaser of the structure. There are few circumstances where the engineer is justified in over-designing and wasting the client's money. There are no circumstances where the engineer is justified in under-designing--even at the client's request.

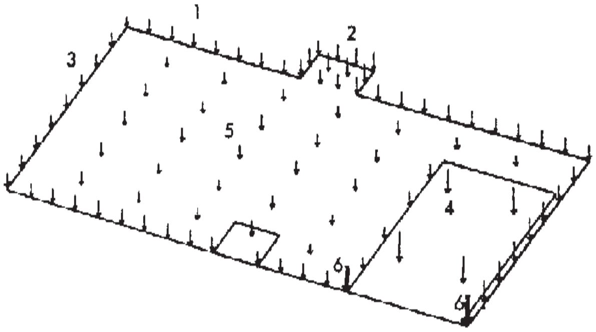
WARNING

It should be recognized that there are certain conditions which neither this procedure nor any other will be able to anticipate. Examples of such problems which might cause difficulty, even to a well designed slab, would be the location of an old fence row beneath the foundations, a broken water pipe, improper drainage away from the foundation, a slab located on top of of previously existing tree or thicket, massive erosion or loss of support due to lack of compliance with proper site preparation standards, poor maintenance, or improper installation. There are numerous documented cases where slabs have exhibited less than the desired results due to one or more of these causes. Most of the causes mentioned above can be mitigated by proper construction and inspection. The others, such as old fence lines, trees, or thickets are generally unknown to the Soils Engineer and the Design Engineer, and, in many cases, cannot be anticipated at all. It is felt that the present state of the art make these conditions fall beyond those for which the designer can properly be considered responsible. The problem with this line of reasoning is that by the time it becomes apparent there is a problem with the slab, it is not possible in most cases to determine that the problem is one of those which could not be anticipated. The owner is having difficulty, and he is seeking relief, and quite often, revenge and restitution. These cases usually end up being decided by a jury. This is one very good reason for not trying to reduce the design standards too far and for trying to get a good standard adopted so it will be clearly defined when the engineer has done all that he can be reasonably expected to do.

LOADING CONSIDERATIONS

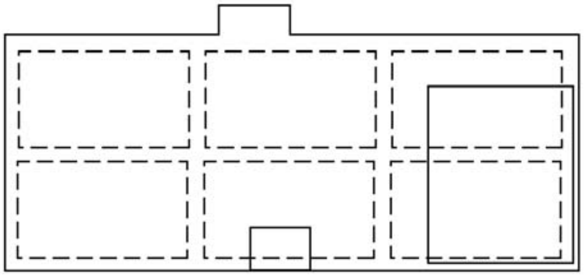
First look at a small slab for a single story house, trussed roof construction, masonry veneer, fire place and one car garage. What do the loads look like? (see Fig. 6)

1. Roof LL & DL, stud wall, brick veneer and ceiling loads
2. Brick chimney load
3. Stud wall and brick veneer
4. Wheel loads
5. Floor live loads (including non bearing partition allowances)
6. Concentrated loads from beam spanning garage doors



Superstructure Loads
Figure 6

Loads in Fig.6 are only the loads applied to the top of the slab. To these must be added the weight of the slab, edge beams, and interior stiffening beams. (see Fig.7).



Slab Configuration
Figure 7

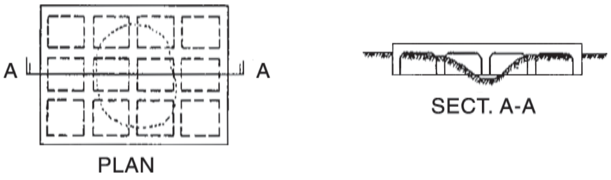
To the soil underneath, these loads are not nearly so clearly defined. For the small slabs generally used under houses and small commercial buildings the loads can be assumed to be uniform. When the unconfined strength of the soil is less than 1 ton/sq. ft., settlement or bearing can be a problem and should be considered, but on stiff expansive clays any distress in the slab and superstructure will be caused by the volumetric movement of the soil due to moisture change. If the soil did not change, the weight of the house or small building would be transmitted directly through the slab and into the under-lying soil which with the exception mentioned above, can easily carry the weight since it is usually less than 500 lbs. per sq.ft.

Since the loads are small, it seems justified to use the simplifying assumption of a uniform load. This has given good results on single story residences.

SUPPORT CONDITIONS

Prior to the time the BRAB report was issued, the writer had been working on the problem some years and had developed a working design procedure.

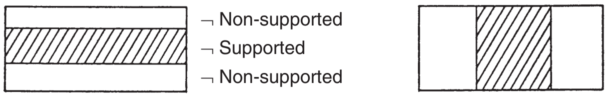
The procedure involved an area of loss of support, (Fig. 8) the diameter of which was a function of the soil (P.I., degree of compaction, etc.) and which was allowed to move to any position under the building. The most critical locations, of course, were under load bearing walls and columns. The equation had been adjusted to give both positive and negative movements.



Area of Support Loss
Figure 8

This procedure was developed entirely from looking at slabs that seemed to work and those which did not and writing an equation which would produce sections equal to those which had been performing satisfactorily. Formulas had been developed which took into account loss in the center as shown above, loss at edges and corners. Also, there were provisions for inclusion of concentrated loads.

This procedure designed only one or two beams at a time. The BRAB report showed support conditions (see Fig.9) which allowed all beams in a given direction to be considered at one time. This simplified the design procedure, and, when the two design procedures were compared, they were found to give similar results. The BRAB procedure produced heavier-designs, but, with minor modifications, they could be adjusted



a. Cantilever

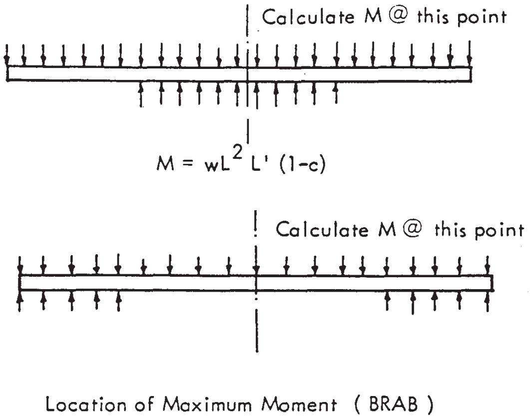
b. Simple Span Beam

Assumed Support Conditions

Figure 9

The moment equations developed by BRAB give a maximum moment, both positive and negative at midspan (Fig. 10). This is not a simple cantilever moment. For short slabs it is a reasonable analysis. For longer slabs it quickly becomes excessive.

To eliminate the problem, several alternatives have been discussed:



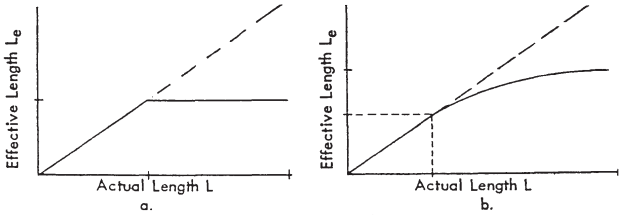
Location of Maximum Moment

Figure 10

a. Design all slabs, longer than a certain length, for a maximum moment based on that length and all slabs less than that, for their exact length (Fig. 11a)

b. Use an effective length in the original BRAB equations. (Fig. 11b)

c. Design all slabs for both positive and negative bending based on some cantilever length. (Fig. 12)



Length Modification Assumptions

Figure 11

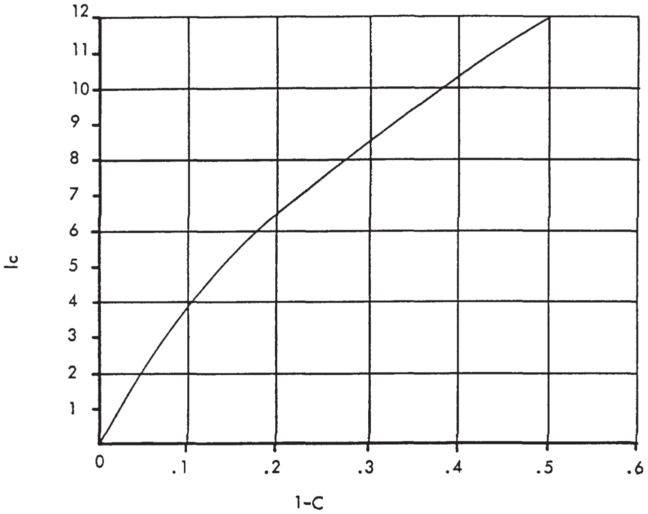
Note that with Figure 11a there is no increase in design moment beyond the assumed maximum length. Obviously, as the slabs get longer, more reinforcing needs to be added to compensate for friction losses, drag, etc. P.C.I. goes into great detail to calculate these losses. By using an effective value for “L” as shown in Figure 11b, these losses are automatically covered. While this was a completely empirical approach, it was easy to use and gave good results.

It has been noted during previous research concerning slab-on-ground construction that the large slabs tend to reach an equilibrium in the center portion and fluctuate only with seasonal moisture change.

Some routine testing during the time of the soils investigations can reasonably define the depth of the zone of seasonal moisture change which, many say, is roughly equal to the horizontal distance moisture may pene-trate under a slab and cause differential movement or pressure. While this does indeed give a cantilever action such as was previously described, the point of maximum moment is not located at a distance from the edge equal to the depth of the seasonal moisture change and nato distance of $\frac{L}{2}$(1-C). Much work has been done trying to define this cantilever distance. This design procedure has developed an empirical curve which, when used with the equations set out later, gives good results. Again, it makes no difference whether the cantilever theory is used or the BRAB equations are used, so long as the proper input is supplied for either criteria.

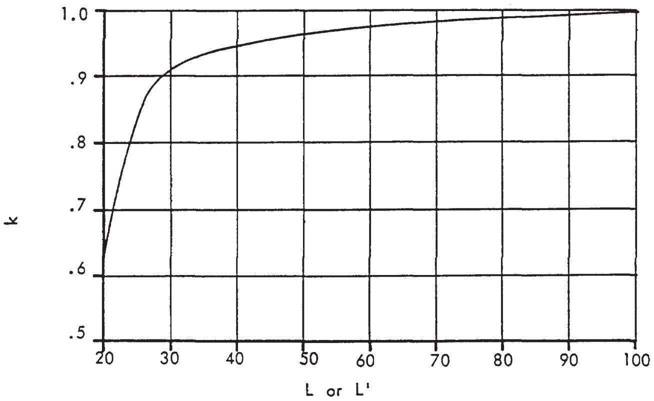
The BRAB equations utilizing an effective length, as opposed to the total length, were used for years and gave good results. Since the P.C.I. and P.T.I. have advocated a cantilever approach, this procedure has been modified to use a cantilever (see Fig. 12) which gives the same results as the modified BRAB equations. Note that in cases both positive and negative reinforcing are supplied.

There is, at this time, a great deal of discussion concerning the relative equality of the positive and negative moments used in design. It seems that a large number of engineers feel that the positive moment is not as significant a design parameter as is the negative moment. Numerous proposals have been offered for the reduction of the positive moment. A look at the loading conditions on most slabs will offer support to this reduction theory, and some experimental work has been undertaken by this firm to evaluate this proposal. The results observed indicate that some reductions are justified and allowable. To date, no findings have been brought forth, backed by any performance data, to indicate what magnitude of reduction should be considered.



(1-C) vs. Cantilever Length (lc)

Figure 12



L or L' vs k

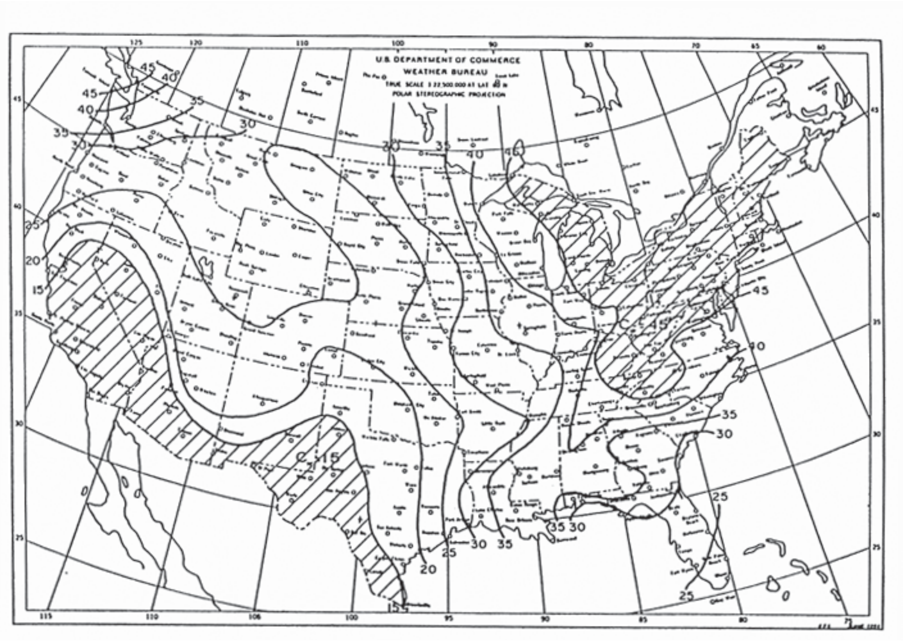
Figure 13

THE SLAB DESIGN

The proper procedures for soils investigations and reporting have been mentioned in this report, and, assuming that the proper information is available, an actual foundation design can be begun. The design procedure begins by determining a unit weight of the building including its foundation. Assume that such weight is distributed uniformly over the entire foundation area. Those conditions where concentrated loads are felt to be of such magnitude that they must be considered, are not covered in this paper.

As previously stated, the weight of the structure is not so significant as the support conditions of the underlying soil material, however, the weight calculated in these procedures is generally indicative of the amount of differential movement which can be tolerated by the superstructure. The heavier the unit weight, the more brittle and sensitive to movement is the superstructure material in general. Also, heavier loads generated by multi-story buildings indicates that additional stiffness must be supplied to the foundation because of the sensitivity of multi-story buildings to differential movement. A very light wood frame structure with wood siding and no masonry would be far less susceptible to structural and cosmetic damage than would be a heavy all-masonry or brick veneer type building. Use of these increased unit weights automatically generates additional moment and deflection criteria to satisfy the need for additional stiffness and strength.

These criteria, incidentally, apply to residential and small commercial construction and not to the more monumental type structures such as banks, churches, and highrise building. These same design procedures could be used for these types of buildings, reducing the allowable deflections and stresses, and including allowances for high concentrated loads to produce the more rigid foundations necessary for this type construction.



Climatic Rating (C_W) Chart

Figure 14

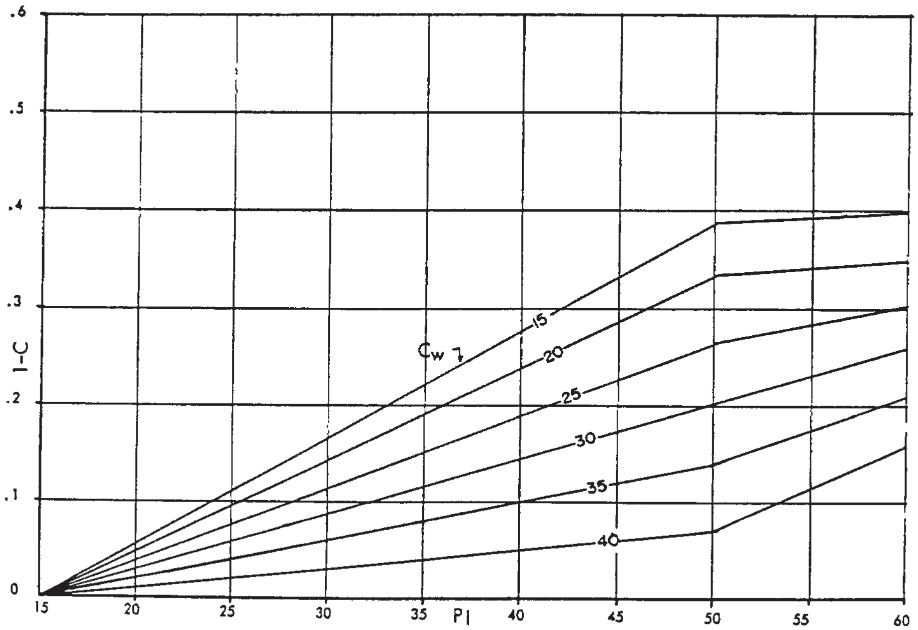
In any event, assume that for this criteria the calculated weight of the house, including the foundation for one story brick veneer type construction, is “w” lbs. per sq. ft. (The value 200 can be used for almost any single story woodframe, brick veneer type construction and not be too far from the actual weight of the house).

With the weight of the house known, refer to Fig. 14 which is extracted directly from BRAB report to select the climatic rating for the city in which the house is to built. The values for Texas range from 15 in west Texas to as high as 30 in east Texas. This chart reflects the stability of the moisture content which may be expected in the soil due to the climatic conditions which may vary from year to year. A very low number indicates an arid climate which will be very low humidity and low ground moisture except for a few weeks or months of the year when a heavy rainfall will occur and the ground will take on a considerable amount of moisture creating a potential for a large volumetric change in a short period of time. The larger numbers, such as those in east Texas, indicate in general a more humid climate where the moisture content of the soil tends to remain more uniform the year round. Refer to the BRAB report for a more complete description of this chart.

The P.I. and the climate conditions now being known, it is possible to select from Fig. 15 the soil-climate support index, indicated as (1-C).

The following formulas will be used to calculate the moment, shear and deflection, using the equivalent lengths shown in Fig. 14 as previously discussed.

These calculations are performed for both the "Long" and "Short" directions. The actual value of L and L' are used when they refer to the width of the slab. The most critical of these is deflection. A slab which deflects too much will cause serious problems for the superstructure, even though the slab does not actually break. In general then, it is best to solve first for "I required".



PI vs. (1-C)

Figure 15

The cross. section of slab is not known, but the value of 200 lbs ./sq.ft. is almost always adequate to include the slab weight.

$$M = \frac{w L ' (Lc)^2}{2}$$

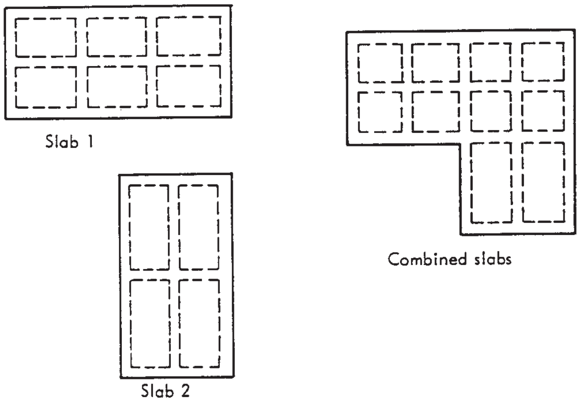
$$\Delta = \frac{w (Lc)4 L'}{4 E_C I}$$

$$V = w L' Lc$$

Where: M = Moment, positive or negative
Δ = Deflection in inches
V = Total shear
w = Unit weight
L' = Width of slab considered
Lc = Cantilever length (lck)
E_C = Creep modulus of Elasticity of concrete
I = Moment of Inertia of section

BEAM SPACING AND LOCATION

Almost all houses, if not a basic rectangle, can be divided into two or more rectangles. If the building under consideration is a combination of two or more rectangles, a set of calculations must be done for each rectangle. The rectangles are then overlaid and the heavier design governs the common areas as shown in Fig. 16. Obviously there will be times when good engineering judgement is required, as all houses are not nice neat modules.

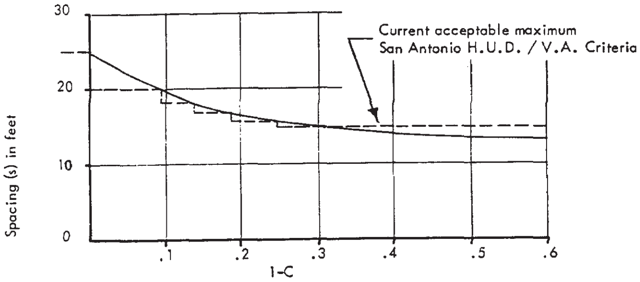


Slab Segments and Combined
Figure 16

On some occasions the geometry of the house will dictate where the beams are to be placed. When this is the case, the beams can be located, and the calculations carried out for width and depth based on the known number of beams in each rectangle.

If the design seems excessively heavy by using the maximum spacing's, it is possible to recalculate beam depths and reinforcing based on supplying additional beams.

Once the spacing and location are known, the size of the beams can be determined by trial and error. BRAB specifies that the maximum face to face distance between beams should be 15'. P.C.I. states the maximum should be 20'. Experience has shown that these are very conservative values. They are to apply to any slab on soil with P.I. of 15 or above. This is a very rigid requirement. Perhaps a more rational approach is one such as is shown in Fig. 17.



1-C vs. Maximum beam spacing
Figure 17

The designer can then use a chart such as the one shown, or the various maximums to make the first run. The number of beams then will be:

No. $= \frac{L}{S} + 1$ **Where S = Spacing from the chart**

With the number of beams known, a width for each can be selected, and a calculation made for the moment of inertia. If desired, a very good first approximation can be made by using the following formulas.

Reinforcing Steel $d = \sqrt[3]{\frac{664\ M\ L_c}{B}}$

Prestresses $d = \sqrt[3]{\frac{553\ M\ L_c}{B}}$

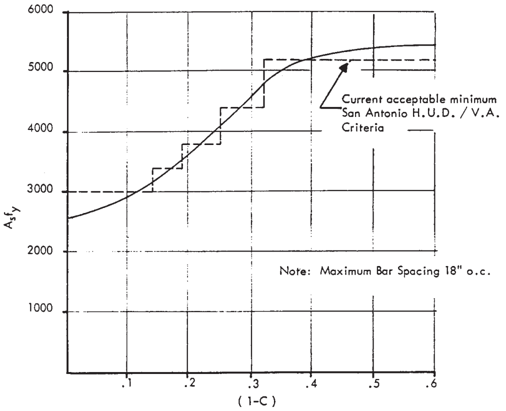
Where: d = Beam Depth
B = Sum of all beam widths
M= Moment in KF
L_c = Cantilever length

The difference in the two equations takes into account the cracked section moment of inertia vs. the gross section allowed in the prestressed slab. Anyone not wishing to use the gross section moment of inertia can use the reinforcing steel for both type slabs. These equations are good only with beam spacings no greater than those shown in Fig. 17.

These solutions will give you "N" no. of beams "b" inches wide and "d" inches deep which will give you an "I" in the order of magnitude required to limit deflection to 1/480*. It is pointless to argue about the relative merits of 1/360 vs. 1/480. In most cases depths based on 1/360 will not be economical when it comes to selecting reinforcing.

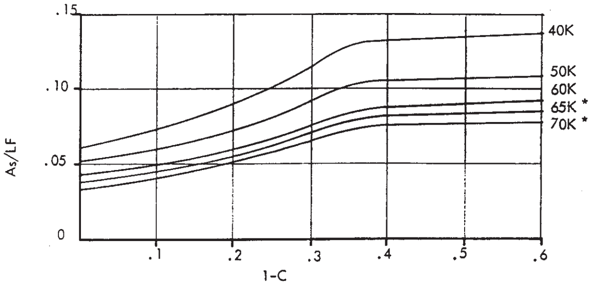
This design moment has been used to select a cross section which will resist deflection. It is now necessary to provide the reinforcing. Referring back to the BRAB (also PCI) it is necessary to provide both positive and negative reinforcing.

SLAB REINFORCING



1-C vs. A_sf_y
Figure 18

BRAB states again an arbitrary maximum spacing; the only choices are #3 bars at 12" (A_sf_y = 4400 lbs./ft. with Grade 40, or 6600 lbs./ ft. with Grade 60 bars), or in a 5" slab, or #3 bars at 10" (A_sf_y = 5200 lbs./ft. with Grade 40, 7920 lbs./ ft. with Grade 60 bars) O.C. in a 4" slab when the beam spacing exceeds 12 feet. Both are excessive in most cases. As long as the beam spacing does not exceed that shown in Fig. 17, it is possible to use less slab reinforcing.

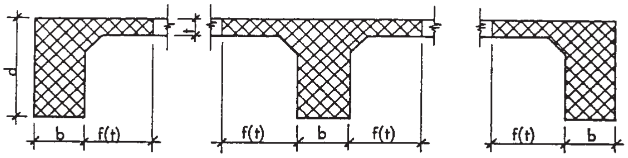


(1-C) vs. A_s/LF
Figure 19

ACI would limit the minimum reinforcing to A_sf_y = 3840 for Grade 40 and A_sf_y = 5184 for Grade 60 in a 4" slab. That is not realistic. Slabs-on-ground are not as sensitive to temperature change as suspended slabs, and need much less reinforcement. Many slabs have been done with A_sf_y less than 2600 with no ill effects. These are on low P.I. designs, of course. On high P.I. sites, the requirements of A_sf_y = 5200 as recommended in BRAB is reasonable. (Fig. 18)

BEAM REINFORCING

Solve for top beam steel based on negative moment, include slab steel falling within the cooperating slab area. (see Fig. 20).



Effective Width of "T" Beams
Figure 20

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* ACI 318-77, Table 9.5 (b) page 12 recommends 1/480 for roof or floor construction supporting or attached to non-structural elements likely to be damaged by large deflections.

*ACI 318-77 allows A_s/LF - $\frac{0.0018 \times 60,000}{f_y}$ which will further reduce the 0.0020 or 0. 0018 requirements when reinforcement with yield strengths exceeding 60,000 PSI measured at a yield strain of 0.35 percent are specified.

Solve for the bottom steel based on calculated (reduced if feasible) positive moment. Put the same size bars in a beam if 2 or more are required. It is usually best not to use more than 3 bottom bars in each beam. If that much steel is required, deepen the beams to increase the lever arms or add more beams or both.

SITE PREPARATION

Often the most overlooked part of the entire operation is the site preparation. The proper sequence should include the following:

1. Site clearing
2. Excavation (if any)
3. Fill selection and placement

Inadequate attention to any of these phases can cause foundation problems even years after the slab is built.

It is very important that the site be cleared of all grass, weeds, old decaying or decayed organics, roots and trash. This material when left under the slab can and will continue to decay and cause settlement at later dates. It is surprising how little settlement is required to cause superstructure distress. The removal of approximately six inches of top soil is usually adequate to remove grass, weeds, etc. and their roots. Trees and large bushes generally require grubbing to greater depths to insure adequate removal. This site clearing should be done prior to beginning any required excavation.

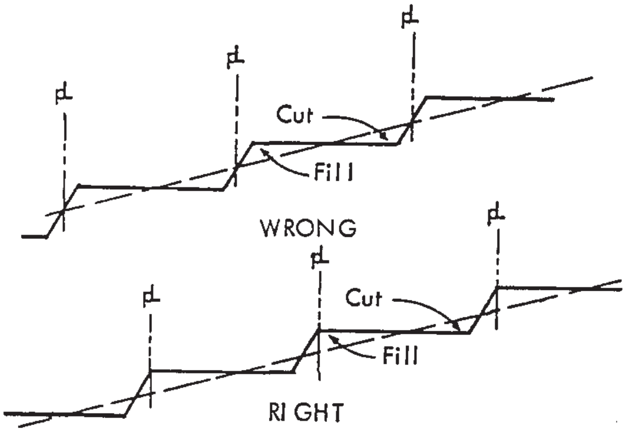
Excavation of on-site material can begin after the clearing and grubbing is completed. This allows any acceptable on-site fill material uncovered to be placed or stockpiled without contamination. This is desirable when practical because it is cost effective to handle the material only once. When a continuous, simultaneous cut and fill operation can be arranged, it will save the owner-developer quite a bit in site preparation costs.

When, for some reason, this operation cannot be arranged, it is necessary to stockpile or waste the excavated material. Stockpiles should be made on prepared sites. They should be cleared the same as a building site. Wasting should be in an area which will not be utilized later for building and which will not be subject to erosion or create drainage blockage.

All on-site material which is not suitable for structural fill should be wasted or removed from the site.

Fill selection is usually governed by the expansive qualities of the natural soil. Fill should always be as good or better than the on-site material on which it is placed. Sometimes more than one type of fill may be used.

In general, lot preparation in subdivisions is poorly done. Side slope lots requiring cut and fill on each lot are usually done without any effort to select the best material or supply any compaction to the fill.



Side Slope Lots
Figure 21

SLAB FORMING

1. Foundation forms are to be built to conform to the size and shape of the foundation, and should be tight enough to prevent leaking of mortar. The bracing must be designed so that the concrete may be vibrated without displacement or distortion of forms.

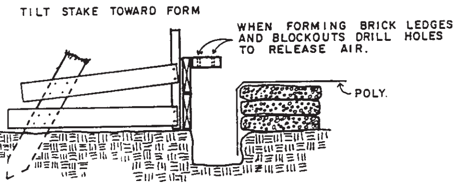
2. Beams should be formed by one of two methods:

   a. Single family slabs have been traditionally done by placing loose fill inside the forms and forming the beams with paper sacks filled with sand or fill material. For small, lightly loaded slabs this seems adequate.

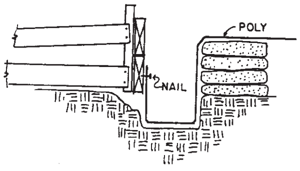
   b. Large slabs such as apartments, warehouse, shopping centers, etc., are often beamed by placing compacted fill to underslab grade and then trenching the beams with a power trencher. This method adds support to the slab and, helps it resist deflection by effectively reducing the potential expansion of underlying soil.

   Unless specified on the plans or specifications for single family foundations, it is assumed that the method described in “a” above will be used. Method “b” may be used if desired, but it is not required. For multi-family foundations or commercial work, method “b” should be required.

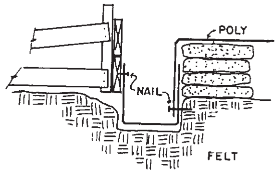
3. After the beams are Formed, a waterproof membrane should be placed.** Either 6 mil poly or hot-mopped asphalt impregnated felt may be used. The waterproofing should be lapped adequately to provide a continuous sheet under the entire slab. When poly is used, care must be taken to see that it does not become entangled in the reinforcing. Nailing the beam sides to the fill just before placing helps. At the exterior beam, the poly should be cut off at the bottom inside face of the beam and nailed as shown (Fig. 22), carried up onto the exterior form and nailed (Fig. 23), or lapped with felt and nailed (Fig. 24).



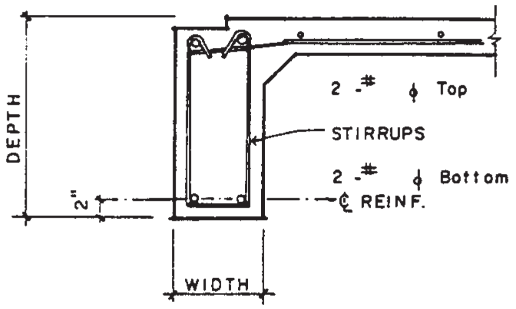
Edge Beam Forming and Staking
Figure 22



Edge Beam Forming
Figure 23



Edge Beam Forming
Figure 24



Typical Steel Placement
Figure 25

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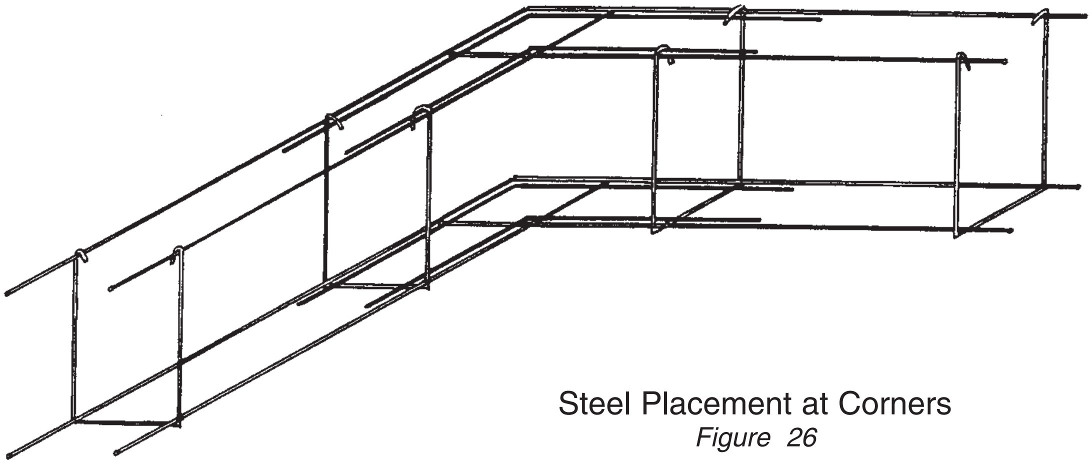
* Not universally accepted, even by all HUD/VA offices, but currently used in San Antonio-Central Texas area.

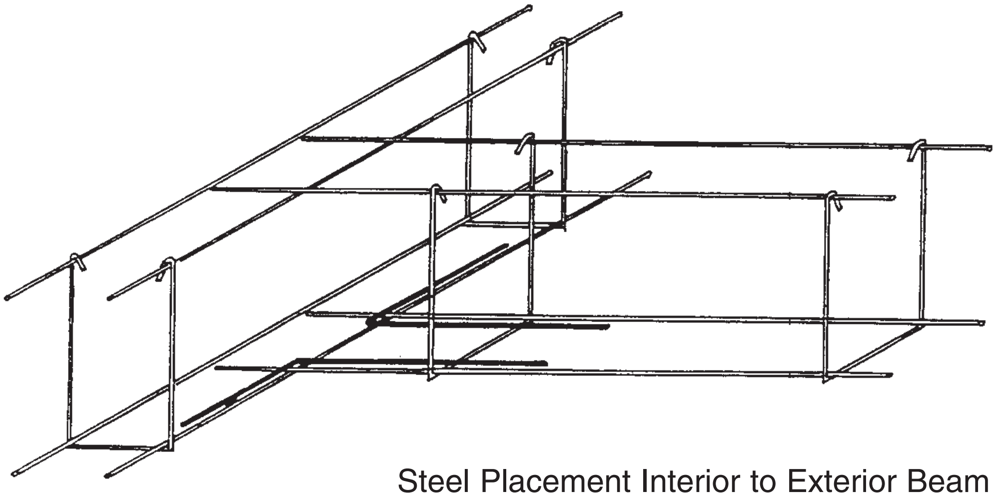
**There is much discussion over the membrane requirement, but it currently is a HUD/VA requirement.

STEEL PLACEMENT

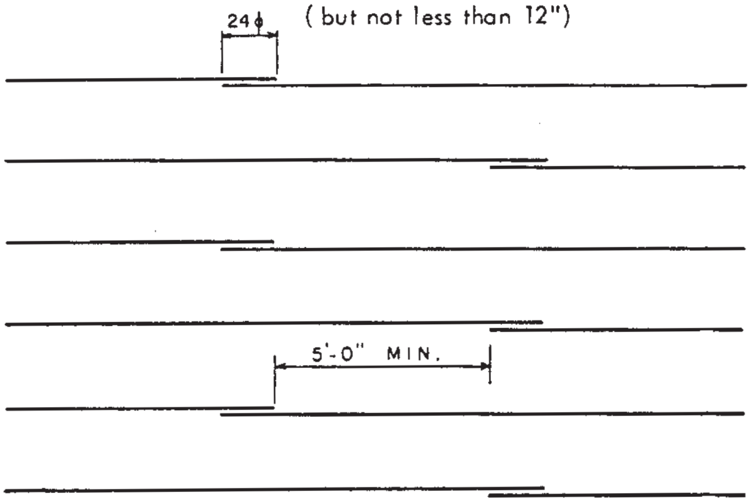
1. For the most part, steel placement in the beams will be two bars in the top and two in the bottom (Fig. 25). The bars will be held in position by stirrups at appropriate spacing. The spacing should be that which will assure the proper positioning of the steel. The bottom bars should be set on concrete bricks or blocks to keep them raised above the bottom of the beams. Corner bars equal in size to the larger size (maximum size - #6 bars) of any bars meeting at an exterior corner (Fig. 26) should be provided both top and bottom. Where interior beams dead end into exterior beams, corner bars should be supplied for bottom reinforcing only and should be the same size as the bottom bars in the interior beam or #6 bar maximum. (Fig. 27)



Steel Placement at Corners
Figure 26



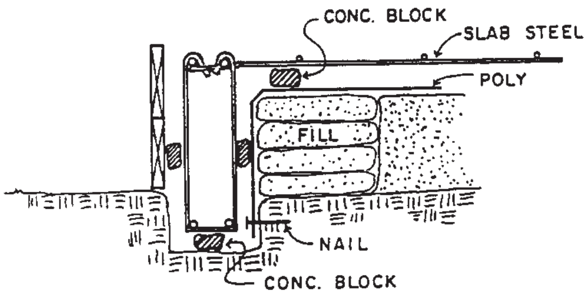
Steel Placement Interior to Exterior Beam
Figure 27



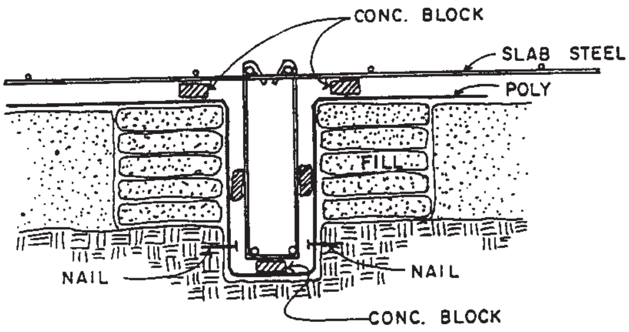
Stagger Laps in Slab Steel
Figure 28

2. After the beam steel is in place, the slab steel is placed. If it is necessary to lop slab steel, the laps in adjacent bars should be staggered at least 5' - 0" (Fig. 28)

   The slab steel is run continuously from side form to side form (lapping 24 diameters mm. where splices are required), allowing 1-1/2" cover over the ends of the bars. On the edges where the bars run parallel to the form, the first bar should be placed a maximum of 12" from the outside form. All slab steel should be securely tied and blocked up by chairs or concrete briquettes. (Figures 29 & 30)



Blocking Steel Exterior Beam
Figure 29



Blocking Steel Interior Beam
Figure 30

3. To insure the lowest possible foundation cost the use of welded wire reinforcement for slab reinforcement should be investigated. Different styles of WWR can furnish the same steel area and the following are suggested for design example:

Welded Plain Welded Wire Reinforcement

ASTM Specification A 185, fy = 65 KSI

A_S req'd = .098 x 40/65 = .060 in^2/ft.
Est. wt. = 42#/CSF
WWR 4 x 4 - W2 x W2
6 x 6 - W3 x W3
12 x 12 - W6 x W6
12 x 12 - W6 x W6 with 2-W3 outside edge wires @ 4" c/c each side.

Welded Deformed Welded Wire Reinforcement

ASTM Specification A497, fy = 80 KSI

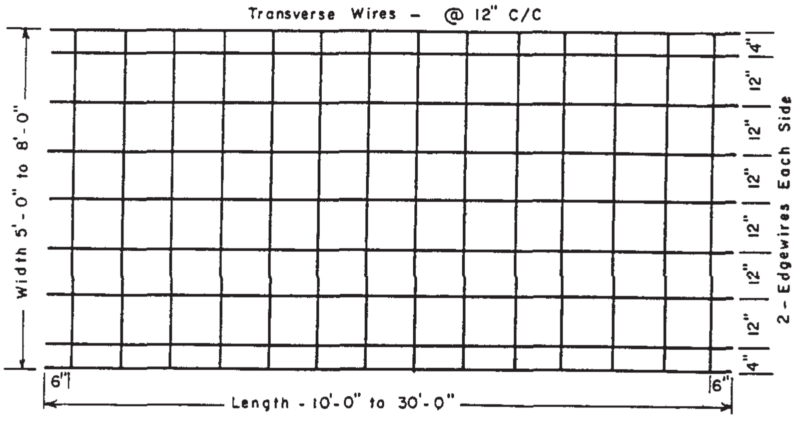
A_S req'd = .098 x 40/70 = .056 in2/ft. est. wt. 36#/CSF

WWR 16 x 16 - D6.5 x D6.5 with one D3.8 outside edge wire each side.

The two welded wire reinforcement styles with 12" spacing for smooth wire and 16" spacing for deformed wire have been recently developed to further improve the efficiency of welded wire reinforcement. The larger wire spacings make it possible to install the welded wire reinforcement at the desired location in the slab because it permits the workmen to stand in the openings and raise the welded wire reinforcement to place the supports.

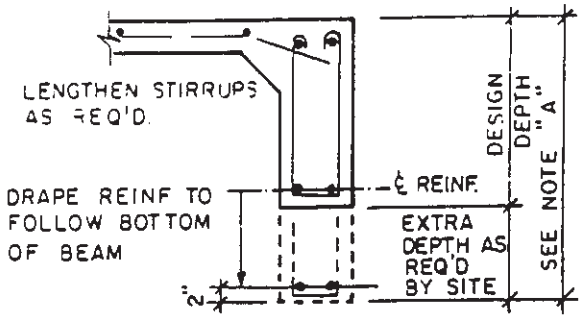
All welded wire reinforcement sheets must be spliced at both sides and ends to develop the full design fy. For smooth welded wire reinforcement ACI 318-77 requires that the two outside cross wires of each sheet be overlapped a minimum of 2 inches and the splice length equals one spacing plus 2 inches with a minimum length of 6 inches or 1.5 l_d whichever is greater. For slabs on ground the one space + 2 inches or 6" minimum will prevail. This means that for a 12" wire spacing the minimum side lap splice would be 14" but by spacing the 2 edge wires at 4" the lap is reduced to the minimum of 6". In addition the lapping of 2 wires in the splice length will provide twice the required steel area. By reducing the area of the 2 edge wires by 50%, the required A_S is provided uniformly throughout the width of the slab. This reduction in wire size does not reduce the capacity of the splice because ASTM Specification A-185 provides that the weld strength shall be not less than 35,000 times the area of the larger wire. These tonnage saving features apply only to side laps but many welded wire reinforcement manufacturers can provide sheets with variable transverse wire spacings and the length of end laps can be reduced even though wire sizes cannot.

The length of splice for deformed welded wire reinforcement is determined by the size and spacing of the spliced wires, and only the outside cross wire is lapped. While the lap length cannot be changed, the size of the outside cross wire can be reduced without changing the strength of the lap. ASTM Specification A497 stipulates that the weld shear strength shall not be less than 35,000 times the area of the larger wire. These engineered welded wire reinforcement styles are not generally available for small foundation slabs, but, when numerous small buildings or large slabs are being considered, it is prudent to check with welded wire reinforcement suppliers because substantial savings in cost can often be accomplished. As with rebar reinforcing, welded wire reinforcement must be chaired or supported on brick or blocks to insure proper placement in the slab.



Engineered Welded Wire Reinforcement Sheet with Special Side Laps
Welded Smooth Welded Wire Reinforcement
Figure 31

SPECIAL CONDITIONS

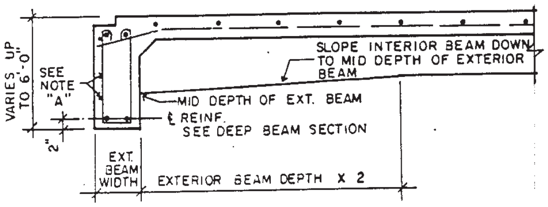


Deepened Beam
Figure 32

NOTE "A"

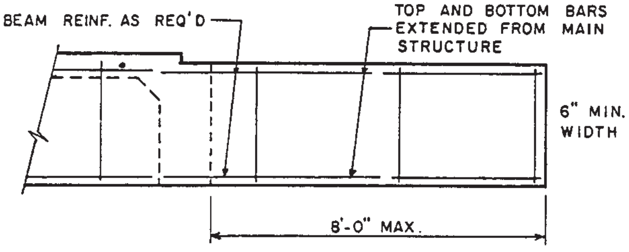
1. WHEN OVERALL DEPTH EXCEEDS 36" ADD 1 - #3 HORIZONTAL IN EXTERIOR FACE OF BEAM FOR EACH 18". START SPACING 6" FROM THE TOP.
2. FOR BEAMS 36" TO 54" DEEP USE #3 STIRRUPS AT MAXIMUM 24" O.C.
3. FOR BEAMS 54" TO 72" DEEP USE #3 STIRRUPS AT MAXIMUM I8" O.C.
4. FOR BEAMS OVER 72" DEEP CONTACT ENGINEER FOR SPECIFIC DETAILS.

1. Special conditions from time to time will arise which will require modifications to beam depths, forms, etc. Many of these are covered by typical details which illustrate what modifications are allowed without approval from the Engineer. If a special condition occurs, such as a deep beam, the Contractor needs instructions in the typical details telling how to handle the situation. In the case of the deep beam, deepen the beam by the required amount and relocate steel (Fig. 32).Refer to "Note A" to see if additional steel is required. Obviously, if the beam exceeds 72", the engineer must be contacted for additional information.



Interior to Exterior Beam
Figure 33

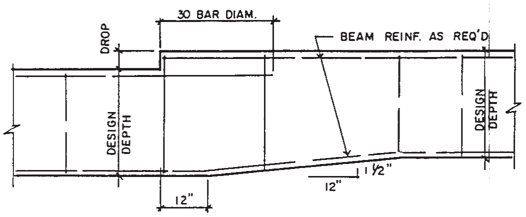
2. When an exterior beam is deepened, some slight changes must be made to the interior beams which intersect the deepened beam (Fig. 33). The bottom of the interior beam should slope down at least as deep as the mid-depth of the deepened exterior beam. If the interior beam depth is already deeper than the mid-depth of the deepened exterior beam, no changes are required to the interior beam.



Extended Concrete Beam
Figure 34

3. Extended beams to carry wing walls should be handled as shown in the typical detail (Fig. 34). It is very important that

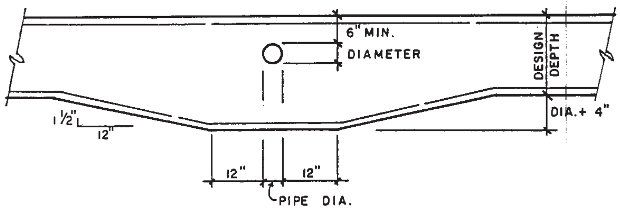
the additional top reinforcing be added as shown; otherwise the beam may be broken off.



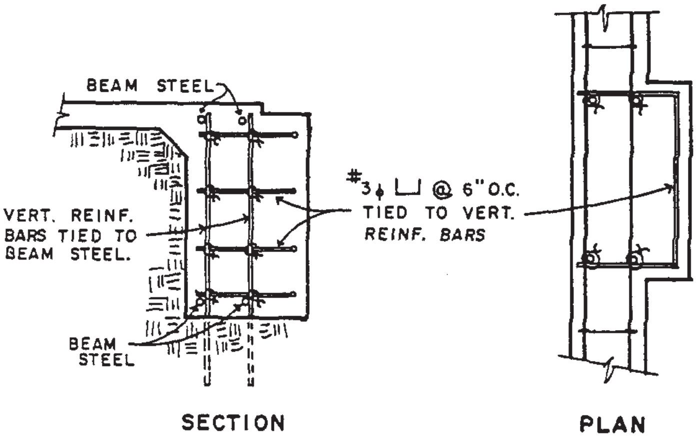
Side View of Typical Beam at Drop
Figure 35

4. Beams that continue through drops must be deepened by the amount of the drop, and the transition sloped (Fig. 35). If the drop is framed as a sharp corner on the bottom of the beam, stress concentrations can occur which may cause difficulties.

5. Other special conditions may arise from time to time but they are too numerous to be covered here.



Drop under Sleeve through Beam



Side View of Typical Beam at Drop
Figure 36

CONCRETE PLACING

Over the years the word “pouring” has come to be used almost exclusively to describe the function of placing concrete. Unfortunately that term is all too descriptive of the practice which has become common throughout the industry. When placing concrete for an engineered foundation, it is imperative that the concrete actually be placed, not “poured”.

Residential floors need to have adequate strength, surfaces that are hard and free of dusting, and the cracking should be held to a minimum. The hardness and finish of the surface will depend on how densely the surface materials are compacted during finishing, and the adequacy of the cement paste. Cracking however is mainly a function of the drying shrinkage which takes place immediately after placing and is generally more controlled by atmospheric conditions than by the consistency of the concrete itself. This is a gross generalization, however, as high water cement ratios will increase the shrinkage problem. Good concrete for slabs-on-ground should be made from a mix in which the water cement ratio is kept low and should contain as much coarse aggregate as possible at the surface. Compressive strengths for concrete slab-on-ground foundations are generally specified as a minimum of 2500 PSI at 28 days. It is important to note the word minimum. The 2500 PSI should be the minimum strength, not the average strength.

This 2500 PSI is a generally accepted figure in the industry, since it has become an accepted figure for HUD/VA construction.

In keeping the water cement ratio low, add mixtures can be of particular benefit. This is particularly true with respect to air entrainment, retardants and accelerators.

Calcium chloride is a common cold weather additive to accelerate settling and hardening. It should properly only be added to the mix in the mixing water. It is important to emphasize that calcium chloride is not to be used in a foundation which is pre-stressed. The use of calcium chloride in foundations with rebar reinforcing or welded wire reinforcing must be limited to a minimum of 2% by weight of cement.

Several operations need to completed before beginning the placing of concrete. Screeds should be set inside the form area to establish finished slab grade prior to beginning concrete placing. This will improve the level of the finished slab and eliminate much of the unevenness of the slabs currently found. Keys for joints may, on certain occasions, be used as screeds, since they need to be placed at proper intervals in large slabs to eliminate or control the shrinkage cracking.

When the concrete is delivered it should be placed as close as possible to its final position in the foundation. It should be spread with short handle, square ended shovels and not by the use of rakes. Internal vibration at the time of placing should be mandatory, as this allows a stiffer mix to be used and facilitates placing.

Screeding, tamping, and bull floating will be of course finished prior to the time the bleed water has accumulated on the surface. After the bull floating, the final finishing should not begin until bleed water has risen and evaporated, and the water sheen has disappeared from the surface. At the time the concrete shall be stiff enough to sustain a man's foot pressure without indentation.

With regard to final finishing operations, the accumulation and evaporation of bleed water will vary considerably with weather conditions and types of mixes. When bleed water is too slow to evaporate, it may be pulled off with a hose, or blotted with burlap. The surface of a foundation should never be dried with what is commonly called dusting. This is a method whereby dry cement and, sometimes, dry cement and sand is placed on the slab to blot the bleed water. This will cause a weak surface and, possibly, subsequent deterioration of the surface.

Immediately after the foundation has been finished, a curing compound should be placed to inhibit further evaporation of water from the concrete mix. This will tend to reduce the amount of shrinkage cracking which will occur in the foundation.

When using a liquid curing membrane, it is important to select a compound that will not interfere with future bonding of floor finishes. There are several such compounds on the market.

Forms should remain on the finished concrete slab for a minimum of 24 hours. Removal prior to that time can cause damage to the concrete. After 24 hours the forms can be carefully removed without damaging the concrete.

INSPECTION

The most general problem encountered was lack of suitable field inspection and control. If the foundations are not constructed in accord with the design drawings and specifications, then benefits to be derived from improvements in codes or the state of the art will be diminished.” [20]

Before placing, the contractor should call for an inspection by some inspection agency. For FHA-VA single family construction, this is handled by the FHA or VA. For non FHA/VA houses it is, or should be, handled by the city, but most cities, especially small ones, do not have enough staff.

Cities should require adequate inspection,, either by their own forces, or by the design engineer who should, after all, be most familiar with his own design.

When the Engineer is not permitted to check the construction, one of the other inspectors should furnish a certificate to the Engineer that the slab is properly installed in accordance with the Engineer s plan. The following is a partial list of points which should be verified by the inspector.

1. Check number of beams
2. Measure beam width
3. Measure beam depth
4. Check beam spacing
5. Check tightness and alignment of forms
6. Check blocking under beam reinforcement
7. Check compliance with fill penetration or have fill certification
8. Check beams for proper number and size of reinforcing bars
9. Check the slab reinforcing for proper size and spacing.
10. Check to see that all slab reinforcing is adequately blocked to insure proper placement in concrete.
11. Test concrete for maximum 6” slump
12. Make cylinders for strength certification
13. See that concrete is vibrated or rodded
14. Insure adequate curing
15. Check for cracking or honeycombing

SHRINKAGE CRACKS

All concrete has cracks! There is not yet in the industry the ability to produce crack-free concrete. What can best be done is to limit or reduce the amount and kinds of cracks.

Those cracks that occur prior to the hardening of the concrete are generally formed by the movement of the form work, settlement of the concrete during setting, or plastic shrinkage cracks which occur while the concrete is still plastic. Other cracks which can occur after the setting of concrete are shrinkage cracks due to drying of the concrete, thermal cracks due to changes in internal heat of hydration or due to external temperature variations, cracks due to stress concentrations, or cracks due to structural overloads.

The most common cracks which are seen in foundations are plastic shrinkage cracks which occur early after the concrete is placed and are due to the rapid drying of the fresh concrete. Even if plastic cracking does not occur, similar type cracks can form during the early stages of hardening even days after the final finishing has taken place. While curing membranes will not eliminate the plastic shrinkage cracks that occur prior to setting, they can be very beneficial in reducing or eliminating the shrinkage cracks which will occur after finishing.

The effects of temperature, relative humidity, and wind velocity are, in general, beyond the control of the engineer or the contractor and must be accepted as risks when the slab is placed.

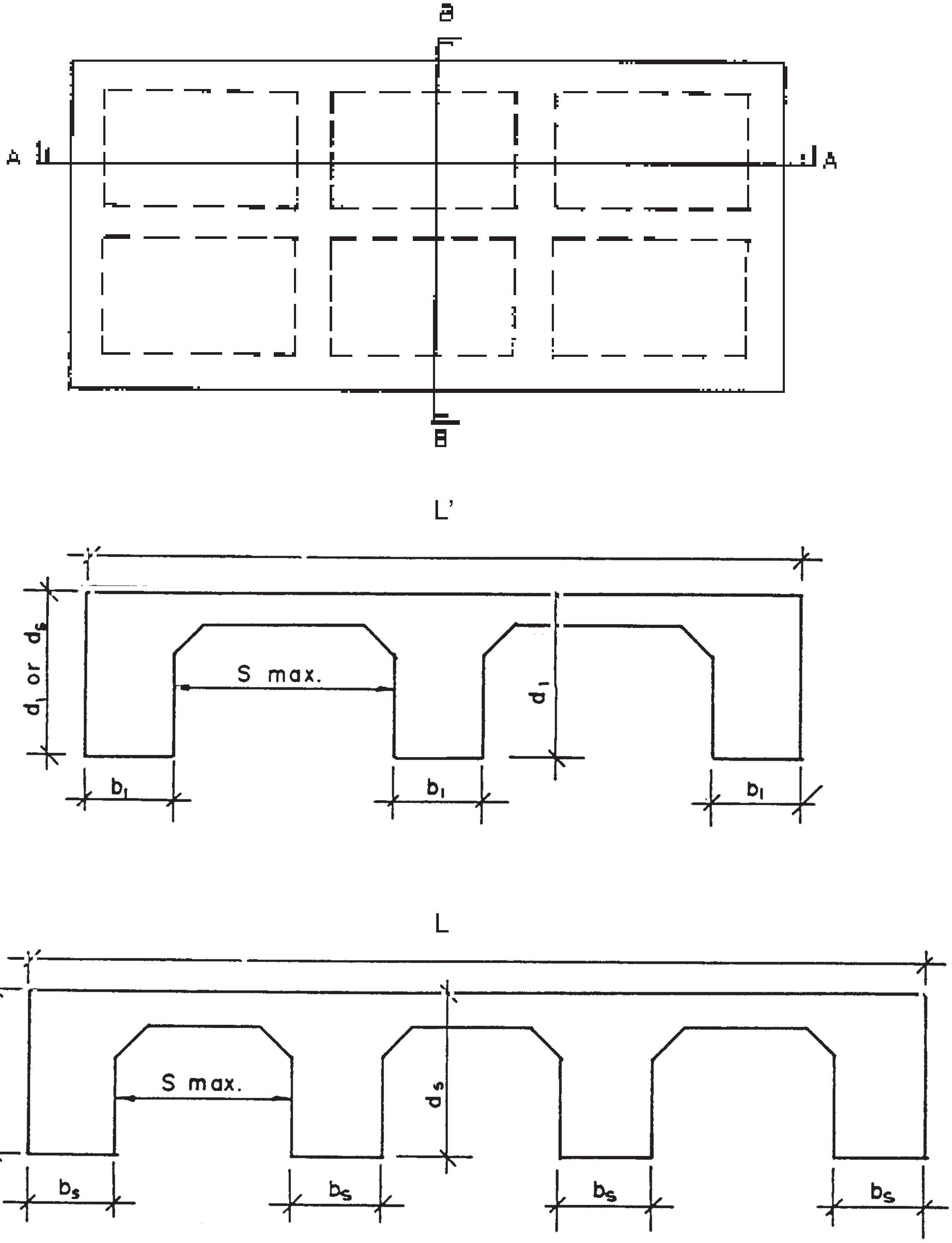
It is, therefore, wise to specify the minimum sacks of concrete which will be expected to give the recommended compressive strength, utilize the minimum water content necessary for workability, and not permit over-wetting of concrete on the job. It cannot be said too often that the use of internal vibration will facilitate placing of concrete and help eliminate internal settlement. The use of a surface curing membrane, placed as soon as possible after final finishing, will help eliminate shrinkage cracks which are caused by drying of the hardened concrete.

It is common practice in commercial work to use control joints to reduce shrinkage crack problems. These are good procedures, but not commonly used in single family construction.

Again, it is important to recognize that all shrinkage cracks cannot be eliminated, given the present state of the art.

NOTATION

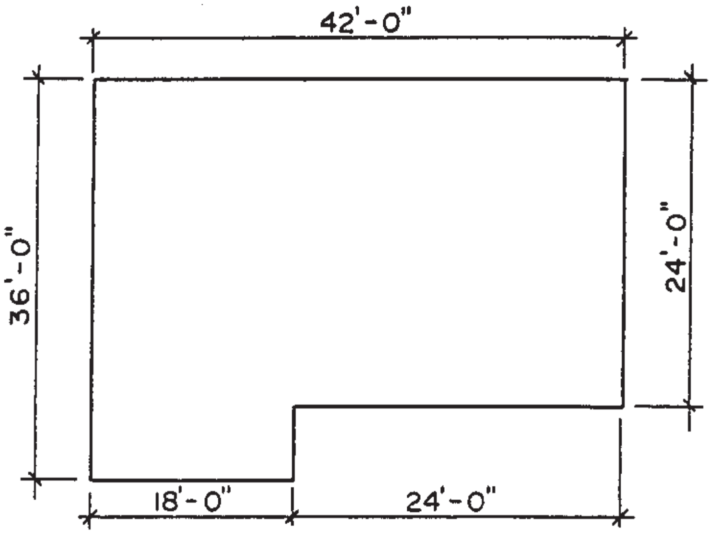
A_c = Gross Area of Concrete Cross-Section
A_{ss} = Area of Steel Reinforcing in Slab
A_{sbb} = Area of Steel Reinforcing in Bottom of Beam
A_{stb} = Area of Steel Reinforcing in Top of Beam
a = Depth of Stress Block (ult. strength)
b_b = Width of Beam Portion of Cross-Section
b_s = Width of Slab Portion of Cross-Section
B = Total Width of all Beams of Cross-Section
C_w = Climatic Rating
d_b = Depth of Beam Portion of Cross-Section
d_s = Depth of Slab Portion of Cross-Section
E = Modulus of Elasticity of Concrete
E_c = Creep Modulus of Elasticity of Concrete
$f'c$ = 28 Day Compressive Strength of Concrete
f_y = Yield Strength of Reinforcing
I_g = Gross Moment of Inertia of Cross-Section
I_o = Moment of Inertia of Segments of Slab Cross-Section
k_l = Length Modification Factor-Long Direction
k_s = Length Modification Factor-Short Direction
L = Total Length of Slab in Prime Direction
L' = Total Length of Slab (width) Perpendicular to L
Lc = Design Cantilever Length (lck)
l_c = Cantilever Length as Soil Function
M_l = Design Moment in Long Direction in kft
M_s = Design Moment in Short Direction in kft
N_l = Number of Beams in Long Direction
N_s = Number of Beams in Short Direction
PI = Plasticity Index
S = Maximum Spacing of Beams
V = Design Shear Force (Total)
v = Design Shear Stress (Unit)
v_c = Permissible Concrete Shear Stress
w = Weight per sq. ft. of House and Slab
q allow = Allowable Soil Bearing
q_u = Unconfined Compressive Strength of Soil
i-c = Soil/Climatic Rating Factor
Δ allow = Allowable Deflection of Slab, in.



Minimum number, width and depth of beams

DESIGN EXAMPLES

For comparison to BRAB Report #33, assume for a design example the same single story residence located in San Antonio, Texas, which was used in BRAB Report #33.



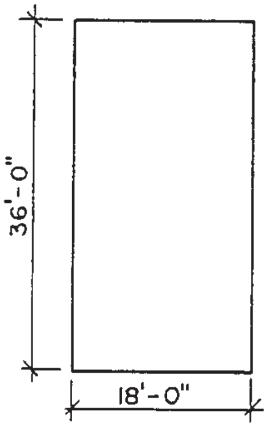
Assume:

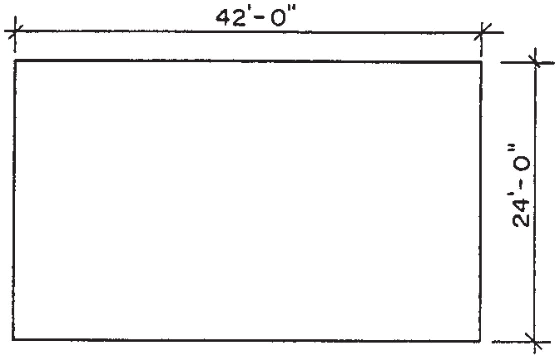
Effective P.1. = 37
Climatic rating C = 17
Slope = 0%

Unconfined compression qu = 2800 >1 TSF.
Unit weight = 200 lbs./sq. ft.

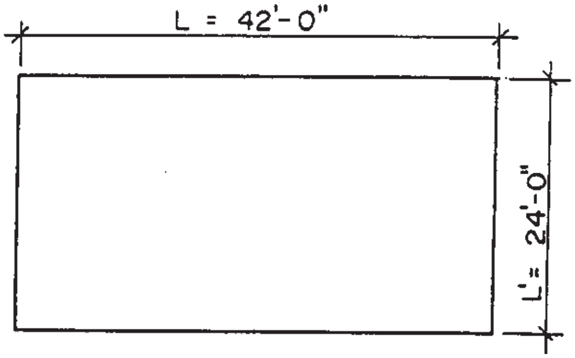
3 x 5 x 41 = 615
2 x 5 x 41 = 410
5 x15 = 75
EPF P.1. = 1100/300 = 36.67

First divide slab into two overlapping rectangles.





For purposes of example, solve the 24' - 0" x 42' - 0 rectangle



From Fig. 15, 1-C = .23
From Fig. 17, S = 16
From Fig. 12, I_c = 7
From Fig. 13, K_l = .95 K_lI_c = .95 x 7.0 = 6.65
K_s = .80 K_sI_c = .80 x 7.0 = 5.60

Number of beams in long direction $N_l = \frac{24.0}{16.0} + 1 = 2.5 = 3$

Number of beams in short direction $N_s = \frac{42.0}{16.0} + 1 = 3.6 = 4$

Assume beam widths = 9" each beam

B_L = 3 x 9 = 27"
B_S = 4 x 9 = 36" Geometry of house causes 5 beams B_S = 45

Solve for long and short moments:

$$M_L = \frac{200\ (6.65)^2 \times 24}{2000} = 106^{kf}$$

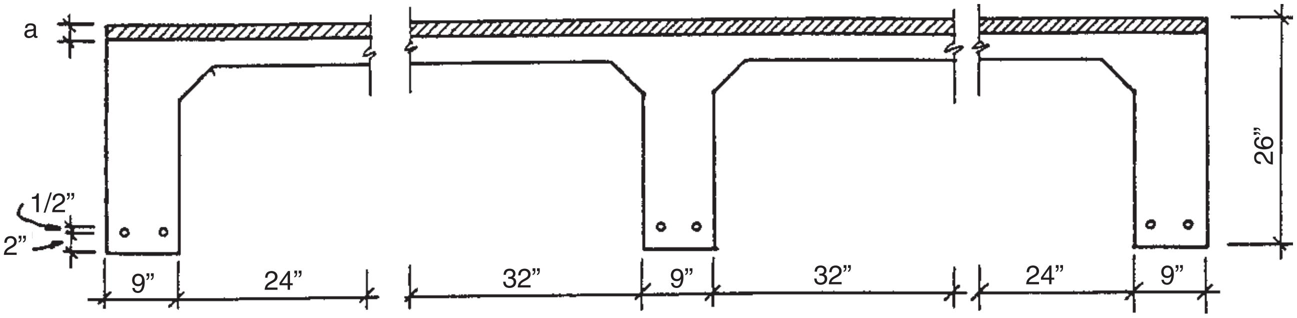
$$M_s = \frac{200\ (5.60)^2 \times 42}{2000} = 132^{kf}$$

Solve for beam depths:

$$d_L = \sqrt[3]{\frac{664 \times 106 \times 6.65}{27}} = 25.9" = 26"$$

$$d_S = \sqrt[3]{\frac{664 \times 132 \times 5.60}{45}} = 22.2" = \text{“say 22”}$$

Solve for steel in bottom of beams : Long direction



Using $f_y = 60{,}000$

Assume: 6 - #5 bars $A_s = 1.86$ sq.in.

$$a = \frac{A_s f_y}{0.85 f'cb} = \frac{1.86 \times 60{,}000}{0.85 \times 2500 \times 139} = 0.378$$

Assume: lever arm for positive reinforcing = d-3"

$$M_u = 1.86 \text{ x}60\ (26\text{-}3) / 12 = 213.9$$
$$M = 213.9 / 1.6 = 133.7 \text{ vs. } 106$$

or using $f_y = 40{,}000$

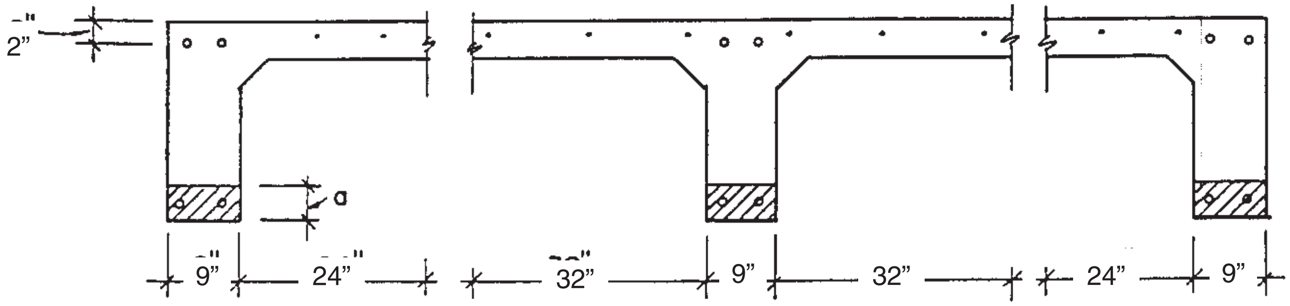
Assume:6 - #6 bars $A_s = 2.64$ sq.in.

$$a = \frac{A_s f_y}{0.85 f'cb} = \frac{1.86 \times 60{,}000}{0.85 \times 2500 \times 139} = 0.358$$

Assume: lever arm for positive reinforcing = d-3"

$$M_u = 2.64 \times 40\ (26\text{-}3) / 12 = 202.4$$
$$M = 202.4 / 1.6\text{-}\ 126.5 \text{ vs. } 106$$

$A_s f_y$ for all three of these combinations = 3.85 k/lf



Assume lever arm for negative reinforcing = d - 4"

- M_u from slab steel - steel in flanges only

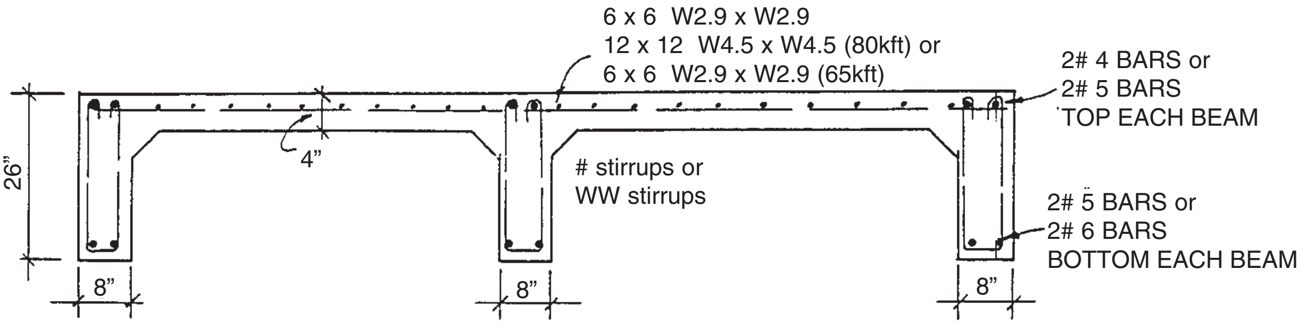
112” / 12 x 3.85 x (26-4) / 12 = 65.9 kf/1.6 - 41.2
-M - 41.2 kf = Moment to be reinforced for 106 - 41 = 65kf

For f_y = 60,000
Assume: 6 - #4 bars A_s = 1.20
M_u = I.20 x 60 x (26’- 4”) / 12 = 132
M = 132 / 12 = 82.5 > 64

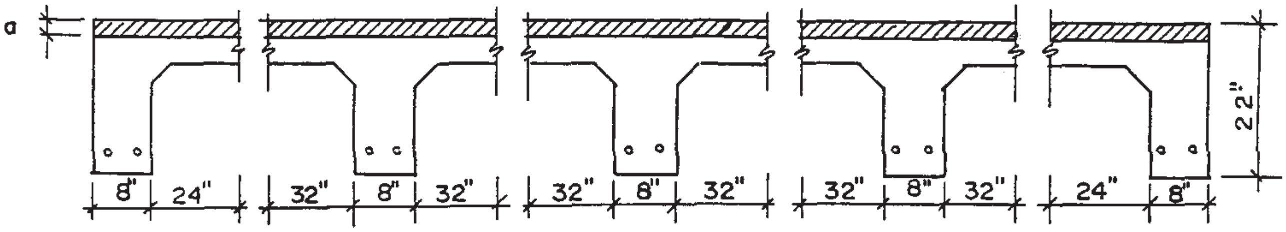
For f_y = 40,000

Assume: 6 - #5 bars A_s = 1.86
$A_s f_y$ = 1.86 x 40 = 74.4k
M_u = 74.4 x (26-4) / 12 = 126.4kf
M = 136.4 / 1.6 = 85.2 > 64

A cross section taken across the slab would then show:



Solve for steel in bottom of beams : Short direction



For f_y = 60,000

Assume: 10 - #4 bars A_s = 2.00

$$a = \frac{2.00 \times 60}{0.85 \times 2.5 \times 240} = 0.235"$$

Assume: lever arm for negative moment = d-3

M_u = 2.00 x 60 (22-3) / 12 = 190.0

M = 190 / 1.6 119 vs. 132

Try increasing two exterior and center beams to 26"

M_u (ext .beams) = 1.80 x 60 (26-3) / 12 = 138kft
M_u (int. beams) = 0.80 x 60 (22-3) / 12 = 76kft
Total M_u = 138 + 76 = 214kft
Total M = 214 / 1.6 = 134 vs. 132

For f_y = 40,000

Assume: 10 - #5 A_s = 3.10 sq.in.

$$a = \frac{3.10 \times 40}{0.85 \times 2.5 \times 240} = 0.243"$$

Assume: lever arm for negative moment = d-3

Mu = 3.10 x 40(22-3) / 12 = 196kft
M = 196/1.6 = l22 vs. 132kft

Try increasing two exterior beams to 26”

Mu (ext. beams) = 1.24 x 40 (26’ 3’)/ 12 = 95kft

Mu (mt. beams) = l.86 x 40(22’ 3”)/ 12 118

Total Mu = 95 + 118 = 213kft

Total M =213/1.6 = 133 vs. 132

Solve for steel in top of beams: Short direction slab steel same as long direction

-M slab steel - steel in flanges only

M slab steel 240/ 12 x 3.85 (22’ 4”)/ 12 = 111.6/ 1.6-73.5kft

-M -73.5 = 132 -73.5 = 58.5kft moment to be reinforced for.

(This is slightly conservative since beams are deepened)

For f_y = 60,000

Assume: top steel to be 10 - #3 bars or W or D9 @ 10” ea. way
$-M_u$ = 0.66 x 60 (26-4) / 12 = 72.6

0.44x 60 (22-4) / 12 = 39.6

Total $-M_u$ = 72.6 + 39.6 = 112.2

$-M_u$ = 112.2 / 1.6 = 70.1 > 58.5

For f_y = 40,000

Assume: top steel to be #3 bars or W or D9 @ 10” ea. way

$-M_u$ = 0.80 x 40 (22-4) / 12 = 58.7

1.20 x 40 (22-4) / 12 = 72.0

Total $-M_u$ = 130

-M = 130.7 / 1.6 = 81.7 > 58.5kft

SUMMARY:

Long Direction Beams

3-9” x 26” beams, reinforced with 2 #4 or 2 #5 bars top, 2 # 5 or

2 #6 bars bottom each beam

Short Direction Beams

2-9” x 26” exterior and center interior beams, reinforced with

2 #3 or 2 #4 bars top, 2 #4 or 2 #5 bars bottom.

2-9” x 22” interior beams, reinforced with 2 #3 or 2 #4 bars top,

2 #4 or 2 #5 bars bottom.

Slab reinforcing to be:

f_y = 80,000 - 12 x 12 - W4.5 x W4.5

f_y = 65,000 - 6 x 6 - W2.9 x W2.9

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APPENDIX D4

GEOTECHNICAL DESKTOP STUDY
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE PROJECT
CITY OF SAN DIEGO

Submitted to:

BROWN & CALDWELL
9665 Chesapeake Drive
San Diego, CA 92123

Prepared By:

ALLIED GEOTECHNICAL ENGINEERS, INC.
9500 Cuyamaca Street, Suite 102
Santee, California 92071-2685

September 19, 2014

AGE

Allied Geotechnical Engineers, Inc.

September 19, 2014

Kathy Haynes, PE
Supervising Engineer
Brown & Caldwell
9665 Chesapeake Drive
San Diego, CA 92123

Subject: **GEOTECHNICAL DESKTOP STUDY**
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE PROJECT
CITY OF SAN DIEGO
AGE Project No. 142 GTS-12-A

Dear Ms. Haynes:

In accordance with your request, we are pleased to submit this report which presents the findings, opinions and recommendations of a geotechnical desktop study that we have performed for the above-mentioned subject project.

We greatly appreciate the opportunity to be of service on this important project for the City of San Diego. Should you have any questions or need further assistance, please feel free to give us a call.

Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

Nichola E. Ba

Exp 5/31/2014
Nicholas E. Barnes
No. 1693

Nicholas E. Barnes, P.G./C.E.G.
Senior Geologist

Sani Sutanto
No. 51873
Exp. 06/30/2016

Sani Sutanto, P.E.
Senior Engineer

NB/SS/TJL:sem
Distr. (3 hard copies and 1 electronic copy) Addressee

GEOTECHNICAL DESKTOP STUDY
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE PROJECT
CITY OF SAN DIEGO

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GEOTECHNICAL DESKTOP STUDY
NORTH CITY TO SAN VICENTE RESERVOIR PROJECT
CITY OF SAN DIEGO

1.0 Project Description

The proposed North City to San Vicente Reservoir Pipeline Project alignment is approximately 27 miles long and extends between the North City Advanced Water Treatment Plant (NCAWTP) and San Vicente Reservoir (SVR). The project alignment traverses along existing public roadways and across open space within the Cities of San Diego and Santee and the community of Lakeside. The pipeline will also cross the United States Marine Corps (USMC) Air Station Miramar and the Miramar landfill. We understand that the pipeline is anticipated to be 42-inch diameter, and is to be installed using conventional cut and cover construction methods with a minimum backfill of 42-inches above the top of pipe. We further understand that trenchless technology will be utilized to install segments of the project alignment at freeway and major river crossings, and between San Vicente Dam and the SVR Discharge Structure.

Due to the length of the pipeline and varying geologic/geotechnical conditions which will be encountered the alignment has been subdivided into six separate reaches. The pipeline alignment and the reaches are shown on the Alignment Map (Figure 1). Land uses along the proposed alignment include a mix of residential and commercial developments, USMC Air Station Miramar, Miramar Landfill, various public works facilities, San Vicente Dam, and undeveloped open space. Elevations along the proposed pipeline corridor vary from a low at approximately +95 feet above mean sea level (msl) at the San Diego River crossing near Princess View Drive to a high of +1200 feet msl at the northeast end of the alignment near San Vicente Dam (GoogleEarth, 2014).

The reaches, designated A-E, are described as follows:

Reach A - NCAWTP to intersection of Copley Drive and Hickman Drive

Reach A begins at the NCAWTP which is located on Eastgate Mall in San Diego. From this location the project alignment extends east and southeasterly along Eastgate Mall to Miramar Road. The alignment then turns west a short distance on Miramar Road, and continues in a south to southeasterly direction through USMC Air Station Miramar near the Miramar National Cemetery. The alignment crosses railroad tracks and Rose Canyon through the open space, then continues southward along the east edge of a large wholesale nursery operation on the base. Near the southeast corner of the nursery the alignment crosses into the lease-area boundary of Miramar Landfill, and continues eastward along the north side of the active West Miramar Landfill Phase II. The alignment then turns south on a service road dividing the active landfill with the inactive West Miramar Landfill Phase I (City of San Diego, 2014a and b). South of the Miramar Landfill lease-area the alignment crosses San Clemente Canyon through open space, and then continues across SR 52 east of I-805.

Reach B - Intersection of Copley Drive and Hickman Drive to Mission Gorge Road

From the end of Reach A, Reach B proceeds south and easterly along Copley Drive, Copley Park Place, Mercury Street and Ronson Road, and crosses I-163 near Ronson Road and continues eastward along Lightwave Avenue. At Ruffin Road the alignment turns north, then east on Clairemont Mesa Boulevard and south on Murphy Canyon Road. Approximately 0.35 miles south of Clairemont Mesa Boulevard the alignment turns east and crosses I-15, then continues in a northeasterly direction through open space within an unnamed canyon to Clairemont Mesa Boulevard. The alignment then continues in a southeasterly direction along Clairemont Mesa Boulevard and turns south onto Santo Road. At Tierrasanta Boulevard the alignment turns east, following the trend of Tierrasanta Boulevard in a southeasterly direction. Past a cul-de-sac at the southeast end of Tierrasanta Boulevard the alignment continues in a southeasterly direction through open space, crossing the San Diego River and intersecting Mission Gorge Road near Princess View Drive.

Reach C - Mission Gorge Road to West Hills Parkway

Reach C starts at Princess View Drive and continues in a general northeasterly direction along Mission Gorge Road. In the central portion of the reach, the alignment crosses a saddle between Cowles and Fortuna Mountains within Mission Trails Regional Park. Approximately 4,000 feet southwest of West Hills Parkway the alignment enters into the City of Santee.

Reach D - West Hills Parkway to Highway 67

From the end of Reach C, Reach D continues in a northerly direction along West Hills Parkway, crosses the San Diego River and SR 52 before turning east onto Carlton Oaks Drive. Reach D continues along Carlton Oaks Drive in an east to northeast direction, and crosses Sycamore Canyon south of Santee Lakes. The alignment then curves northward and the road name changes to Halberns Boulevard. At the intersection with Mast Boulevard the alignment turns east, and continues past a dead end on Mast Boulevard and crosses open space between Santee and the unincorporated community of Lakeside. Within approximately 0.25 miles Mast Boulevard resumes, with the road name changing to Riverside Drive at the intersection with Riverford Road. The alignment continues in an easterly direction along Riverside Drive, turning northeast at Lakeside Avenue which the alignment then follows to Highway 67.

Reach E - Highway 67 to San Vicente Dam

From Lakeside Avenue the alignment follows Highway 67 approximately 0.1 miles north, and turns east at Willow Road. The alignment then crosses San Vicente Creek, and turns north along Moreno Avenue. At Vigilante Road Moreno Avenue veers northeast toward SVR. From this point the alignment continues past the guard shack and into a City of San Diego maintenance yard located approximately 2,000 feet south of San Vicente Dam. This maintenance yard is located near the historic town of Foster, a stage stop and railroad station that was occupied from 1880 until the land was sold to the city of San Diego in the 1930's for the San Vicente Dam (EDAW, 2009).

Reach F - San Vicente Dam to San Vicente Reservoir Discharge Structure

From the City’s maintenance yard the alignment turns east through open space, then continues in a northeasterly direction along the southeast side of SVR to the discharge structure.

2.0 OBJECTIVE AND SCOPE OF STUDY

The objective of this desktop study is to provide general information and to evaluate potential major geologic and geotechnical issues and constraints which could impact the proposed project alignment. The scope of the desktop study includes the performance of several tasks/services which are more fully described below.

2.1 Information Review

For this task, we have reviewed information pertaining to the project area that was readily available from a variety of sources which include the following:

- AGE's in-house references and aerial photographs;
- Published geologic literature and maps, including geologic and fault maps published by the City of San Diego, California Geological Survey and United States Geological Survey;
- Pertinent project-related information, including geotechnical reports prepared by others;
- Aerial photography available at Google Earth.

A listing of the references that were reviewed for this study is presented in Section 7.0.

2.2 Site Reconnaissance

The information obtained from our literature review was supplemented with visual observations gathered during our field reconnaissance visits that were conducted on August 26 , 29 and September 9, 2014. The purpose of the site visits was to observe existing site conditions and geologic exposures along the project alignments and in surrounding areas.

2.3 Data Evaluation and Reporting

This task involved a synthesis and evaluation of the data collected during the information review and field reconnaissance phases of this study, particularly with respect to known and anticipated geotechnical conditions and potential geologic hazards, such as faulting and seismicity; seismic-induced hazards, slope stability issues, and landslides. Based on an evaluation of the data, we have prepared this report to present a summary of our preliminary findings and opinions.

3.0 GEOLOGIC CONDITIONS

3.1 Geologic Setting

The project study area is located within the Peninsular Ranges geomorphic province, a north-south oriented mountain range which extends from the southern edge of the Los Angeles Basin into Baja California, Mexico. Basement rocks of the Peninsular Ranges province include Cretaceous crystalline rocks of the Southern California Batholith and Jurassic metasedimentary and metavolcanic rocks of the Santiago Peak Volcanics. The basement rocks are exposed in the easternmost portion of the alignment near SVR and along portions of Reach C within Mission Trails Regional Park.

The majority of the project alignment is situated within the San Diego Embayment, a deep sedimentary-filled basin which is underlain at depth by the basement rock complex. The sedimentary formations consist of nearly flat-lying to gently southwest dipping, marine and non-marine sediments which range from Cretaceous to Holocene in age.

Mapped geologic units along the project alignment include Eocene to Holocene age sedimentary deposits and Jurassic/Cretaceous basement rocks. Although not shown on the published maps, man-made fills are known to occur at various locations along the project alignment.

3.2 Tectonic Setting

Tectonically, the San Diego region is situated in a broad zone of northwest-trending, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province west of California and northern Baja California. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 50 miles to the west, and the San Andreas fault located about 90 miles to the east.

Major active regional faults of tectonic significance include the Coronado Bank, San Diego Trough, San Clemente, and Newport-Inglewood/Rose Canyon fault zones which are located offshore; the faults in Baja California, including the San Miguel-Vallecitos and Agua Blanca fault zones; and the faults located further to the east in Imperial Valley which include the Elsinore, San Jacinto and San Andreas fault zones.

3.3 Geologic Units

For site characterization purposes, the subsurface materials along the project alignment can be categorized into ten (10) geologic units, which include (in order of increasing age): fill materials; young alluvial deposits; old alluvial deposits; Very Old Paralic Deposits; Mission Valley Formation; Stadium Conglomerate; Friars Formation; Scripps Formation; granitic rocks; and undifferentiated metasedimentary and metavolcanic rocks. Each geologic unit can be distinguished by its origin or depositional character and has different compositional characteristics.

The location and distribution of these geologic units are depicted on the generalized geologic map which has been compiled based on the findings of our information review and field reconnaissance visits (see Generalized Geologic Map, Figure 2).

3.3.1 Fill Materials

Fill materials associated with roadway construction and land developments were observed at various locations along the project alignment. Based on visual observations, the fills appear to be composed of a wide variety of materials ranging from boulder to clay-size particles, and can be expected to vary significantly in both lateral and vertical extent and consistency. Documentation regarding the composition and placement of the majority of fill materials is not available.

During our site reconnaissance on September 9, 2014 we observed fills estimated at 40 feet or greater in depth at two locations on Clairemont Mesa Boulevard in Tierrasanta. One of these locations occurs where the alignment crosses an unnamed tributary of Murphy Canyon northwest of Repecho Drive, and the second location is where Clairemont Mesa Boulevard crosses Shepherd Canyon.

Our site reconnaissance and review of historic topographic maps also confirm that significant earthwork was performed during construction of Tierrasanta Boulevard and adjacent land developments. The earthwork included the partial infilling of several canyons to create the roadway and the placement of fill materials extending in a southeasterly direction between a cul-de-sac at the southeast end of Tierrasanta Boulevard and the northern edge of the San Diego River basin. We estimate that the fill materials may locally exceed 40 feet in depth at several of the canyon crossings on Tierrasanta Boulevard. Where Carlton Oaks Drive crosses Sycamore Canyon in Santee, fill materials were encountered to depths of 14 to 18 feet bgs (AGE,2012). The fill materials generally consisted of silty fine to coarse sands with gravel.

3.3.2 Young Alluvial Deposits

The published geologic map depicts Holocene age young alluvial deposits along the valley floor in Rose, San Clemente, and Murphy Canyon, the San Diego River Valley, and Moreno Valley (Kennedy and Tan, 2005). The alluvium is described as consisting of unconsolidated to locally poorly consolidated sand, silt, clay and gravel, including modern sediments along small drainage channels.

3.3.3 Old Alluvial Deposits

Old Alluvial Deposits of late to middle Pleistocene age (Tan, 2002; Kennedy & Tan, 2005) are mapped along the northern flank of the San Diego River in Santee (Reach D). The deposits consist of moderately well consolidated, poorly sorted and permeable gravel, sand, silt and clay of fluvial origin that is commonly slightly dissected. The Old Alluvial Deposits are also referred to as stream terrace deposits by Kennedy and Peterson (1975).

3.3.4 Very Old Paralic Deposits

Portions of Reach A and Reach B are underlain by Very Old Paralic Deposits of middle to early Pleistocene age (Kennedy and Tan, 2005). These deposits are also referred to as the Lindavista Formation (Kennedy & Peterson, 1975) of early Pleistocene age. The formation consists of interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate with a distinct reddish-brown color due to ferruginous cement. The combination of strong cementation and locally abundant gravels and cobbles pose difficult excavation conditions even for heavy duty construction equipment.

3.3.5 Mission Valley Formation

The Mission Valley Formation conformably overlies the Stadium Conglomerate in portions of Kearny Mesa. This formation consists of marine, lagoonal, and non-marine sandstone. Based on fossil assemblages, the Mission Valley Formation has been assigned an upper Eocene age (Kennedy and Peterson, 1975). The sandstone member is typically light gray, fine to medium grained, and friable. Cobble-conglomerate tongues similar to the underlying Stadium Conglomerate may also be encountered in the formation. There are no surface outcrops of this unit along the project alignment.

3.3.6 Stadium Conglomerate

The Eocene age Stadium Conglomerate consists of a massive cobble-conglomerate with a yellowish brown silty sand matrix that is locally strongly cemented. The clasts are generally of rhyolite, dacite, and quartzite composition, and are typically well rounded, elongated and flattened. The conglomerate is locally interbedded with lenses and layers of sandstone that is similar in composition to the matrix (Kennedy and Peterson, 1975). This unit will be encountered primarily in Reach A and Reach B. The combination of its locally strong cementation and very high gravel and cobble content pose difficult excavation conditions even for heavy duty construction equipment.

3.3.7 Friars Formation

The Friars Formation is a marine and non-marine lagoonal sandstone, siltstone, and claystone deposit that is in conformable contact with the overlying Stadium Conglomerate. The unit has been assigned a middle to late Eocene age based on fossil assemblages (Kennedy and Peterson, 1975). The sandstone is generally described as a massive, yellowish gray, medium grained, poorly indurated, and caliche-rich. The claystone is dark greenish gray, well indurated, expansive, highly plastic and weak, and susceptible to slope stability and landslide hazards.

3.3.8 Scripps Formation

The Scripps Formation is a middle Eocene age sandstone with occasional cobble-conglomerate interbeds. (Kennedy, 1975), and is anticipated to be encountered in Reach A. The unit is in conformable contact with the overlying Friars Formation and is in disconformable contact with the Very Old Paralic Deposits in western portions of the Linda Vista Terrace. The combination of local cobble-conglomerate zones and strong cementation pose difficult excavation conditions even for heavy-duty construction equipment.

3.3.9 Granitic Rock

Granitic rocks assigned to the Western Sequence of the Peninsular Ranges Batholith (Kennedy and Peterson, 1975; Tan, 2002; Todd, 2004) have been mapped in the northeast portion of the proposed project alignment and within Mission Trails Regional Park. Mapped units include tonalite, granodiorite, quartz diorite, monzonite, monzogranite, and minor gabbro. The granitic rocks are all assigned an Early Cretaceous age, and are generally described as light to dark gray, medium to coarse-grained and locally deeply weathered. Portions of Reaches C, D, E, and F are anticipated to encounter granitic rocks.

3.3.10 Santiago Peak Volcanics

Metavolcanic/metasedimentary rocks of Santiago Peak Volcanics have been mapped along portions of the proposed project alignment (Kennedy and Peterson, 1975; Tan, 2002; Todd, 2004). Along the proposed project alignment these rocks may be encountered near the southeast end of Tierrasanta Boulevard, in portions of Mission Trails Regional Park and in the vicinity of San Vicente Dam. The Santiago Peak Volcanics are assigned an early Cretaceous age by V. Todd (2004), a Jurassic/Cretaceous age by Tan (2002), and a Jurassic age by Kennedy and Peterson (1975). These rocks are described as dacitic and andesitic breccia, tuff and flows, with lesser basalt and rhyolite.

3.4 Groundwater

Reach A

The depth of the regional groundwater table beneath the project alignment is unknown but may be assumed to be in excess of 100 feet bgs where it traverses the Linda Vista Terrace. However, localized shallow perched water conditions are known to occur on the mesas, particularly during the wet (rainy) season.

The Geotracker website (www.Geotracker.com)contains a groundwater monitoring report by Geo-Logic Associates (2014) for the West Miramar Landfill. The report states that both a perched (alluvial) and regional (bedrock) aquifer exist at the landfill site, which is situated on the Linda Vista Terrace. Monitoring of eight deep wells from 2002 through 2014 determined that the regional water table varied from approximately 70 to 253 feet bgs (elevations of 161 to 203 feet msl), and that the perched water table varied from approximately 10 to 50 feet bgs (elevations of 222 to305 feet msl) from 1996 to 2014.

Reach B

The west portion of Reach B is located on the Linda Vista Terrace, with the east portion of the reach descending to the San Diego River. The depth of the regional water level beneath the top of the mesa is estimated to be in excess of 100 feet bgs. Shallow to near-surface groundwater is anticipated to exist beneath Murphy and Shepherd Canyons, and in the San Diego River valley.

A letter by SGI Environmental, Inc., (2013) describes a groundwater monitoring program for a Ryder Truck facility located at 5345 Overland Avenue in Kearny Mesa. The letter states that groundwater was measured at depths varying from approximately 18 to 28 feet bgs within twenty monitoring wells at the site in 2013. The letter further states that the Lindavista Formation/Very Old Paralic Deposits extends to an approximate depth of 20 feet bgs and is underlain by the Friars Formation which extended below the bottom of their deepest wells at 70 feet bgs. Groundwater encountered at the Ryder site is likely perched.

ETIC Engineering Inc. (2011) prepared a groundwater monitoring report for an ExxonMobil gasoline station located at 10496 Clairemont Mesa Boulevard (at Santo Road) in Tierrasanta. The report states that groundwater was measured at depths varying from approximately 33to 46 feet bgs in eleven groundwater monitoring wells between 2009 and 2011. The report further states that the site is predominantly underlain by sandstones, siltstones, conglomerates and clays belonging to the Friars Formation, which extends below the bottom of their deepest well of 60 feet bgs. Groundwater encountered at the ExxonMobil site is likely perched.

Reach C

The depth of the regional water table is estimated to be 100 feet or greater in the central portion of Reach C, where Mission Gorge Road crosses an elevated saddle between Cowles and Fortuna Mountains. Shallow groundwater is anticipated near the west and northeast ends of Reach C, which are situated within or near the San Diego River Basin.

H.E.M.C. Environmental Management Corporation (2010) prepared a groundwater monitoring report for a car-wash facility located at 7751 Mission Gorge Road in Santee. The report indicates that groundwater was measured at depths varying from approximately 9 to 14 feet bgs in four monitoring wells at the site on June 28, 2010. The report further states that the site geology consists of fill and alluvial materials above granitic rock materials, and that drilling refusal on the dense granitic rock was encountered at depths ranging from 20 to 26 feet bgs.

Near the Sycamore Creek crossing on Carlton Oaks Drive groundwater was measured at an approximate depth of 8 feet bgs by AGE (2012).

Reach D

Much of Reach D is located within the San Diego River Basin and on gentle slopes overlooking the north side of the valley in the City of Santee. Review of well log data (Bondy and Huntley, 2001), indicates that the depth to groundwater within young alluvial deposits in the Santee portion of the San Diego River valley has historically fluctuated from approximately 10 feet bgs to nearly 50 feet bgs. The report attributes fluctuations in the groundwater depth with seasonal variations in precipitation and variations in groundwater pumping. The report further states that significant groundwater recharge occurs during wet rainfall years, and as a result of periodic spillage from upstream dams at San Vicente, El Capitan and/or Lake Jennings reservoirs. An unpublished consulting report by AGE (2006) indicated that groundwater levels at a site on the north side of the San Diego River Basin in Santee had risen by up to 9 feet between 2002 and 2005 (2004-05 was the third wettest rainy season in recorded history for San Diego County).

Antea Group (2011) prepared a ground water monitoring report for a Circle K store located at 10219 Mast Boulevard in Santee. The report describes groundwater at this location as varying between approximately 7 and 12 feet bgs between 1998 and 2011. The report further states that the groundwater is within alluvial soils that are underlain by weathered granitic rock at 21.5 feet bgs.

Stantec Consultants, Inc. (2013) prepared a ground water monitoring report for a 7-11 store located at 9750 Cuyamaca Street in Santee. The report describes groundwater at this location as varying between approximately 13 and 25 feet bgs in ten monitoring well locations between 1999 and 2012. Stantec also states that the site is underlain with weathered granitic rock materials.

Along Reach D localized perched groundwater at shallow depths can be expected to occur in overburden (fill, weathered rock zone, and alluvial/colluvial) materials above the contact with the underlying basement rocks, particularly during the wet (rainy) season.

Reach E

The majority of this reach is within alluvial soils in Moreno Valley, downstream of San Vicente Reservoir. Review of well log data (Bondy and Huntley, 2001), indicates that the depth to groundwater within the valley has historically fluctuated between approximately 20 feet bgs and nearly 50 feet bgs. Fluctuations in the groundwater depths are largely associated with seasonal variations in precipitation and variations in groundwater pumping. Significant groundwater recharge has occurred during periods of above-average rainfall, and as a result of periodic spillage from San Vicente Reservoir.

Reach F

Reach F will be excavated within granitic and metavolcanic basement rock similar to those encountered near the San Vicente Dam. During excavations performed for the construction of the raised dam URS (2011) reported only minor groundwater seepage along fractures in the rock at rates of less than a few gallons per minute. Similar seepage will likely be encountered within the basement rocks during excavation of Reach F.

3.5 Groundwater Basins

The project study area encompasses the Miramar and Tecolote Hydrologic Areas of the Penasquitos Hydrologic Unit, and the Mission San Diego, Santee and Fernbrook Hydrologic Subareas of the San Diego Hydrologic Unit (San Diego Regional Water Quality Control Board, 1995).

Groundwater in the Miramar Hydrologic Area has no designated beneficial uses along the proposed alignment. Groundwater in the Tecolote Hydrologic Area has been exempted by the Regional Board for the municipal use designation under the terms and conditions of the State Board Resolution No. 88-63, “Sources of Drinking Water” Policy.

Groundwater in the Mission San Diego Hydrologic Subarea has a potential beneficial use for municipal supply, and beneficial agricultural, industrial service and industrial process supply uses.

Groundwater in the Santee Hydrologic Subarea has beneficial municipal, agricultural, industrial service and industrial process supply uses.

Groundwater in the San Vicente Hydrologic Area, which encompasses the Fernbrook Hydrologic Subarea, has beneficial municipal and agricultural uses.

4.0 GEOLOGIC HAZARDS

Geologic hazards are those hazards that could impact a site due to local and regional geologic and seismic conditions. Our evaluation of the various geologic hazards and their potential impact on the project alignment are discussed in the following sections.

4.1 Local Faulting

The pipeline alignment crosses or nearly crosses two faults at three locations along Eastgate Mall. One fault crosses Eastgate Mall at a location immediately north of Miramar Road, and again near the intersection with Eastgate Drive. The other fault is located adjacent to Eastgate Mall at Eastgate Court (Kennedy, 1975; City of San Diego, 1995). Both faults are shown as concealed lineaments and classified as potentially active on the City of San Diego Seismic Safety Study map (1995). Kennedy (1975) identifies one of these concealed features as part of the Torrey Pines fault, a high-angle fault with mostly vertical displacement trending in a general east-west orientation between the coastline and the USMC Air Station Miramar. The second concealed fault appears to be a strand of the Salk Fault, a high-angle fault with mostly vertical displacement similar to the Torrey Pines fault. This fault strand displays a northwest-southeast orientation near Eastgate Mall.

A short fault named the Left Abutment Fault (URS, 2008 and 2011) underlies the left abutment of San Vicente Dam, extending in a general northwest to southeasterly direction with a reported north to northeasterly dip of 55 to 80 degrees. URS describes the fault as a shear zone containing breccia and gouge up to 5-feet wide, with a weathered and altered zone containing closely spaced fractures over a width of up to 15 feet. To the northwest the fault is hidden below San Vicente Reservoir, and to the southeast the fault becomes concealed on a steep hillside in open space.

The nearest mapped major active fault to the proposed pipeline alignment is the Rose Canyon fault zone (RCFZ), located approximately 2.1 miles southwest of the NCAWTP. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend from off the coast near Carlsbad to offshore south of downtown San Diego. Investigations of the RCFZ in the Rose Creek area (Rockwell et al, 1991) and in downtown San Diego (Patterson et al, 1986; Woodward-Clyde Consultants, 1994) found evidence of multiple Holocene earthquakes. Based on these studies, several fault strands within the RCFZ have been classified as active faults, and are included in Alquist-Priolo Special Studies Zones. A summary of the fault parameters is shown in Table 1 on the next page.

Table 1
Summary of Fault Parameters

	Rose Canyon fault zone (San Diego Section)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	6.477 km
Rjb*	6.477 km
Rx*	6.477 km
Fnorm*	0
Frev*	0

	Rose Canyon Fault Zone (Del Mar Section)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	10 km
Top of Rupture Plane	0
Rrup*	6.555 km
Rjb*	6.555 km
Rx*	6.555 km
Fnorm*	0
Frev*	0

Table 1 (Continued)
Summary of Fault Parameters

	Rose Canyon fault zone (Silver Strand section-Spanish Bight fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	15.052 km
Rjb*	15.052 km
Rx*	7.162 km
Fnorm*	0
Frev*	0

* Definition of Terms in Table 1

Rrup	-	Closest distance (km) from NCAWTP to the fault rupture plane.
Rjb	-	Joyner-Boore distance - The shortest horizontal distance from NCAWTP to the surface projection of the rupture area. Think of this as the nearest horizontal distance to the area directly overlying the fault. RJB is zero if the site is located within that area.
Rx	-	Horizontal distance from NCAWTP to the fault trace or surface projection of the top of rupture plane. It is measured perpendicular to the fault (or the fictitious extension of the fault).
Fnorm	-	Fault normal
Frev	-	Fault reverse

The project alignment is subject to moderate to severe ground shaking in response to a major earthquake occurring on the RCFZ or on one of the major regional active faults. The closest active regional faults to the project alignment with recurring magnitude 4.0 and greater earthquakes are the Coronado Bank, the Vallecitos-San Miguel, and the Elsinore fault zones. Other more distant, active regional faults that are considered potential sources of seismic activity include the offshore located San Diego Trough and San Clemente fault zones and some of the faults in Imperial Valley which include the San Jacinto and San Andreas fault zones.

The location of the project alignment in relation to the active faults in the region is shown on the Regional Fault Map (Figure 3). The computer program EQFAULT (Blake, 2000, updated 2004) was used to approximate the distance of known faults to the NCAWTP site. Seven (7) known active faults are identified within a search radius of 50 miles from the NCAWTP site. A summary of seismic source characteristics for faults that present the most significant seismic hazard potential to the pipeline are presented in Table 2 below.

Table 2
Summary of Seismic Source Characteristics

Fault	Maximum Magnitude (Mw)	Peak Site Acceleration (g)	Closest Distance to Site (miles)
Rose Canyon	6.8	0.483	2.1
Coronado Bank	7.4	0.209	7.4
Newport-Inglewood (offshore)	6.9	0.068	33.0
Elsinore - Julian	7.7	0.064	39.7
Elsinore - Temecula	7.7	0.046	44.6
Earthquake Valley	6.5	0.038	44.7
Elsinore - Coyote Mountain	7.7	0.043	48.1

4.2 Historical Seismicity

EQSEARCH is a program that performs automated searches of a catalog of historical Southern California earthquakes. As the program searches the catalog, it computes and prints the epicentral distance from a selected site to each of the earthquakes within a specified radius (100 kilometers). From the computed distance, the program also estimates (using an appropriate attenuation relation) the peak horizontal ground acceleration that may have occurred at the site due to each earthquake. For the purpose of this report, we have performed the earthquake catalog search based on the NCAWTP site which is anticipated to experience the most severe seismic events along the project alignment. A V_{S30} of 400 m/s was estimated for the NCAWTP site.

Based on the estimated shear wave velocities and our visual observations of the on-site geologic units, site Class D attenuation was used for all of our analysis. We used a combined earthquake catalog for magnitude 5.0 or larger events which occurred within 100 kilometers of the project alignment between 1800 and December 1999. The earthquake catalog for events prior to about 1933 is limited to the higher magnitude events.

The search results indicate that the nearest earthquake of magnitude 5.0 occurred on May 25, 1803 about 8.3 miles from the project NCAWTP site on an unmapped fault in the Allied Gardens area of San Diego. The seismic event resulted in a calculated ground acceleration of 0.109g. The largest site acceleration generated from this search is 0.245 g which was the result of a 6.5 magnitude earthquake which occured on November 22, 1800 on a strand of the Rose Canyon Fault Zone (Del Mar Section). The largest magnitude earthquake reported was a magnitude 7.0 event in 1858, located 78.9 miles of the NCAWTP site on a strand of the Fontana Fault in the Riverside area of California which resulted in a calculated ground acceleration of 0.041 g.

It is our opinion that the major seismic hazard affecting the project alignment would be seismic-induced ground shaking. The alignment will likely be subject to moderate to severe ground shaking in response to a local or more distant large magnitude earthquake occurring during the life of the proposed pipeline. For project design purposes, we recommend that the RCFZ be considered as the dominant seismic source.

4.3 Seismic Design Parameters

Due to the length and the variable subsurface conditions along the project alignment, seismic design parameters for the project alignment will vary widely. The parameters presented herein were calculated for the NCAWTP site using the 2010 ASCE 7 - Minimum Design Loads for Buildings and Other Structures procedures which has been adopted for the California Building Codes 2013.

For structural design in accordance with the 2010 ASCE 7 procedures, the United States Geological Survey Design Maps (USGS, 2013) were used to calculate ground motion parameters for the project alignment. The Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion response acceleration is calculated based on the most severe earthquake effects considered by ASCE 7-10 determined for the orientation that resulted in the largest maximum response to the horizontal ground motions and with adjustment to the targeted risk. The Maximum Considered Earthquake Geometric Mean (MCE_G) is determined for the geometric peak ground acceleration and without adjustment for the targeted risk. The MCE_G Peak Ground Acceleration (PGA) adjusted for site effects (PGA_M) should be used for design and evaluation of liquefaction, lateral spreading, seismic settlements, and other soil related issues.

The calculated seismic design parameters are presented in Table 3 on the next page. The design criteria are based on the soil profile type as determined by existing subsurface geologic conditions, on the proximity of the site to a nearby fault and on the maximum moment magnitude and slip rate of the nearby fault. The Design Response Spectrum and Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum are shown on Figures 4 and 5, respectively.

Table 3
Summary of Seismic Design Parameters

REFERENCE	PARAMETER
Table 20.3-1 Site Classification	Site Class = D
Figure 22-1	Ss = 1.056 g
Table 11.4-1 Site Coefficient Fa	Fa = 1.078
Figure 22-2	S_1 = 0.405 g
Table 11.4-2 Site Coefficient Fv	Fv = 1.595
Equation 11.4-1	S_{MS} = 1.138 g
Equation 11.4-2	S_{M1} = 0.646 g
Equation 11.4-3	S_{DS} = 0.759 g
Equation 11.4-5	S_{D1} = 0.431 g
Figure 22-12	T_L = 8 seconds
Figure 22-7	PGA = 0.439 g
Equation 11.8-1	PGA_M = 0.466 g
Figure 22-17	C_{RS} = 0.907
Figure 22-18	C_{R1} = 0.966

Figure 22-1	Ss Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-2	S1Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-12	Mapped Long-Period Transition Period, TL (s), for the Conterminous United States.
Figure 22-7	Maximum Considered Earthquake Geometric Mean (MCEG) PGA, %g, Site Class B for the Conterminous United States.
Figure 22-17	Mapped Risk Coefficient at 0.2 s Spectral Response Period, CRS.
Figure 22-18	Mapped Risk Coefficient at 1.0 s Spectral Response Period, CR1.

Based on the calculated S_{DS} of 0.759 g and S_{D1} of 0.431 g, a Seismic Design Category of “D” may be used for design of facilities with risk categories I, II and III.

4.4 Liquefaction

Seismic-induced soil liquefaction is a phenomenon during which loose, saturated granular materials undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground vibrations induced by earthquakes. Manifestations of soil liquefaction can include loss of bearing capacity below foundations, surface settlements and tilting in level ground, and instabilities in areas of sloping ground. Soil liquefaction can also result in increased lateral and uplift pressures on buried structures.

Along Reaches A and B young alluvial deposits with a low potential for liquefaction underlie the bottoms of Rose Canyon and San Clemente Canyon.

Along Reach C similar materials with a low potential for liquefaction occur at and along the bottoms of Murphy and Shepherd Canyons and an unnamed canyon which is located between the east side of I-15 and Clairemont Mesa Boulevard in Tierrasanta. Young alluvial deposits with a high liquefaction potential will likely be encountered at the San Diego River crossing.

Along Reaches D and E young alluvial deposits with a high liquefaction potential exist within the San Diego River Basin and Moreno Valley. The remainder of formational materials encountered along these two reaches, which include granitic and metavolcanic basement rock, older alluvial/terrace deposits, and Friars Formation, are not considered susceptible to seismic-induced soil liquefaction or ground settlement.

Reach F is underlain by granitic and metavolcanic basement rock units that are not considered susceptible to seismic-induced soil liquefaction or ground settlement.

4.5 Landslides

Several landslides have been mapped along portions of Reach B in the communities of Tierrasanta and Navajo (Kennedy and Peterson, 1975; City of San Diego, 1995; Tan, 1995). All of these landslides occur within the Friars Formation, which is composed of massive to poorly bedded sandstones, claystones, and siltstones. Typically, the claystones are highly plastic and weak, and prone to slope stability and landslide hazards. The mapped landslides are described as follows:

- North of Tierrasanta Boulevard between Rueda Drive and Tambor Road. The mapped slide is classified as “confirmed, known or highly suspected” in the City of San Diego Seismic Safety Study (1995). Residential subdivisions have been built in the area where this slide is mapped. Although information regarding mitigation of the landslide is not available, it may be assumed that potential landslide hazards were properly evaluated and adequately mitigated during the development of the subdivisions.

- Between Tierrasanta Boulevard and Pendiente Court a small slide classified as “possible or conjectured” by the City of San Diego (1995) partially straddles Tierrasanta Boulevard, with an adjacent larger slide to the east classified as “confirmed, known or highly suspected”. An apartment complex has been built on the suspected slide. Although information regarding mitigation of the landslide is not available, it may be assumed that potential landslide hazards were properly evaluated and adequately mitigated during the development of the apartment complex.

Within Reach C several landslides have also been mapped within the Friars Formation at the locations described below:

- The City of San Diego Seismic Safety Study (1995) depicts the presence of two "confirmed, known, or highly suspected" landslides on the southeast side of Mission Gorge Road west of Jackson Drive. The landslides are classified as "questionable" by Tan (1995). The results of a geotechnical investigation performed for design of the Deerfield Pump Station (CWP Geosciences, 1992) indicated that the toe of one or both of these ancient landslides partially encroached beneath the pump station site, which is located on the northwest side of Mission Gorge Road. The landslide extends to the south beneath Mission Gorge Road and an existing residential subdivision known as Knollwood Park that is located across from the pump station site. Considering that mass grading operations performed during construction of the Knollwood Park subdivision included extensive landslide hazard mitigation measures, CWP Geosciences concluded that the potential for renewed movement of the ancient landslide was very low.

- A landslide classified as "questionable" by Tan (1995), straddles Mission Gorge Road near El Banquero Place. This slide is mapped as "confirmed, known, or highly suspected" by the city of San Diego (1995). A residential subdivision has been built at the location of this landslide. Although information regarding mitigation of the landslide is not available, it may be assumed that potential landslide hazards were properly evaluated and adequately mitigated during the development of the subdivision.

No landslides have been mapped along the proposed alignment in Reach D, but several slides have been mapped nearby as follows:

- A total of approximately 17 landslides were mapped by Geocon, Inc (1997) within the proposed Fanita Ranch development located in a hilly area north of Mast Boulevard in the City of Santee. One of the landslides is located within approximately 300 feet of the proposed alignment, northwest of the intersection of Mast Boulevard and Cuyamaca Street. Geocon describes the landslides as shallow to deep-seated, relatively intact block-glide movements with varying degrees of slip plane development and slide mass disturbance. Geocon further reports that the slides occur within the Friars Formation, typically along weak, sheared low angle bedding planes or weak, thinly laminated claystones. The maximum landslide thickness was 44 feet. Geocon also mapped several debris flows within the proposed Fanita Ranch development, with the flows typically occurring on steep slopes and resulting in an accumulation of topsoil, colluvium and debris.

No landslides have been mapped along Reaches E and F. Two landforms which have geomorphic expressions that are typically associated with ancient landslides are located east of San Vicente Reservoir and were investigated by GEI Consultants (2005) for the San Diego County Water Authority. One of the suspected "landslides", referred to as the South Landform by GEI, encompasses a portion of Reach F along the proposed pipeline alignment. Based on geologic mapping, regional fracture analysis, and core borings, GEI concluded that the above-mentioned landforms were in fact not landslides. GEI further reported that their two core borings in the South Landform extended to approximate depths of 400 feet bgs, and that slightly weathered to fresh rock was encountered below depths of 40 to 60 feet.

Along Reach F there is a slight potential hazard of rock falls on steep slopes underlain by bedrock materials.

4.6 Lateral Spread Displacement

The proposed pipeline alignment is not considered susceptible to seismic-induced lateral spreading, considering the absence of nearby active faults and the generally competent nature of geologic units along the proposed pipeline alignment.

4.7 Differential Seismic-Induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with largely homogenous properties if the seismic shaking is uneven due to variable geometry or thickness of the soil deposit. The project alignment is generally underlain by competent soil and bedrock materials that are considered to have a very low potential of differential settlement. There may be a low to moderate potential of differential settlement in areas that are underlain by deep fill and/or young alluvial deposits.

4.8 Ground Lurching

Ground lurching is a permanent displacement or shift of the ground in response to seismic shaking. Ground lurching occurs in areas with high topographic relief, and usually occurs near the source of an earthquake. These displacements can result in permanent cracks in the ground surface. Considering the absence of nearby active faults, it is our opinion that ground lurching does not represent a potential hazard along the proposed pipeline alignment.

4.9 Expansive Soils

Soil materials generated from the siltstone and claystone facies of the Friars Formation are typically moderately to highly expansive. In areas that are underlain by deeply weathered gabbro or rocks of the Santiago Peak Volcanics, the weathering products are typically composed of clay-rich soils which possess low to moderate expansion potential.

4.10 Compressible Soils

Loose and potentially compressible soils are anticipated to occur in areas underlain by undocumented fills and young alluvial deposits.

4.11 Corrosive Soils

Soils derived from the Very Old Paralic Deposits (Lindavista Formation) and the siltstone/claystone facies of the Friars Formation are typically moderately corrosive. Clay rich soils derived from weathering of the Santiago Peak Volcanics and gabbro may also display moderate corrosivity. It is anticipated that the remainder of soil and rock materials encountered along the pipeline alignment possess low to negligible corrosivity characteristics.

4.12 Secondary Hazards

Given the elevation of the proposed pipeline alignment and the absence of nearby large bodies of water, the risk of inundation as a result of seismic-induced seiches or tsunamis is considered negligible. Seasonal flooding from heavy rainfall events may pose a potential hazard in Rose, San Clemente and Murphy Canyons, and within the San Diego River Valley. Within the San Diego River Valley there is also a hazard of flooding from upstream dam releases and/or dam failures.

5.0 CONSTRUCTION CONSIDERATIONS

It is anticipated that excavations in man-made fill, sedimentary deposits, and deeply weathered granitic and metavolcanic bedrock can generally be performed with conventional heavy-duty construction equipment. Locally strong cementation and abundant gravels and cobbles may pose difficult excavation conditions in the Very Old Paralic Deposits, Stadium Conglomerate, and Scripps Formation. Excavations made within slightly weathered to unweathered bedrock materials will likely require blasting.

A summary of the relevant geotechnical criteria which should be considered in the design and construction of the proposed project is presented in Table 4 at the end of this report.

5.1 Miramar Landfill Crossing

Reach A traverses both the USMC Air Station Miramar and the lease area of the Miramar Sanitary Landfill. Within the landfill the proposed alignment will extend along a narrow corridor between the north side of the West Miramar Landfill Phase II and the air station, then continue south in open space between Phases I and II of the West Miramar Landfill(City of San Diego 2014a and b). The landfill site plans indicate that this proposed alignment will be located outside the limits of refuse but that there are numerous buried utilities within this corridor (City of San Diego, 2014c). The plans also indicate that the existing utilities continue north of the landfill along the proposed pipeline alignment through USMC Air Station Miramar. The depths of the existing utilities are not known.

The City of San Diego landfill plans (2014a) further indicate that the proposed pipeline alignment will be in very close proximity to the limits of refuse for the inactive South Miramar Landfill along portions of Copley Drive and Copley Park Place. Considering the close proximity of portions of the project alignment in Reach A to the mapped landfill limits, there is a potential risk of encountering buried trash or methane gas.

5.2 Trenchless Construction

It is our understanding that trenchless excavation methods are being considered at eight locations along the proposed project alignment as described in the following sections of this report.

Reach A

- No trenchless excavations are proposed in this reach.

Reach B

- I-163 near Ronson Road.
  Subsurface information near this location is limited to a test boring performed for the Clairemont Mesa Boulevard overcrossing of I-163 (Caltrans, 1958). The boring did not encounter groundwater at the time of the investigation in 1956. The boring was extended to only 14 feet bgs (393 feet msl elevation) on 12/4/56. Soils encountered in the boring are described as very dense reddish brown cobble-conglomerate with lenses of reddish orange medium to coarse sandstone. Soil borings performed for the widening and retrofitting of the westbound overcrossing of I-163 by Clairemont Mesa Boulevard (Caltrans, 2007) encountered soil materials identified as Lindavista Formation/Very Old Paralic Deposits consisting of very dense light to moderate brown silty sand with gravel extending to the bottom of their deepest boring at an elevation of 403 feet msl. No groundwater was encountered during their field investigation which was performed on June 19, 2003.

  Based on the available subsurface information, it is anticipated that the trenchless crossing will extend through either the Very Old Paralic Deposits or the underlying Friars Formation. It is our opinion that Auger boring may be the appropriate trenchless method at this location.

- I-15 between Clairemont Mesa Boulevard and Balboa Avenue .
  Test borings performed for the Clairemont Mesa Boulevard overcrossing of I-15 (Caltrans, 1986) indicate that static groundwater was encountered at an elevation of 246.5 feet msl (26 feet bgs) in their boring B-3 on March 1, 1982. The soils encountered in their borings are generally described as dense to very dense, uncemented gravelly sandstone with cobbles, underlain with very dense gravelly and cobbly slightly cemented sandstone with clayey and silty binder. The descriptions are consistent with young alluvial deposits and the cobble-rich facies of the Friars Formation. The four borings by Caltrans extended to depths of 40 to 70 feet bgs, with boring B-3 extended to the deepest elevation of 232 feet msl.

  Considering the history of this site where a substantial amount of fill has been placed, it is recommended a site-specific subsurface investigation be performed in order to select an appropriate trenchless method.

- San Diego River between Tierrasanta Boulevard and Princess View Drive.
  It is anticipated that a deep crossing at this location will most likely encounter metavolcanic rocks of the Santiago Peak Volcanics. In this case, either drill-and-blast or Horizontal Directional Drilling (HDD) trenchless methods may be appropriate for this location.

Reach C

- Along Mission Gorge Road, beginning west of Golfcrest Drive and continuing northeast to the approximate Santee city limits.
  This location is underlain by granitic rock with lesser amounts of metavolcanic rock. The subsurface condition is anticipated to be similar to that at the San Diego River crossing location, and either drill-and-blast or HDD trenchless methods may be appropriate for this location.

Reach D

Two trenchless excavation sites are proposed along this reach, as follows:

- San Diego River at SR 52, along West Hills Parkway.
  The nearest available subsurface information consists of test borings performed for the SR 52 bridge over the San Diego River (Caltrans, 1994). The boring logs indicate static groundwater at approximate elevations of 285 to 293 feet msl (5 to 11 feet bgs) on March 11, 1991. The boring logs further indicate that the young alluvial deposits extend to depths varying between approximately 10 and 30 feet bgs, which is underlain by granitic bedrock materials. The granitic rock is generally described as slightly weathered to fresh quartz diorite that is closely fractured to massive.

  This crossing is also underlain by shallow young alluvial deposits overlying granitic bedrock. Shallow groundwater likely. The subsurface condition is anticipated to be similar to that at the San Diego River crossing location, either drill-and-blast or HDD trenchless methods may be appropriate for this location.

- Carlton Oaks Drive at Sycamore Canyon.
  The nearest available subsurface information include test borings performed by AGE (2012) which encountered artificial fill to depths of 14 to 18 feet bgs, respectively, below the roadway near the west and east bridge abutments. The fill materials were found to be underlain by siltstone, sandstone and claystone facies of the Friars Formation, which extended below the bottoms of the borings at 51 feet bgs. Groundwater was encountered at a depth of 8 feet bgs. In addition, Kleinfelder (1991) reportedly encountered alluvial materials to depths up to 10 feet bgs at the Mast Boulevard bridge crossing over Santee Lake No 2. The alluvial deposits in Kleinfelder's borings were found to be underlain by weathered granitic rock materials that extended below the bottom of their deepest boring at 30 feet bgs.

At this location, the trenchless excavation is anticipated to encounter the Friars Formation, and be below the water table. In this case, microtunneling may be the appropriate trenchless method for this location.

Reach E

- Highway 67 near Willow Road and San Vicente Creek.
  The nearest available subsurface information consists of test borings performed for the Willow Road Bridge at San Vicente Creek (County of San Diego, 1983). The County's borings encountered alluvial materials underlain by granitic bedrock consisting of granodiorite and tonalite at the bridge site. The alluvial deposits are described as coarse to fine sands, silty sands, sandy gravels and cobbles, and silty clayey sand. The boring logs indicate that the granitic bedrock slopes downward from west to east at an approximately 4:1 (horizontal:vertical) inclination, from 375feet msl (26 feet bgs) near the west bridge abutment to 328 feet msl (79 feet bgs) near the east bridge abutment. Static groundwater was reportedly encountered at an elevation of 391 feet msl (8 feet bgs) in their boring B-1 on October 2, 1980.

  This crossing may encounter either young alluvial deposits or granitic rock depending on the depth. HDD may be the appropriate trenchless method for this location.

Reach F

- San Vicente Dam to San Vicente Reservoir Discharge Structure.
  The nearest available subsurface information can be obtained from the work performed by URS Corporation for the design of the San Vicente Dam Raise Project (2008). URS performed over 110 UCS tests on selected rock core borings near the dam raise site. UCS tests performed within the metavolcanic rock varied from 4,320 to 48, 450 psi, and UCS tests performed within the granitic rock (granodiorite) varied from 6,000 to 51,767 psi. Generalized borehole information provided by URS indicates that the bedrock materials are predominantly moderately weathered to fresh, with some localized zones of intensely weathered rock. URS also reported that only minor localized seepage was encountered during excavation for the San Vicente Dam raise.

  This site is underlain by metavolcanic and granitic bedrock, and either drill-and-blast or HDD would be the appropriate trenchless method.

A summary of the relevant geotechnical criteria which should be considered in the design and construction of the proposed project is presented in Table 4.

6.0 LIMITATIONS

The information presented in this report is intended for the sole use of Brown and Caldwell and the City of San Diego in their planning and design of the subject project. Our firm did not perform an investigation to evaluate the subsurface conditions along the project alignment. This report is based on a review and evaluation of readily available information, various assumptions to bridge over data gaps, and our previous experience in the general project study area.

This study was performed in accordance with the authorized scope of work for this project. The findings and professional opinions presented in this report were developed in general conformance with the current practices and standard of care exercised by local geotechnical engineering consultants performing similar tasks at the present time. No other warranty, either expressed or implied, is made with regard to the findings and professional opinions presented in this report.

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TABLE 4

SUMMARY OF GEOTECHNICAL DESIGN CRITERIA

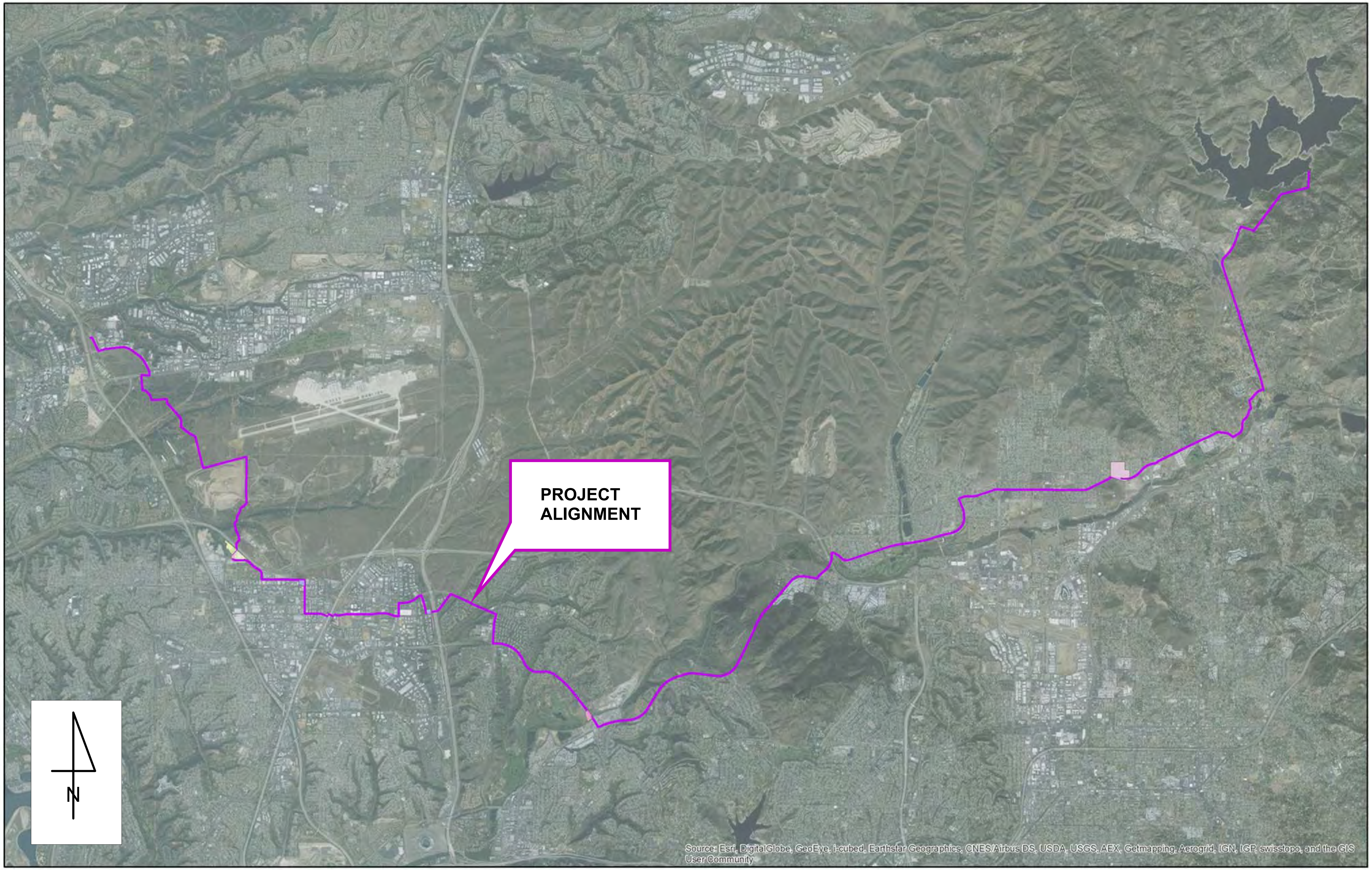
Reach	A	B	C	D	E	F
Approximate Ground Surface Elevation (feet, msl)	270'-390'	95'-425'	110'-695'	290'-450'	415'-450'	450'-1,200'
Subsurface Materials	Fill materials, Very Old Paralic Deposits, Stadium Conglomerate, Friars Formation, Scripps Formation, local young alluvial deposits.	Deep fill materials in certain areas, Very Old Paralic Deposits, Stadium Conglomerate, Friars Formation, Mission Valley Formation, young alluvial deposits, and metavolcanic bedrock	Deep fill materials in certain areas, Very Old Paralic Deposits, Friars Formation, granitic and metavolcanic bedrock, local young alluvial deposits	Young alluvial deposits, Old alluvial deposits, Friars Formation, granitic and metavolcanic bedrock	Young alluvial deposits, granitic and metavolcanic bedrock	Granitic and metavolcanic bedrock
Approximate Depth to groundwater (feet bgs)	100'+	0' to 100'+	10' to 100'+	Less than 10' to 50'	20' to 50'	100'+
Suspected Fault Crossing	Three crossings of the projected eastward extensions of the potentially active Salk/Torrey Pines faults	None	None	None	None	Possible crossing of potentially active Left Abutment fault near San Vicente Dam
Liquefaction Susceptibility	Low potential within young alluvial deposits in Rose/San Clemente Canyons	Low potential within young alluvial deposits in Murphy Canyon and tributaries, high potential at San Diego River crossing	High potential in young alluvial deposits in San Diego River Basin	High potential in young alluvial deposits in San Diego River Basin	High potential in young alluvial deposits in Moreno Valley	None
Expansive Soil	Yes (localized)	Yes (within Friars Formation)	Yes (within Friars Formation)	Yes (within Friars Formation)	None (based on available data)	None (based on available data)
Compressible Soil	Localized to fill and young alluvial deposits	Localized to fill and young alluvial deposits	Localized to fill and young alluvial deposits	Localized to fill and young alluvial deposits	Localized to fill and young alluvial deposits	Absent based on available data
Mapped Landslides	None	Yes	Yes	The project alignment does not cross any known/mapped landslides. However, several landslides were mapped within 300 feet of the project alignment.	None	None
Construction Considerations	Reach A can be installed using conventional cut & cover methods. The majority of the soil materials are considered suitable for use as compacted trench backfill materials, with the exception of soil materials generated by excavations made in Very Old Paralic Deposits and Stadium Conglomerate that are likely to contain abundant gravels and cobbles.	The majority of Reach B will have similar conditions as Reach A. A small portion of Reach B may encounter metavolcanic bedrock. Excavations made in slightly to unweathered metavolcanic bedrock will likely require blasting. Soil generated from excavations and/or blasting in the metavolcanic bedrock will require screening of oversized material prior to use as trench backfill.	The western half of Reach C is very similar to Reach B. The eastern half is primarily underlain by granitic bedrock which has similar excavation characteristics as metavolcanic bedrock.	The majority of Reach D will extend through alluvial deposits and Friars Formation which can generally be excavated using conventional equipment. The eastern end of Reach D may encounter granitic bedrock which may require blasting, and screening if materials generated from excavation/blasting are to be used as backfill.	The majority of Reach E will extend through alluvial deposits which can be excavated using conventional equipment. The eastern and western ends of Reach E may encounter granitic bedrock which may require blasting, and screening if materials generated from excavation/blasting are to be used as backfill.	The entire Reach F will be underlain by granitic bedrock which may require blasting, and screening if materials generated from excavation/blasting are to be used as backfill.

TABLE 4

SUMMARY OF GEOTECHNICAL DESIGN CRITERIA
(Continued)

Reach	A	B	C	D	E	F
Trenchless Construction	No trenchless crossings planned in this reach.	• The I-163 crossing near Ronson Road can likely be excavated using a horizontal auger boring. • Additional geotechnical investigations should be performed prior to selection of the appropriate trenchless crossing method for I-15 south of Clairemont Mesa Boulevard. • The San Diego River crossing near Princess View Drive can likely be performed by either drill & blast or HDD methods.	• The trenchless excavation below Mission Gorge Road through Mission Trails Regional Park can likely be performed by either drill & blast or HDD methods.	• The San Diego River crossing near Princess View Drive may be performed by either drill & blast or HDD methods.	• The Highway 67 crossing near Willow Road and San Vicente Creek may be performed by HDD method.	• Either drill & blast or HDD are considered appropriate trenchless methods along Segment F.
Other Unusual Conditions With Cost Consequences	Numerous buried utilities along proposed pipeline corridor in USMC Air Station Miramar and within Miramar Landfill. Due to the proximity of the project alignment to the mapped limits of the landfill, there is a potential risk of encountering buried trash and/or methane gas intrusion.	None anticipated except where contaminated soil and/or groundwater are encountered	None anticipated except where contaminated soil and/or groundwater are encountered	None anticipated except where contaminated soil and/or groundwater are encountered	None anticipated except where contaminated soil and/or groundwater are encountered. It appears that the proposed project alignment traverses the historic townsite of Foster near San Vicente Dam (EDAW, 2009).	None anticipated except where contaminated soil and/or groundwater are encountered.

FIGURES



Source: Esri, DigitalGlobe, GeoEye, i-cubed, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

ALIGNMENT MAP
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE

ALLIED GEOTECHNICAL ENGINEERS, INC.

FIGURE 1

PROJECT NO.
142 GTS-12

EXPLANATION

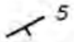
Qya	Young alluvial flood plain deposits (Holocene & late Pleistocene)
Qoa	Old alluvial flood plain deposits (Late to middle Pleistocene)
Qvop	Very old paralic deposits (Middle to early Pleistocene)
Tmv	Mission Valley Formation (Middle Eocene)
Tst	Stadium Conglomerate (Middle Eocene)
Tf	Friars Formation (Middle Eocene)
Tsc	Scripps Formation (Middle Eocene)
Kgu	Granodiorite & Tonalite undivided (Mid-Cretaceous)
Kt	Tonalite undivided (Mid-Cretaceous)
Kgr	Granitoid Rocks (Early Cretaceous)
Ksp	Santiago Peak Volcanics (Early Cretaceous)
Mzu	Metasedimentary & Metavolcanic Rocks undivided (Mesozoic)

CONTACT

(dashed where approximately located;
dotted where concealed)

FAULT

(dashed where approximately located;
dotted where concealed; U, upthrown side;
D, downthrown side)



Strike and dip of beddings
in sedimentary rocks.

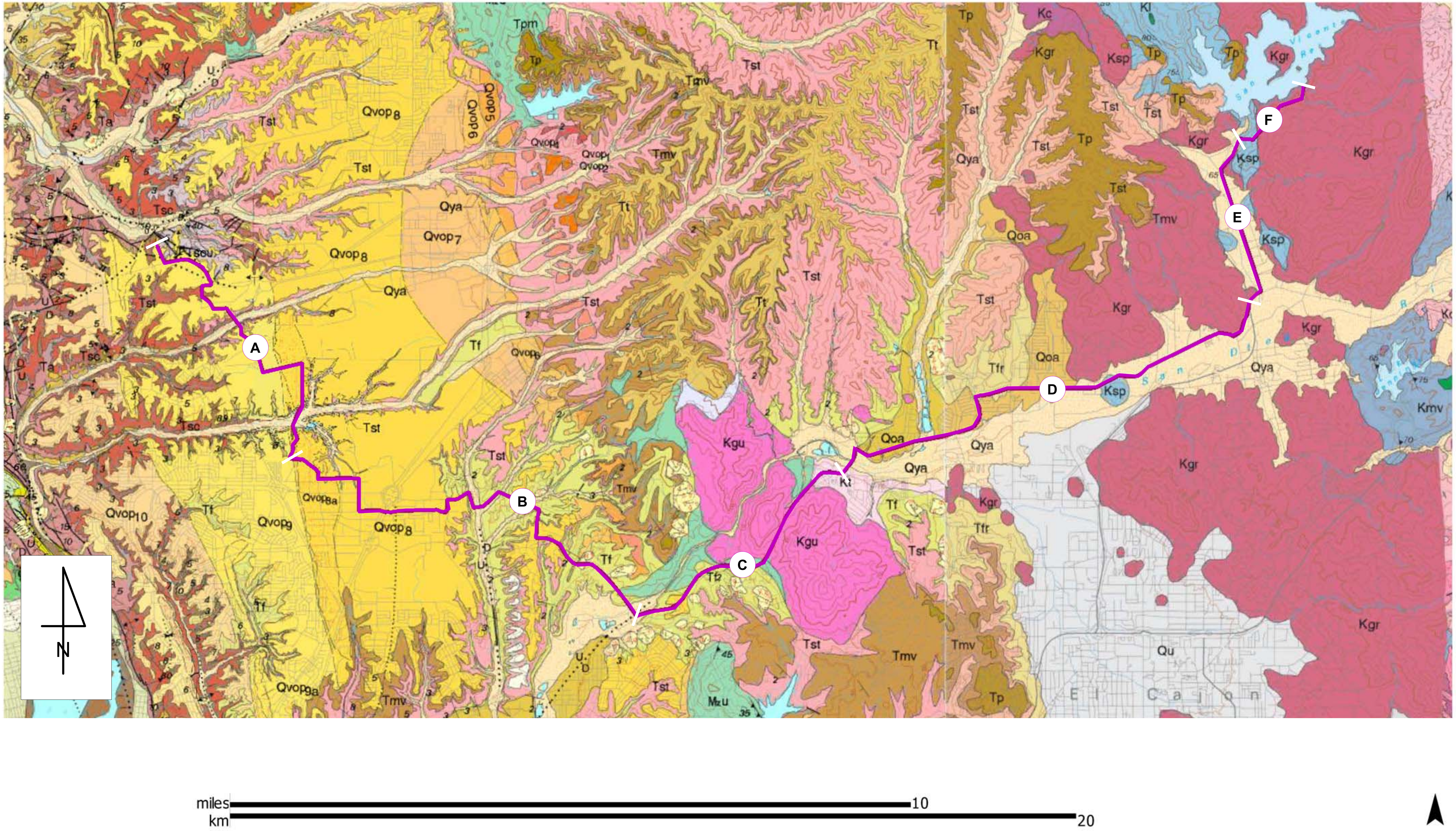
Strike and dip of beddings
in metasedimentary rocks.

Landslide with directional movement
indicated by arrows.

Approximate North City to San Vicente Reservoir
Pipeline Alignment
("Reach" designation is in circle)

GEOLOGIC MAP - LEGEND
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE

PROJECT NO. 142 GS-12	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 2-1



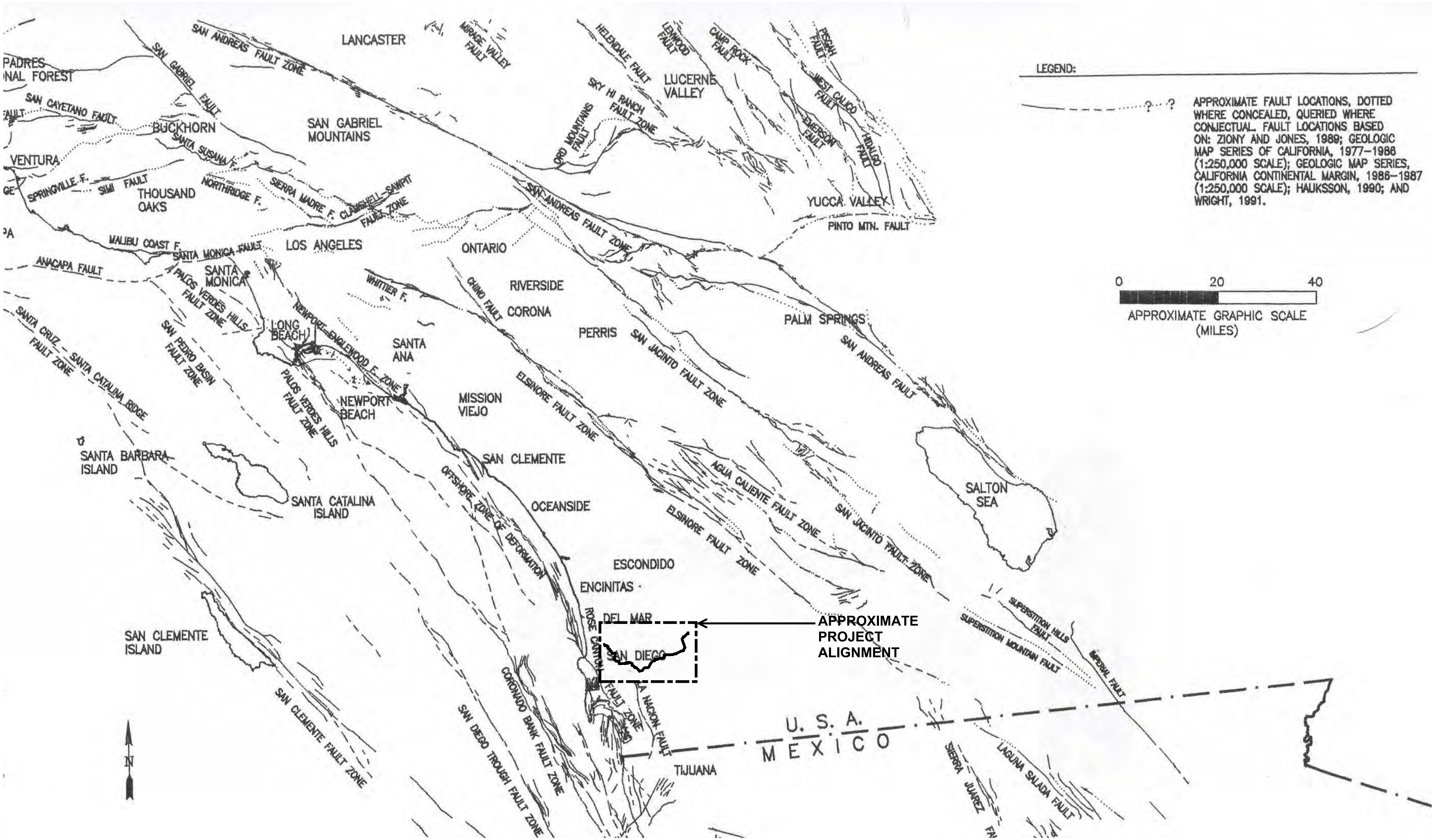
Source: USGS Geologic Map of the San Diego of the 30' x 60' Quadrangle, California compiled by Michael P. Kennedy and Siang S. Tan (2005) and Preliminary Geologic Map of the El Cajon 30' x 60' Quadrangle, California compiled by V. Todd (2004).

GENERALIZED GEOLOGIC MAP
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE

ALLIED GEOTECHNICAL ENGINEERS, iNC.

FIGURE 2-2

**PROJECT NO.
142 GTS-12**



LEGEND:

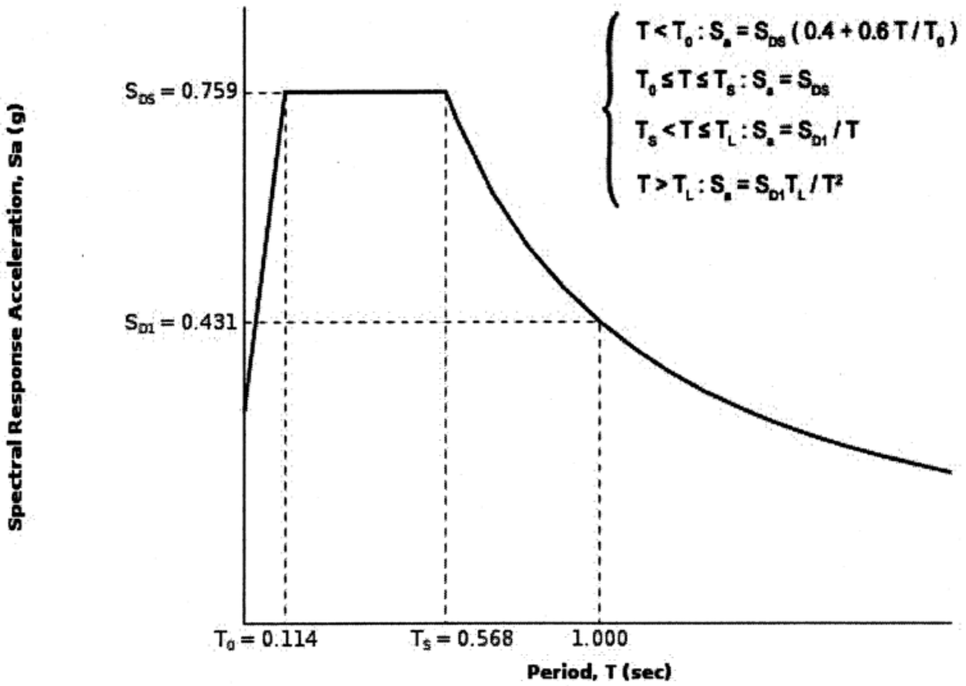
APPROXIMATE FAULT LOCATIONS, DOTTED WHERE CONCEALED, QUERIED WHERE CONJECTUAL. FAULT LOCATIONS BASED ON: ZIONY AND JONES, 1989; GEOLOGIC MAP SERIES OF CALIFORNIA, 1977-1986 (1:250,000 SCALE); GEOLOGIC MAP SERIES, CALIFORNIA CONTINENTAL MARGIN, 1986-1987 (1:250,000 SCALE); HAUKSSON, 1990; AND WRIGHT, 1991.

REGIONAL FAULT MAP
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE

ALLIED GEOTECHNICAL ENGINEERS, INC.

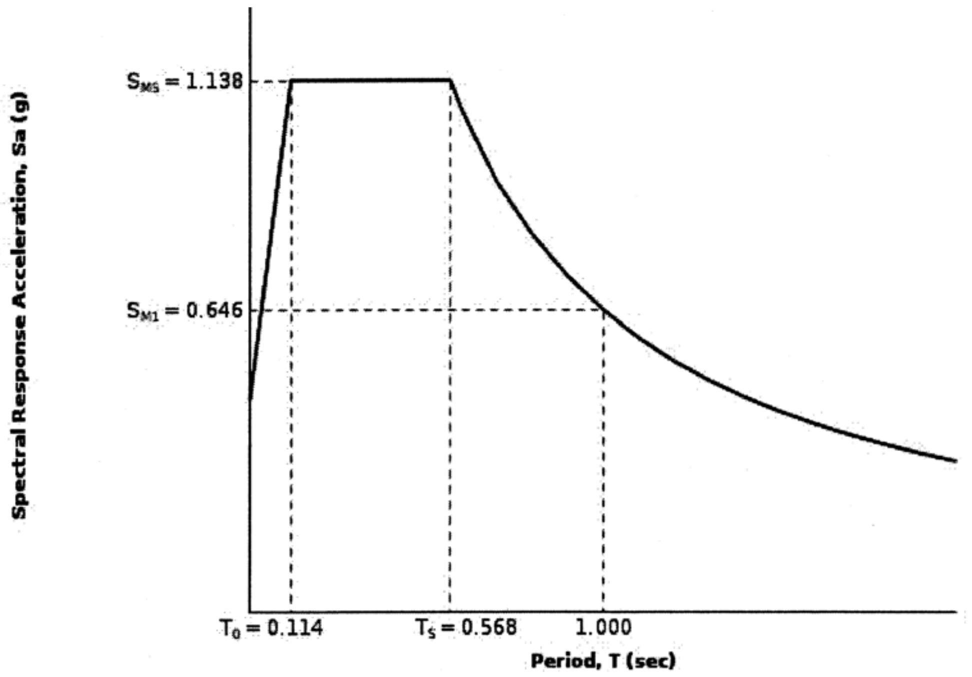
FIGURE 3

PROJECT NO.
142 GTS-12A



DESIGN RESPONSE SPECTRUM
CAMPO ROAD SEWER REPLACEMENT PROJECT

PROJECT NO. 142 GTS-12A	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 4



RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R)
NORTH CITY TO SAN VICENTE RESERVOIR PIPELINE

PROJECT NO. 142 GTS-12A	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 5

APPENDIX D5

PRELIMINARY
GEOTECHNICAL INVESTIGATION

CITY OF SAN DIEGO PURE WATER PROGRAM
TASK ORDER 02

PREDESIGN - NORTH CITY PLANT UPGRADES

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
TASK 2.3

SAN DIEGO, CALIFORNIA

Prepared for:

MWH/Brown and Caldwell
Task Order T10507369-103097-OM

(K2 Engineering Job No. G2015007-2.3)

June 2, 2016

K2 ENGINEERING, INC.
GEOTECHNICAL CONSULTANTS

June 2, 2016

MWH/Brown and Caldwell
Pure Water Team
9665 Chesapeake Drive, Suite 201
San Diego, California 92123

Transmitted via e-mail: LSkutecki@BrwnCald.com
MJayakumar@BrwnCald.com

Subject: **Preliminary Geotechnical Investigation**
Predesign – North City Plant Upgrades
Proposed North City Advanced Water Purification Facility
San Diego, California
K2 Engineering Job No. G2015007-2.3

We are pleased to present our "Report, Preliminary Geotechnical Investigation, Predesign – North City Plant Upgrades, Proposed Advanced Water Purification Facility, San Diego, California". This report was prepared in accordance with the Scope of Work outlined on Task Order 2: Predesign of North City Upgrades. This investigation was undertaken as Task Order T10507369-103097-OM.

We previously presented the results of a desktop investigation in a report dated September 25, 2015. The purpose of this report is to present the results of the field explorations, laboratory tests and geologic-seismic evaluation performed as part of the 30% design phase of the project. Preliminary recommendations for earthwork and for design of foundations are also included in this report.

Please call us if you have any questions or if we can be of further service to you on this or future projects. It has been a pleasure working with you.

Respectfully submitted,

K2 ENGINEERING, INC.

Susana C. Kemmerer

Susana Kemmerer, RGE 2287
President
K2 Engineering, Inc.

Susana Kemmerer
No. 2287
Exp. 6/30/17

Vincent W. Gaby

Vincent Gaby, CEG 1755
Engineering Geologist

Vincent W. Gaby
No. 1755
Exp. 7/31/17

K2\REPORTS\MWH-BC\Pure Water Program\Task 02\G2015007-2.3 NCAWPF 6-2-16.doc

ENGINEERING, INC.
GEOTECHNICAL CONSULTANTS

REPORT

PRELIMINARY GEOTECHNICAL INVESTIGATION

CITY OF SAN DIEGO PURE WATER PROGRAM
TASK ORDER 02

PREDESIGN – NORTH CITY PLANT UPGRADES

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
Task 2.3

SAN DIEGO, CALIFORNIA

(K2 Engineering Job No. G2015007-2.3)

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1.0 SUMMARY

This report presents the results of a geotechnical investigation performed to provide preliminary earthwork and foundation design recommendations for the proposed North City Advanced Water Purification Facility (NCAWPF). The NCAWPF site is located on the northeast corner of the intersection of Eastgate Mall and Interstate 805 in San Diego, California. The approximate location of the NCAWPF site is presented in Plate 1, Vicinity Map. The approximate locations of the exploratory borings and percolation tests are presented on Plates 2 and 3. A summary of our findings and recommendations is presented below.

- Fill soils were not encountered at the locations drilled but may be present at other locations. Poorly to moderately cemented formational materials consisting of stiff to hard siltstone and dense to very dense sandstone of the Scripps Formation, were encountered to the maximum depth drilled of 31½ feet. Naturally occurring overburden soils, composed of medium stiff, silty clay up to 3 feet in thickness capped the Scripps Formation. The overburden and formational materials have a medium to high expansion potential (Expansion Index of 85 to 113).

- Groundwater was not encountered at the boring locations. Seepage between lithologic units may occur during periods of heavy rainfall or due to irrigation.

- No active or potentially active faults are known to cross the site. Accordingly, the risk of surface rupture due to faulting at the site is considered low. The site could be subject to severe ground shaking in the event of a major earthquake, but this hazard is common to Southern California, and the effects of the shaking can be reduced if the structures are designed and constructed in accordance with current engineering practice and building codes.

- A strand of the Torrey Pines Fault has been mapped as crossing the site in a northwest to southeast direction. Sheared bedding within the formational units, as well as strata with significant variations in bedding orientation, which may indicate the presence of faulting, was observed in the exposed west-facing slope. The Torrey Pines fault has been previously classified as an inactive fault. Field explorations to determine the location and activity of the fault, as well as the composition of subsurface materials are recommended.

- The potential for deep seated landslides is considered low. However the exposed slopes west of the site or future slopes are susceptible to significant erosion and surficial failure. The potential for other geologic hazards is considered low.

- The on-site soils have a medium to high expansion potential. As such, the on-site soils are not considered suitable for use as retaining wall backfill or for support of foundations, floor slabs or pavements without some improvement. Replacement with non-expansive imported soils or lime stabilization is recommended.

- For buildings supported on grade, a minimum of 4 feet of non-expansive compacted fill or lime-stabilized soil is recommended beneath the floor slabs and 2 feet beneath footings.

- Structures extending below grade such as the Process Building may be supported on undisturbed formational materials. Foundations may consist of spread footings or mat foundations.

- Drilled cast-in-place piles may be used to resist uplift forces.

- Remedial grading or alternative foundations may be required if zones of weakness are encountered within, or in the vicinity of, the fault zone.

- Heavy duty earth moving equipment will be required to complete the excavations. Difficult digging may occur in the strongly cemented layers.

- The on-site soils may be used as compacted fill providing oversize material, debris or organic matter are removed. These soils may be placed in deeper fills at least 4 feet (vertically) beneath finish grade and not less than 15 feet (horizontally to the back of any wall or face of slope.

- The on-site soils have a very severe corrosion potential to metal loss, low sulfate ion concentrations and severe chloride ions concentrations.

- o0o -

2.0 SCOPE

This report presents the results of a preliminary geotechnical investigation performed to provide planning and design criteria for the proposed NCAWPF in San Diego, California. This investigation was completed in conjunction with the 30% design phase of the NCAWPF to aid in the design and cost estimate of the project. Additional field explorations, laboratory testing and engineering analysis will be required prior to final design.

The purpose of this investigation was to determine the static physical characteristics of the on-site soils; and to provide preliminary geotechnical recommendations for foundation design, grading, excavation and backfill for the proposed facilities. More specifically, the scope of the investigation included the following:

- Evaluation of the existing surface and subsurface conditions, including groundwater conditions (if encountered), within the areas of proposed construction.

- Performance of field explorations: Drilling and logging of three (3) borings to a maximum depth of 31½ feet and completing three (3) percolation tests. The approximate locations of the exploratory excavations are presented in Plate 2, Site Plan.

- Laboratory tests to estimate the physical properties of the onsite materials.

- Developing preliminary recommendations for earthwork.

- Providing preliminary foundation design recommendations and associated design parameters.

- Presenting general recommendations concerning construction procedures and quality control measures relating to earthwork.

Our recommendations are based on the results of our field explorations, laboratory tests and associated geotechnical analyses. The results of our field explorations and laboratory tests are presented in Appendix A. This investigation did not include studies to assess the environmental hazards that may affect the site however, this does not imply that such hazards affect the site.

Our professional services have been performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report. This report has been prepared for the MWH/Brown and Caldwell Pure Water Team and their design consultants to be used solely in the evaluation and preliminary design of the subject project. This report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses

3.0 PROJECT DESCRIPTION

The NCAWPF will be located on the northeast corner of Eastgate Mall and Interstate 805, north of the existing North County Water Reclamation Plant (NCWRP). The proposed North City Plant upgrades are part of the City of San Diego Pure Water Program. The program requires that the NCAWPF provides treatment to 30 million gallons per day (mgd) of tertiary effluent flow originating at the NCWRP. The purified influent is then to be pumped to the San Vicente Reservoir. The influent is to be pumped from the NCWRP and delivered to the NCAWPF through a pipeline extending under Eastgate Mall. The Influent Pump Station and associated pipelines are part of Task 5.2. The approximate location of the NCAWPF site is presented in Plate 1, Vicinity Map.

Based on the information provided, current plans include the following facilities.

- Operations and Maintenance Building
- Process Building
- NCAWPF Pump Station
- Lime Facility
- Chemical Storage Area
- RO Feed Tanks
- Product Water Tank
- CO_2 Facility
- Electrical Buildings
- Paved parking and driveway areas.

According to preliminary grading plans, significant earthwork is proposed for this project. Final grade elevations will vary from about 371 feet above mean sea level (m.s.l.) at the south entrance from Eastgate Mall to 378 feet msl on the north side of the site. Graded slopes up to 18 feet in height are proposed along the east side of the project and up to 12 feet at the northwest site boundary. Current plans indicate that parking is to be constructed north and south of the Operations Building and north of the pump station. A retaining wall up to 16 feet in height is planned along the north site boundary.

We understand that the Process Building and the Pump Station will extend 18 to 20 feet below grade and that the other facilities may be established at grade. The location of the proposed facilities and proposed grades are presented in Plate 2, Site Plan.

4.0 SITE CONDITIONS

4.1 EXISTING CONDITIONS

The NCAWPF site is located north of Eastgate Mall and east of the I-805 Freeway. It is bound to the east by the SDGE right-of-way and power lines and to the north by the SDGE substation. Site topography is generally flat to gently sloping. Beyond the project limits, the ground surface slopes down from the site to the north, east and west. The property is undeveloped and covered with native chaparral and wild grasses. Unpaved access roads occur along the east, west and south sides of the site. The Pueblo North Vernal Pools are reportedly located within the site along the western access road and the southwest corner of the site.

4.2 GEOLOGIC SETTING

The NCAWPF site is located within the coastal plain portion of the Peninsular Ranges geomorphic province. The general structural trend of the province is northerly to northwesterly. The coastal plain is approximately 5 to 10 miles wide, consisting of sedimentary units which are part of the San Diego Embayment (Kennedy, 1975). The deposits associated with the Eocene-age Scripps Formation are the primary units underlying the site. Locally, in the central and southwestern portions of the site, the Scripps Formation is capped by the Pleistocene-age Lindavista Formation, also known as Very Old Paralic Deposits (Kennedy and Tan, 2008). At depth, the Scripps Formation is inferred to rest on a basement complex consisting of Cretaceous-age metavolcanic rocks of the Santiago Peak Formation.

Looking East at NCAWPF Site

4.3 SUBSURFACE CONDITIONS

4.3.1 Geologic Materials

The site is underlain by silty sandstone, siltstone and claystone that been mapped as belonging to the Eocene-age Scripps Formation (Kennedy and Tan, 2008). The Plio-Pleistocene age Lindavista Formation was noted as occurring within the central and southwestern sections of the site. The Lindavista Formation was not exposed at the boring locations. At the boring locations, the Scripps Formation was mantled by colluvium. Each of these units is described below from oldest to youngest. The local site geology is presented on Plate 3, Local Geology. The site is shown in relation to local geologic features on Plate 4, Geologic Map.

Scripps Formation (T_{sc}) – The Scripps Formation is described by Kennedy and Moore (1971) as pale yellowish-brown medium grained sandstone, with minor interbedded layers of cobble conglomerate. It is exposed in the slope which descends to Interstate 805 adjacent to the west project boundary. At this location it occurs as pale yellow, silty, fine grained sandstone. It contains interbeds of thinly laminated, fissile, very fine grained sandstone and sandy siltstone. It is moderately well indurated, thinly bedded and generally dips very gently (5 degrees) to the south. A change in the bedding orientation was noted in the slope northwest of the site. Vertical joints trending towards the south-southwest are common in the slope. Numerous small head-scarps 1 to 3 feet in vertical height suggest that the Scripps Formation at this location is prone to shallow surficial slope failure and erosion.

At the boring locations, the Scripps Formation consisted of stiff to hard siltstone interbedded with dense to very dense silty sandstone. Individual bedding appeared to be very thin and nearly horizontal to very gently dipping. These materials are poorly to moderately cemented. Testing of a composite sample indicated that these materials may be classified as having a medium expansion potential (Expansion Index 85).

Lindavista Formation (Q_{ln}) – Recent studies by Kennedy and Tan (2008) have identified this unit as **Very Old Paralic Deposits (Q_{vop-8})**. However, for continuity with the previously completed geotechnical reports we will continue to refer to them as the Lindavista Formation. This formation is characterized by Kennedy (1975) as consisting predominantly of reddish-brown interbedded conglomerate and sandstone. The reddish color distinguishes this formation from other sedimentary units that have a similar appearance (conglomeratic sandstones). Another diagnostic feature of the Lindavista Formation are the presence of small mound-like hills on the surface. These topographic features called "Mima-Mounds" by Kennedy (1975), are on the order of 30 feet in diameter and up to 3 feet in height.

On the project site the Lindavista Formation appears to be limited in both horizontal and vertical extent to the central and southwest portions of the site. Materials of the Lindavista Formation were not encountered in the borings drilled for this investigation but were reportedly encountered during prior investigations. Good exposures of this unit are not present. The contact between the Lindavista Formation and underlying Scripps Formation is approximateled based on the abundance of conglomerate, and changes in vegetation. The Lindavista Formation seems to support woody chaparral while the Scripps Formation, especially the disturbed or weathered sections, is primarily covered with wild grasses.

Colluvium – For the purposes of this study the term "Colluvium" was used to describe topsoil and gravity deposited slopewash, as well as in-situ developed soil. Colluvium was encountered in each of the exploratory borings. It consisted of soft to medium stiff, silty clay and was up to 3 feet in thickness. The laboratory test results indicated that these soils may be classified as highly plastic with a high expansion potential.

4.3.2 Groundwater

Groundwater was not encountered within any of the exploratory borings. According to the documents reviewed, it is estimated that groundwater occurs at a depth of approximately 100 feet within the general site vicinity. However, groundwater conditions could develop and/or seepage may occur depending on annual precipitation and irrigation. Seepage may occur along lithologic changes within the on-site soils and at the interface between the fill and the less permeable formational materials.

4.3.3 Percolation Tests

Three percolations tests were performed to evaluate the infiltration characteristics of the near-surface on-site materials. The approximate percolation test locations are presented on Plates 2 and 3, Site Plan and Local Geology.

The percolation tests were performed in general accordance with the guidelines of the San Diego County Public Health Department. The tests results are indicative of the permeability of the on-site soils at their current condition. Percolation rates will be affected by future construction activity such as earthmoving and soil compaction.

The percolation tests consisted of drilling three 8-inch diameter test holes to depths of 3 to 5 feet. After completion of drilling, the holes were cleaned and a minimum of 12 inches of clean water was carefully poured into the percolation test holes and allowed to presoak overnight. After the presoaking period (over 12 hours) the water level in P-1 and P-2 remained unchanged, which indicated no percolation. Very minimal absorption was detected in P-3.

Percolation testing was performed on P-3. The loose materials were removed and about 2 inches of pea gravel was added to the bottom of the hole. Clean water was added to the hole and the variations in the water level were measured at approximate 30 minute intervals. Readings in P-3 indicated no change in the water level. The percolation test results indicate very slow percolation rates (less than 0.06 inches per hour). The results of the tests are presented in Table 1, Percolation Test Results.

Table 1, Percolation Test Results

Test Number	Percolation Rate (min/inch)	Permeability (in/hr)	Rate of Flow(1)
P-1	Did not percolate	<0.06	Very Slow
P-2	Did not percolate	<0.06	Very Slow
P-3	Did not percolate	<0.06	Very Slow

(1)*Based on USDA Soil Survey Glossary*

5.0 GEOLOGIC EVALUATION

5.1 GEOLOGIC HAZARDS

5.1.1 General

Geologic hazards that could impact the subject site are essentially limited to those derived from earthquakes. The major cause of damage from earthquakes is violent shaking from earthquake waves. Damage due to actual displacement or fault movement beneath the site is less frequent. The violent shaking would occur not only immediately adjacent to the earthquake epicenter, but in areas for many miles in all directions.

The west facing slope contains vertical joints which makes it prone to erosion and shallow slope failure.

5.1.2 Faults

The numerous faults in Southern California include active, potentially active, and inactive faults. The definitions of fault activity terms used here are based on those developed for the Alquist-Priolo Special Studies Zone Act of 1972.

Active faults are those faults that have had surface displacement within Holocene time (approximately the last 11,000 years) and/or have been included within an Alquist-Priolo Special Studies Zone. Faults are considered potentially active if they show evidence of surface displacement since the beginning of Quaternary time (about two million years ago), but not since Holocene time. Inactive faults are those which have not had surface movement since the beginning of Quaternary time.

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). Based on a review of geologic literature, no active faults are known to occur beneath the study area.

According to the City of San Diego Seismic Study Map No. 34, a strand of the Torrey Pines Fault has been inferred as crossing the site in a northeast-southwest direction. It is classified in this document as “inactive, potentially active, presumed inactive or activity unknown”. Furthermore, an evaluation of faulting in the site vicinity by Ziony (1973), documents displacement along the Torrey Pines Fault of Eocene stratigraphy but not within the Quaternary Lindavista Formation. The Torrey Pines Fault was mapped starting at a point about 700 feet north of the southwest site corner and at about 400 feet north of the southeast corner. During our site reconnaissance of September 1, 2015, sheared and fractured sediments of the Scripps Formation were observed in the west facing slope above the I-805. This shear zone, located approximately 500 feet north of Eastgate Mall may represent the Torrey Pines Fault. Materials with significant variations in strike and dip were observed during our site reconnaissance on the west-facing slope in the vicinity of the area previously mapped as landslide debris (GTC, 1990a). These features, located about 1,000 feet north of Eastgate Mall, may be indicative of faulting rather than slope failure. This fault is not to be considered active therefore it appears that there is a low probability of surface rupture due to faulting beneath the site.

There are, however, several faults located in sufficiently close proximity that movement associated with them could cause significant ground motion at the site.

Nearby faults include the Rose Canyon fault zone which lies approximately 4 miles to the west, the La Nacion fault zone located about 4.8 miles southeast, and the Coronado Bank fault zone located approximately 16.5 miles to the west (offshore). Evidence suggesting movement along the Rose Canyon fault zone during the Holocene has been presented by Moore and Kennedy (1975). The State of California has zoned portions of the Rose Canyon fault zone as active under the Alquist-Priolo Senate Bill. This has come about as a result of faulted paleosols in Rose Canyon that are considered to be unquestionably of Holocene age (T. Rockwell, 1989). In addition, work performed by several consultants prior to and during construction of the Police Administration and Technical Center in downtown San Diego have indicated displacement of Holocene soil units (dated between 5,000 and 10,000 years before present) by what they have concluded to be a continuation of the Rose Canyon fault zone (Schlemon et al, 1989).

The La Nacion fault zone is considered to be potentially active. This is based on the observed displacement of Pleistocene sediments (Foster, 1973). Furthermore, the offset of Holocene age alluvium has been suggested by Artim and Pinckney (1973). However, findings reported by others (Kuper 1989, Elliot 1989) have presented conclusions suggesting that activity along the La Nacion fault zone ceased prior to Holocene time.

The Coronado Bank fault zone is a complex series of left-and-right stepping en-echelon faults. Marine geophysical studies performed by Kennedy and others (1980) have provided evidence that Holocene sediments have been offset by several faults associated with the Coronado Bank fault zone. Therefore this fault system should also be considered active.

The impact of regional fault zones must also be considered. The closest of these to the site are the Newport-Inglewood and the Elsinore fault zones located approximately 23 miles to the northwest and 34 miles northeast, respectively. These faults are considered active.

The table below, Seismicity of Major Faults, presents the maximum considered earthquake magnitudes (MCE) and estimated peak accelerations (PGA) anticipated at the site. These accelerations are based on the assumption that the earthquake occurs on specific faults at the closest point on that particular fault to the site. The table below was developed using the program EQFAULT (Blake, 2000). The site accelerations were estimated using the relationships developed by Boore (Boore et. al., 1993a). Different PGA values may be required for design.

The maximum considered earthquake (MCE) is defined as the maximum earthquake that appears to be reasonably capable of occurring under the conditions of the presently known geologic framework. The probability of such an earthquake occurring during the lifetime of this project is considered low. The severity of ground motion is not anticipated to be significantly greater at this location than in other areas in San Diego County.

Table 2 - Seismicity of Major Faults
Deterministic Site Parameters

FAULT NAME	Approximate Distance - miles -	Maximum Earthquake Magnitude	Maximum Site Acceleration - g's -	Est. Site Intensity Mod. Mercalli
ROSE CANYON	3.7	7.2	0.376	IX
LA NACION	4.8	6.5	0.290	IX
CORONADO BANK	16.5	7.6	0.188	VIII
NEWPORT-INGLEWOOD (Offshore)	23.2	7.1	0.112	VII
ELSINORE-JULIAN	33.8	7.1	0.084	VII
ELSINORE-TEMECULA	35.9	6.8	0.068	VI
EARTHQUAKE VALLEY	41.4	6.5	0.052	VI
ELSINORE-COYOTE MOUNTAIN	48.9	6.8	0.054	VI

5.1.3 Liquefaction

The potential for seismically induced liquefaction is greatest where shallow ground water and poorly consolidated, well sorted, fine grained sands and silts are present. Liquefaction potential decreases with increasing density, grain size, and clay and gravel content, but increases as the ground acceleration and duration of seismic shaking increases.

The project site is underlain by stiff to hard siltstone and dense to very dense sandstone. Groundwater or water seepage was not observed in our exploratory borings, nor was it reported within the exploratory borings drilled by others. Based on the density of the underlying materials and the absence of shallow groundwater, the potential for generalized liquefaction in the event of a strong to moderate earthquake on a nearby fault is considered remote.

5.1.4 Landslides and Slope Stability

The site is located on generally flat to gently sloping terrain which is encompassed within areas that have been identified as having minimal to moderate risk (City of San Diego Seismic Study, 2008) for slope failure. Within the general vicinity of the site, there are mapped landslides in the southwest facing slopes northeast of the site (Kennedy and Tan, 2008). Evidence of such landsliding was observed in the 1966 aerial photograph. Indications of erosion and surficial slumping were also noted on the exposed west-facing slope adjacent to I-805 during our site reconnaissance.

I-805 Slope Looking South

An area of surface creep was reportedly observed by Geotechnical Consultants, Inc. (1990a) on the northwest section of the site. Based on the review of available information and the bedding observed at the site, it appears that this feature may be associated with faulting rather than landsliding.

The sedimentary units exposed within the project area appeared to be very gently diping (5 degrees) to the south. No adverse structures, jointing, fracture planes or bedding dipping out of slope were exposed in our exploratory borings, observed in the outcrop exposures or reported by others. However, we did observe vertical joints in the slope west of the site that where oriented subparallel to the strike of the slope face. These features create the potential for shallow slumping and erosion on exposed surfaces. The potential for deep seated slope stability problems at the site is considered low.

The on-site sediments and overburden soils may also have an impact on temporary excavations which may be prone to shallow slope failure. Measures to prevent runoff into the excavations are recommended during construction.

5.1.5 Seismic Settlement and Differential Settlement

Seismic settlement occurs when loose to medium dense granular soils densify during ground shaking. Such seismically induced settlement can occur in both dry and partially saturated granular soils, as well as in saturated granular soils. Due to lithologic variations, such settlement can occur differentially across a site. Differential settlement may also be induced by ground failures, such as liquefaction, flow slides, and surface ruptures. The materials beneath the site consist of stiff to hard siltstone and dense to very dense sandstone, mantled by a thin layer of poorly consolidated clay soils. Significant seismically induced settlement is not expected to occur within the sedimentary bedrock or the overburden soils.

5.1.6 Subsidence

The proposed site is not in an area of known ground subsidence and is not underlain by an area of oil or gas extraction (Alfors, et at., 1973). The potential for such subsidence is therefore considered very low.

5.1.7 Flooding, Tsunamis, and Seiches

The potential for significant inundation of the study area as a result of storm flooding, tsunamis, reservoir containment failure, or seiches (oscillations in a confined body of water due to strong ground shaking) is considered remote.

The site is located on a mesa surface that drains to the Soledad Canyon to the north and it does not appear to be located within a flood-prone area. Furthermore, available published hazard maps do not include the site within the flood zone area.

The study area is not located in a coastal zone and, therefore, does not appear to be subject to tsunami hazards. Lastly, the study area is not located downslope from any large bodies of stored water that could pose an inundation hazard in the event of earthquake-induced failure of storage facilities or seiches.

5.2 SEISMIC DESIGN PARAMETERS

The proposed NCAWPF will be located at latitude N32.882° and longitude W117.199°. The closest potentially active or active fault is the Rose Canyon Fault Zone located about 4 miles west from the site. In accordance with the 2013 California Building Code, the following design parameters may be used for design of the proposed NCAWPF.

S_s = 106% g	Maximum Considered Ground Motion for 0.2 second Spectral Response Acceleration, 5% of Critical Damping, Site Class B
S_1 = 41% g	Maximum Considered Ground Motion for 1.0 second Spectral Response Acceleration, 5% of Critical Damping, Site Class B

The following spectral acceleration factors may be used to develop the response spectra for the deterministic Maximum Considered Earthquake.

$F_a = 1.0$

$F_v = 1.39$

$S_{ms} = F_a S_s$

$S_{m1} = F_v S_1$

The materials beneath the site consist of stiff to hard siltstone and dense to very dense silty sandstone. Based on our subsurface explorations and the documents reviewed, a very dense soil and soft rock classification “C” may be assigned to the site.

According to the USGS hazard maps the MCE Geometric Mean PGA of 0.44g has been estimated for the site.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 GEOLOGY

Based on the review of available information, no active or potentially active faults are known to exist beneath the project site. Accordingly, the possibility of surface rupture at the study area due to faulting is considered low. The site would be subject to strong ground shaking in the event of an earthquake; however this hazard is common to Southern California, and the effects on the proposed project can be mitigated if the structures are designed and constructed in accordance with current engineering practice and building codes.

A strand of the Torrey Pines Fault has been mapped as crossing the site in a northwest to southeast direction. This fault has been previously classified as an inactive fault. Sheared bedding within the formational units as well as strata with significant variations in bedding orientation, which may indicate the presence of faulting, was observed in the exposed west-facing slope. Information regarding the Torrey Pines Fault and what the subsurface conditions may be along the fault zone is limited. As such, we recommend that future studies include trenching to determine the location and age of faulting; as well as the subsurface conditions as these features are typically associated or may have zones of weakness.

A west facing cut slope for I-805 is present along the western boundary. No adverse bedding was observed in the exposed slope. The potential for deep seated landslides is considered low, but vertical joints may result in the slope being susceptible to erosion and surface slumping. Erosion control may be needed for exposed slopes. The potential for other geologic hazards is considered low.

Based on the results of this investigation and information previously gathered during our desk top investigation, the site is suitable for development of the proposed facilities, provided the design and construction incorporate means to mitigate the potential geologic hazards.

6.2 FOUNDATIONS

Based on the results of the exploratory borings, the NCAWPF site is underlain by weakly to moderately cemented formational materials, consisting of stiff to hard sandstone interbedded with dense to very dense, silty sandstone of the Scripps Formation. At the locations drilled the overburden surficial deposits (Colluvium) are composed of soft to medium stiff clay. Materials associated with the Lindavista Formation (sandstone and cobble conglomerate) were observed within the south-central portion of the site but were not exposed in the borings. Fill soils were not encountered at the locations drilled but may be present at other locations not explored.

The materials encountered at the site include clays of high plasticity and with a high expansion potential. In their present condition, these soils are not considered suitable for use as structural fills. To provide more uniform support and to reduce the potential for damage due to expansion, we recommend that the subgrade soils be stabilized using lime. Recommendations for lime stabilization are presented in Section 6.7 of this report. As an alternative, the soils may be overexcavated and replaced with non-expansive compacted fill. Expansive soils may then be placed in deeper fills at least 4 feet (vertically) beneath finish grade and not less than 15 feet (horizontally) from the back of any wall or face of slope.

We recommend that a minimum of 4 feet of non-expansive compacted fill or lime treated soils be placed below the finish grade or 2 feet below the bottom of the footings, whichever is greater. The proposed facilities may be then supported on shallow spread footings established in the non-expansive fill or lime-stabilized soil.

Structures extending below grade, such as the Process Building, may be supported on the undisturbed formational materials. Foundations for the proposed facilities may consist of spread footings or mat foundations.

All foundations should be supported entirely on the formational materials, lime-stabilized soil or the non-expansive materials. No fill-formational transition zones should be allowed beneath buildings.

Overexcavation should be performed if this condition occurs. Overexcavation should extend to a depth of 2 feet below the bottom of the deepest footing.

We understand significant uplift forces may be generated by the proposed lime storage tanks. If required, drilled cast-in-place piles may be used to resist the uplift forces.

A minimum of 2 feet of non-expansive, compacted, fill soils or lime-treated subgrade are recommended beneath all pavements.

6.3 SPREAD FOOTINGS

6.3.1 Bearing Value

Spread footings at least 2 feet in minimum dimension, and extending at least at least 2 feet below the lowest adjacent final grade, may be designed to impose the net dead plus live load pressures noted in Table 3, Foundation Design Parameters. A one-third increase in the bearing value may be used for wind or seismic loads.

While the actual bearing value of the compacted fill/lime treated soils will depend on the material used and the compaction methods employed, the quoted value will be applicable if the soils are prepared and compacted as recommended in this report. The bearing value of the fill and/or lime treated soils should be confirmed after completion of the grading.

The recommended bearing values are net values. Therefore, the weight of the concrete in the footings may be assumed to be 50 pounds per cubic foot, and the weight of the soil backfill over the footings may be neglected when determining the downward load on the footings. A one-third increase in the bearing values may be used for short term wind or seismic loads.

Table 3 - Foundation Design Parameters

	Undisturbed Formational Materials	**Imported Fill/ Lime Treated Soils**
Bearing Capacity	4,500 psf*/ up to 6,000 psf	2,000 psf
Passive Pressure	300 psf	350 psf
Frictional Capacity	0.30	0.35
Subgrade Modulus	100 pci	150 pci

**500 psf per foot of depth increase may be considered for embedment depths greater than 2 feet*

Remedial grading or alternative foundations may be required if zones of weakness are encountered within, or in the vicinity of, the fault zone. Additional field explorations including test pits to determine the fault location and subsurface conditions are recommended.

6.3.2 Settlement

Based on prior experience with similar structures and subsurface conditions, settlements of the proposed facilities if supported as recommended in this report are anticipated to be within acceptable limits. The estimated settlement of the structures should be evaluated as part of the final design.

6.3.3 Lateral Loads

Lateral loads may be resisted by soil friction and by the passive resistance of the subgrade material as noted in Table 3, Foundation Design Parameters. A one-third increase in the passive value may be used for resistance to wind or seismic loads. The frictional resistance and the passive resistance of the materials may be combined without reduction in determining the total lateral resistance.

6.3.4 Footing Installation and Observation

To verify that satisfactory materials are present at the design elevations, all foundation excavations should be observed by a qualified geotechnical engineer. Foundation excavations should be cleaned of any loosened soil and debris before placing steel or concrete. Any expansive soils, organic matter, loose and/or disturbed natural materials, should be removed prior to placement of any new fill or foundations.

All applicable requirements of the local governing bodies, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met. Inspection of footing excavations may be required by the appropriate reviewing governmental agencies. The contractor should familiarize himself with the inspection requirements of the reviewing agencies.

6.3.5 Backfill

All required backfill around the foundations and within utility trenches should be mechanically compacted in layers not more than 8 inches in loose thickness; flooding should not be permitted. Fills should be compacted to at least 95% of the maximum density obtainable by the latest reapproval of ASTM Designation D1557 method of compaction. The exterior grades should be graded to drain away from the structures in order to reduce ponding of water adjacent to structures.

Compaction of the backfill as recommended in this report will be necessary to reduce settlement of the backfill and consequent settlement of the overlying improvements and buried utilities. Even at 95% compaction (ASTM D1557), some settlement of the backfill should be anticipated. Accordingly, any utilities supported therein should be designed to accept differential settlement.

In order to reduce the amount of backfill required, the foundations may be cut neat and poured against the excavated fill or natural soils.

6.4 DRILLED PILES

6.4.1 Drilled Pile Capacities

The estimated ultimate downward and upward capacities of drilled 12- and 24-inch diameter drilled piles are presented on Plate 5, Drilled Pile Capacities. The vertical capacities of other pile sizes may be assumed to be proportional to the perimeter of the pile. Dead-plus-live load capacities are shown; a one-third increase in these capacities may be used when considering wind or seismic loads. The pile capacities presented are based on the strength of the soils. The compressive and tensile strength of the pile section itself should be checked to verify the structural capacity of the piles.

Piles in groups if required, should be spaced at least 2½ diameters on centers. No reduction in the downward pile capacities due to group action will be needed if they are spaced as previously stated.

6.4.2 Lateral Loads

Lateral loads may be resisted by the piles and the passive resistance of the soils against pile caps.

It may be assumed that the soils adjacent to a 12-inch diameter pile, at least 40 feet long can resist horizontal loads imposed at the top of the pile of up to 20,000 pounds. The lateral resistance of other sizes of piles may be assumed to be proportional to the pile diameter. In calculating the maximum bending moment in the pile, the lateral load imposed at the top of the pile may be multiplied by an assumed moment arm of about 3½ feet. For design, it may be assumed that the maximum bending moment will occur at or near the top of the pile and that the bending moment will decrease to zero at a depth of about 15 feet below the pile cap. The lateral capacity and reduction in the bending moment are based in part on the assumption that any required backfill adjacent to the pile caps and grade beams will be compacted as recommended herein.

The passive resistance of the compacted fill soils against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase in the passive value may be used for wind or seismic loads.

The lateral resistance of the piles and the passive resistance of the soils against pile caps and grade beams may be combined without reduction in determining the total lateral resistance.

6.4.3 Installation

All drilled pile excavations should be observed by a representative of a qualified firm to document that the desired diameter and penetration below pile cap are achieved.

No unusual difficulties are anticipated during drilling and installation of the piles, but some hard excavating should be anticipated when drilling through the cemented layers. Precautions should be taken during the installation to reduce caving and raveling of the shaft walls. Such precautions may include, but may not be limited to, reduction of drilling speed and the use of special techniques.

Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least 8 hours before drilling an adjacent pile. The concrete should be poured as soon as possible after drilling and inspection are completed. Pile excavations should not be left open overnight. During concrete placement, precautions should be taken to prevent the concrete from hitting the shaft walls. The concrete should not be allowed to fall freely more than 5 feet.

6.5 EXCAVATION

The formational materials are dense to very dense and stiff to hard. The borings drilled at the site were advanced using hollow-stem-auger drilling equipment. Refusal was encountered in Boring B-3 at a depth of 18 feet. It is anticipated that conventional, heavy duty, excavation equipment in good working condition could be used for the proposed excavations.

Temporary, unsurcharged, vertical excavations that are less than 5 feet in height, may be excavated without shoring. Where the necessary space is available, temporary, unsurcharged, excavations may be sloped back in lieu of using shoring. Temporary, unsurcharged, excavations may be sloped back at 1:1 (horizontal to vertical). Barricades should be used to prevent vehicles and storage loads within 10 feet of the tops of sloped excavations. The exposed excavations should be observed by a competent geotechnical firm so that modifications of the excavation criteria may be made if necessary. Where space is not available for sloped-back excavations shoring will be required.

All applicable requirements of the local governments, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met. Conventional earth moving/excavation equipment may be used to excavate the on-site materials. Erosion control and drainage devices should be used to prevent water from entering the excavated areas.

6.6 GRADING

6.6.1 General

Based on current plans, cuts and fills up to 7 feet and 14 feet, respectively, may be required to obtain the design surface grades. The proposed grades are presented in Plate 2, Site Plan. Fill slopes are proposed at the northwest, eastern and southeastern edges of the project. These will reach maximum vertical heights of 10 to 12 feet at most locations with a limited area along the east boundary standing up to 18 feet. A cut slope at the western edge of the project will be on the order of 3 to 5 feet.

In addition to the surface elevations, several of the structures may be embedded up to 20 feet below finish grade.

The site is mantled by medium stiff overburden clayey soils up to 3 feet in thickness that are underlain by well consolidated formational materials associated with the Scripps Formation. The overburden soils have a high expansion potential and are not considered suitable for use as structural fill or for support of the proposed facilities. These soils should be either removed and replaced with non-expansive compacted fill soils or stabilized with lime. The depth of replacement or stabilization should extend at least 5 feet beyond the building footprints and 4 feet below the finish grade or 2 feet below the bottom of the footings, whichever is greater.

At least 2 feet of non-expansive compacted fill or lime stabilized subgrade is recommended beneath paved areas and concrete walks and slabs. Recommendations for lime treatment are presented in Section 6.7.

The formational materials are dense to very dense and stiff to hard. The borings drilled at the site were advanced using hollow-stem-auger drilling equipment. Slow drilling and refusal was encountered at a depth of 18 feet in Boring B-3. Borings B-1 and B-2 were drilled to their scheduled depth of 31½ feet. It is anticipated that conventional heavy duty excavation equipment in good working condition could be used for the proposed excavations.

The on-site soils may be used as compacted fill providing oversize material, expansive soils, debris or organic matter are removed. Due to their expansion potential, it is recommended that the on-site soils be be placed in deeper fills at least 4 feet (vertically) beneath finish grade or 2 feet below foundations and not less than 15 feet (horizontally) to the back of any wall or face of slope. To aid in the characterization of the soils to be excavated, an Expansion Index test was performed in a composite sample from Boring B-2. The test results are presented in Plate A-7.

Temporary excavations within the formational materials may be sloped back at 1 to 1. These materials are susceptible to erosion and surficial slumping when exposed. Erosion control measures will be required.

To reduce moisture infiltration beneath the proposed structures and pavement, proper site drainage compatible with existing or proposed storm drain systems should be provided. Finished grades should be sloped to drain away from the structures. We recommend that all planters be waterproofed and provided with drains and low-flow irrigation systems. We also recommend that all roof and structure drains be extended away from structures and constructed to discharge into storm sewers or unto paved surfaces draining off-site.

6.6.2 Subgrade Preparation

After clearing the site, the exposed materials should be carefully observed to verify the complete removal of unsuitable deposits. Prior to placement of any new fills, any existing fills, debris, organic material, expansive clays and soft or loose soils should be removed. Any required fill should be compacted as recommended in this report. After overexcavating as recommended, the exposed soils should be scarified to a minimum depth of 6 inches and compacted to at least 95% of the maximum dry density (ASTM D1557). The moisture content of the on-site soils should be maintained at 3% to 5% above optimum. If granular soils are exposed at the bottom of the excavation, the moisture content should be maintained within 2% of optimum.

6.6.3 Fill Placement and Compaction

Any required fill should be placed in loose lifts not more than 8 inches in thickness. All fill soils should be compacted to 95% of the maximum dry density (ASTM D1557). The moisture content of the imported non-expansive fill soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site soils should be maintained 3% to 5% above optimum.

6.6.4 Material for Fill

The on-site materials, less any debris, organic matter, contaminated soils, and rocks greater than 6 inches in maximum dimension, may be used in the required fills but should not be used within 4 feet of final subgrade level or foundation level without lime stabilization.

Any imported fill should consist of relatively granular soil with an Expansion Index of 20 or less, an angle of internal friction of at least 33° and an R-value of at least 40. The material should contain sufficient fines (binder material) to result in a stable subgrade.

6.6.5 Observation and Testing

The excavation of the overburden materials and the compaction of all required fill should be observed and tested by a qualified geotechnical firm. The geotechnical engineer should also approve any imported fill material for use prior to importing.

The governmental agencies having jurisdiction over the project should be notified before beginning grading so that the necessary grading permits may be obtained and arrangements made for the required inspection(s).

6.7 SUBGRADE STABILIZATION

6.7.1 General

Based on prior local experience, about 5% quicklime by dry weight may be added to the subgrade to improve the load bearing capacity of the on-site clayey soils and to reduce their expansion potential. If lime treatment is to be used, we recommend that the precise percentage of lime as well as the specific treatment requirements be confirmed during final design and prior to the grading operations.

Prior to stabilization, preparation of the subgrade soils as noted in Section 6.6 is recommended. If the upper 2 feet of subgrade are stabilized, depending on the equipment used, two to three lifts of lime-treated soils may be required. Stock piling of the excavated soils will be required. Deeper stabilization depths may be proportionally achieved in several lifts. Stabilization may be achieved by uniformly mixing lime slurry with the excavated soils. The lime slurry should meet the requirements of ASTM C977.

The lime admixture should be evenly spread over the excavated soils. The spreading equipment should be specifically designed for the purpose of spreading lime and should be metered to verify the required distribution. A single application is recommended.

The use of a traveling single-pass mixer is recommended. The mixer should be used to thoroughly blend the lime with the subgrade soils to a maximum lift depth of 12 inches. Sufficient water should be added to maintain the moisture content of the soil mixture at about 3% to 5% above the optimum moisture content of the treated soil. Laboratory tests indicate sulfate concentrations of 270 to 420 parts per million (ppm) in the soils sampled. To reduce the potential for localized heave in areas of higher soluble sulfate concentrations, which may occur at the site, a mellowing period may be required. Water should be added during mellowing to maintain the moisture content. After allowing the lime-soil mixture to cure for a period of at least 48 hours, the mixer should be used to pulverize and remix the stabilized soil. The lime stabilized soil should then be compacted as described in Section 6.7.2.

The previously stockpiled soils, which are to be used for the second and third lifts, should be stabilized and cured as described above.

6.7.2 Compaction of Stabilized Subgrade Soils

All lime-treated soils should be compacted to at least 95% of the maximum dry density obtained using the ASTM D1557 method of compaction. The moisture content of the stabilized soil mixture should be maintained 3% to 5% above of the optimum moisture content during compaction. Compaction equipment used should be capable of achieving the specified compaction throughout each lift of stabilized fill or the 12 inches of the material stabilized in place.

The stabilized subgrade should be allowed to cure for at least 2 days after final compaction prior to placing any asphalt, concrete or base.

6.8 SLOPES AND EROSION CONTROL

Permanent slopes may be constructed at 2:1 (horizontal to vertical) or flatter. Fill slopes up to 18 feet and cut slpes on the order of 5 feet are planned. Fill slopes should be keyed into the dense natural materials. The key should extend through all incompetent soils and be established at least 2 feet into dense competent materials. The key should be at least 2 feet deep at the toe of the slope and fall with a 5% grade toward the interior of the proposed fill areas. The bottom of the key shall have a width of at least 15 feet. A fill slope detail is presented in Plate 6, Fill Slope Key. The Soil Engineer, Engineering Geologist or their representative in the field should inspect all keys.

The on-site materials are poorly cemented and may be susceptible to erosion. Evidence of slumping and rilling was observed in the exposed slope west of the site. To reduce the potential for erosion of the slope faces, permanent erosion control and drainage devices should be provided. Slope erosion, including sloughing, riling and slumping of surface soils may be anticipated if the slopes are left unprotected for a long period of time, especially during the rainy season. Erosion control may include (but may not be limited to): erosion resistant vegetation and/or erosion control geofabrics. Slopes should be planted with appropriate drought-resistant vegetation as recommended by a landscape architect. Slopes should not be over-irrigated.

Drainage devices designed to carry surface water from overlying areas should not be blocked or destroyed, and should be maintained regularly. Water should be prevented from ponding in pad areas, or from overtopping and flowing down graded or natural slopes. At a minimum, concrete drainage swales should be installed at the top of the slopes to prevent surface runoff over the top of the slope and to reduce the erosion on the face and toe of the slope.

Animal burrows can serve to collect normal sheet flow on slopes and cause rapid and destructive erosion, and should be controlled or eliminated.

6.9 RETAINING WALLS

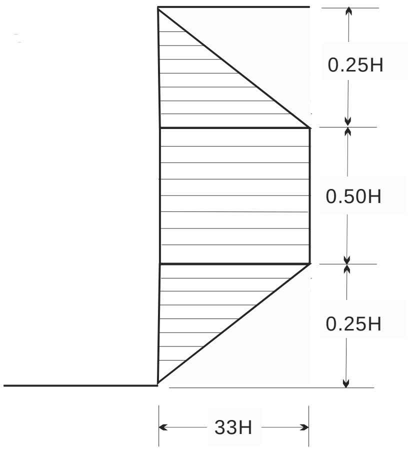
A retaining wall with a maximum height of about 16 feet is planned along the north site boundary. For the design of cantilevered retaining walls or shoring with heights of 15 feet or less, where the backfill consists of non-expansive on site or imported materials, and the backfill is level and well drained, it may be assumed that the soils will exert lateral pressures equal to those indicated on Table 4, Lateral Earth Pressures.

Table 4 – Lateral Earth Pressures

	Design Value			
	Undisturbed Formational Materials		***Imported/Lime Treated Soils**	
Active Pressure	45 pcf	33H	35	25H
At-rest Pressure	65 pcf	47H	50	36H

**Assumed values, should be verified by testing*

For design of tied-back retaining walls, we recommend that a trapezoidal pressure distribution such as that presented below be used. The recommended maximum pressure may be taken as 33H in pounds per square foot, where H is the height of the retaining wall in feet. The recommended pressure is for a level backfill. If the at-rest pressure is used then the maximum pressure should be 47H for the restrained walls. Similar values are presented for the non-expansive imported fills meeting the specifications outlined in Section 6.6. Recommendations for tie-back shoring including the use of soldier piles and anchors may be provided at if required.



Lateral Pressure Distribution

If loads are kept at least 10 feet from the face of the wall, no additional pressure needs to be considered. Otherwise an additional uniform surcharge pressure of 100 pounds per square foot should be used for design.

6.10 FLOOR SLAB SUPPORT

If the soils are prepared as recommended, concrete slabs-on-grade may be supported entirely on the non-expansive compacted fill or stabilized subgrade.

To reduce the potential for water entrapment and to provide protection against vapor or water transmission through the slabs, we recommend that, at a minimum, the slabs-on-grade be underlain by a layer of Caltrans Class 2 permeable material or crushed rock at least 6 inches thick. A suggested gradation for the gravel layer is as follows:

Table 5, Suggested Gravel Gradation

Percent Passing	Sieve Size
¾”	90 – 100
No. 4	0- 10
No. 100	0-3

To provide additional protection against water vapor transmission through the slab in areas where vinyl or other moisture-sensitive floor covering is planned, we recommend that a durable 10-mil-thick impermeable membrane such as Stego Wrap, Perminator or equivalent be installed below the slab. The vapor barrier should be installed in accordance with the manufacturer’s instructions. We recommend that at least a 2-foot lap be provided at the membrane edges or that the edges be sealed.

A low-slump concrete (4-inch maximum slump) should be used to further minimize possible curling of the slabs. The concrete slabs should be allowed to cure properly before placing vinyl or other moisture-sensitive floor covering.

Concrete slab thickness should be provided in accordance with the anticipated use and loadings on the slab and as recommended by the Structural Engineer. As a minimum, slabs-on-grade should be 4 inches in thickness. This recommendation is a minimum only and should be verified by the Structural Engineer. The required thickness and reinforcing of the concrete slabs will depend on the imposed loadings as well as the structural characteristics of the concrete. Construction joint spacing and placement should be provided by the Structural Engineer.

6.11 PAVING

The on-site soils have a high expansion potential and possess low R-values. These soils are not considered suitable for support of pavements. To provide support for paving, the subgrade soils should be prepared as recommended in the previous sections on Grading. Compaction of the subgrade to at least 95%, including trench backfills, will be important for paving support.

The required paving thickness will depend on the subgrade soils, the flexural strength of the concrete and on the Traffic Index applicable to the intended usage. For purposes of pavement design, it was assumed that imported fill soils or the on-site soils stabilized as recommended in this report, will be used as the supporting subgrade. The pavement thickness should be confirmed prior to construction so that any required modifications may be made based on the actual fill materials to be used.

For preliminary pavement design it was assumed that the on-site materials stabilized with about 5% lime will have an R-value of at least 60, and that imported non-expansive soils will have an R-value of 40. Traffic Indexes of 5 and 6 were assumed for design of the proposed pavements.

The paving sections presented in Table 5 are based on assumed Traffic Indexes of 5 and 6 in accordance with the City of San Diego Standard Drawings.

Table 5, Estimated Paving Sections

Traffic Index	Subgrade	Paving Section
5 (Automobile and light truck traffic)	Non-expansive compacted fill	3” AC + 5” CTB or 5” AC 6½” p.c.c.
	Lime stabilized subgrade	3” AC + 5”CTB or 4½” AC 6” p.c.c.
6 (Truck traffic)	Non-expansive compacted fill	3” AC + 5½” CTB or 6½” AC 7” p.c.c.
	Lime stabilized subgrade	3” AC + 5” CTB or 5½” AC 6½” p.c.c.

The cement treated base (CTB) should meet the specifications for CTB as defined in Section 27 of the 2010 State of California, Department of Transportation, Standard Specifications. The base course should be compacted to at least 95%. Careful inspection should be performed to verify that the recommended thickness or greater are achieved and that proper construction procedures are used.

7.0 SOIL CORROSIVITY

Based on the laboratory test results, the on-site soils have low sulfate ion concentrations (270 to 420 parts per million (ppm)) and severe to very severe concentrations of chloride ions (1230 to 1600 ppm), therefore, Type II or V cement is recommended. Concrete should be thoroughly vibrated. The test results are presented on Plate A-9, Corrosion Test Data. The soils exhibit resistivity values of 190 ohm-cm, indicating a very severe potential for metal loss due to electrochemical corrosion processes. Therefore, a minimum concrete cover of 3 inches should be provided over all re-bar, anchor bolts or metallic embeds placed within the foundations and to 18 inches above the ground surface. Reinforcing steel should be protected with a concrete cover of at least 1½ inches for formed surfaces not exposed to weather or not in contact with the ground. If the minimum cover is not achieved corrosion protection of steel members such as epoxy or asphalt coatings may be used. **We recommend that a corrosion engineer be consulted for final corrosion protection recommendations.**

8.0 BASIS FOR RECOMMENDATIONS

The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data collected during the subsurface exploration. We have made our recommendations based on experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in building loads, building locations, or site grades should be provided to us so we may review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based on the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as anticipated. This also provides for the procedure whereby the Client can be advised of unanticipated or changed conditions that would require modifications of our original recommendations. In addition, the presence of our representatives at the site provides the Client with an independent professional opinion regarding the geotechnically related construction procedures. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be reduced to the extent that we are no longer the engineer of record.

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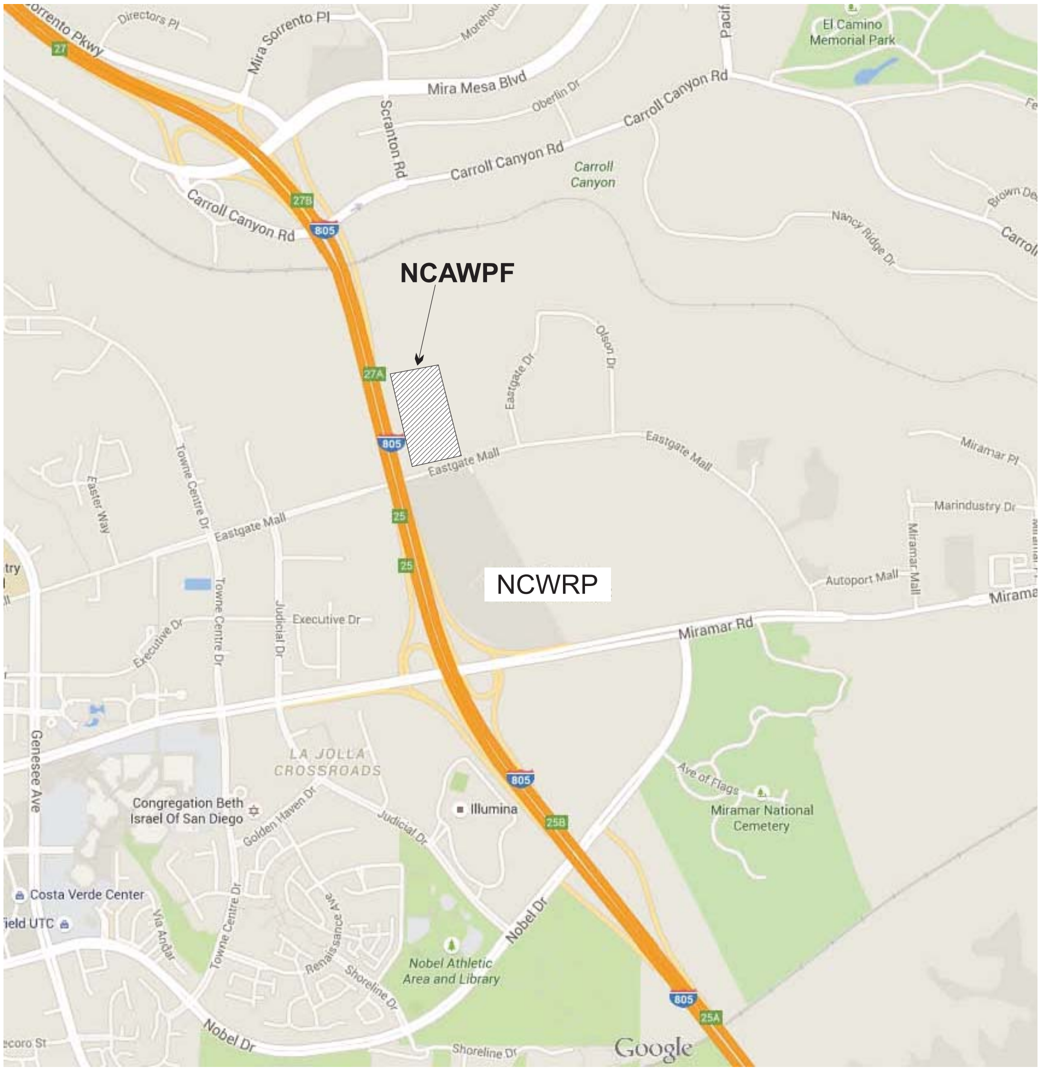
JOB G2015007-2.3 DATE 5/22/2016 BY ____ SK ____ ENGR ____ SCK ____ REV ____



VICINITY MAP

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
SAN DIEGO, CALIFORNIA

▨ **Proposed Facility Approximate Location**

(NOT TO SCALE)

REFERENCE: Google Maps and Imagery (2015)

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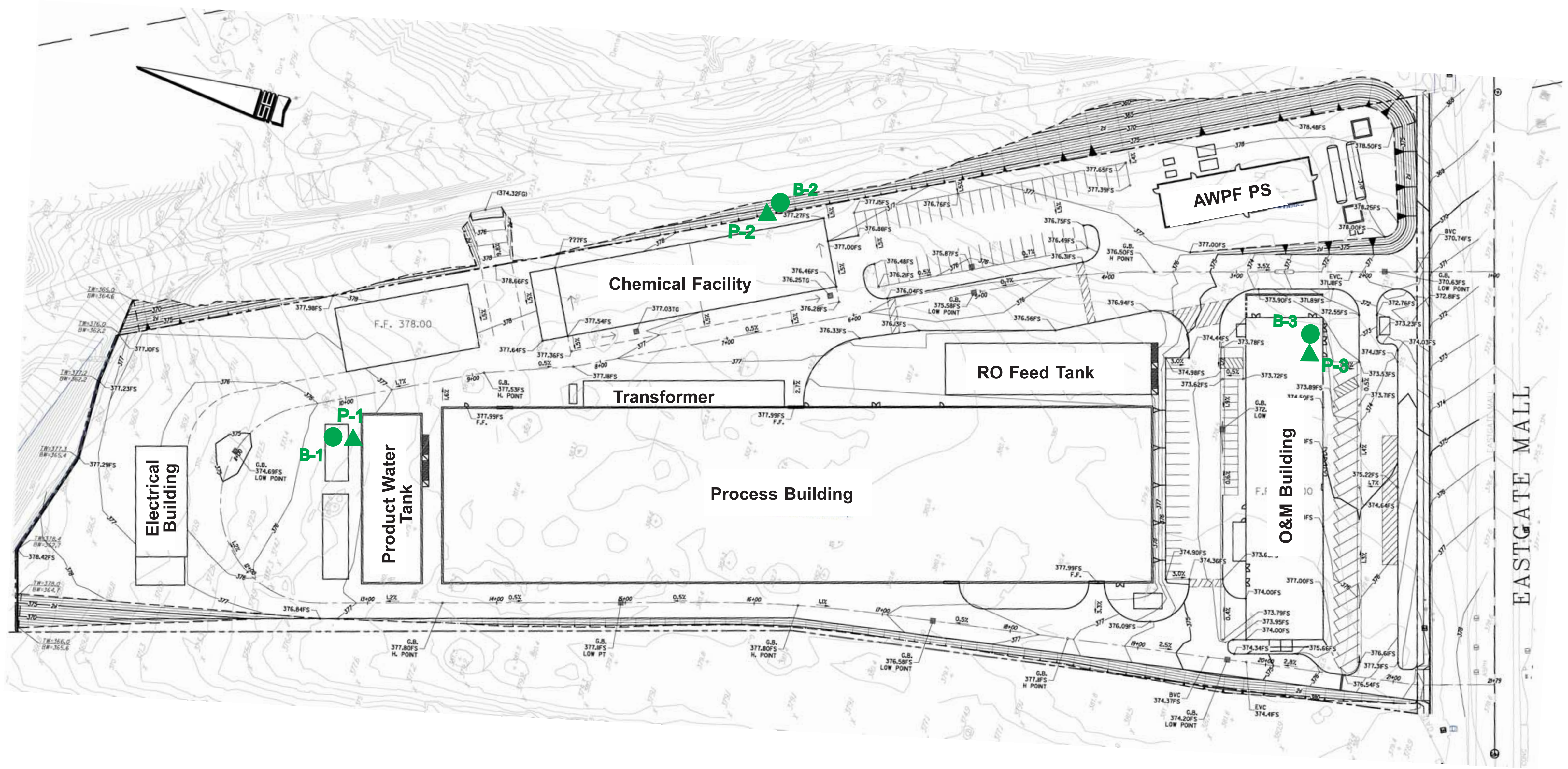
JOB G2015007-2.3 DATE 5/31/2016 BY SCK/VG ENGR ____ REV ____



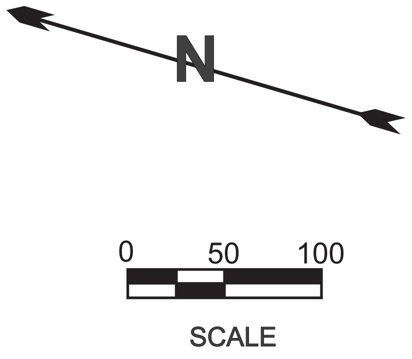
KEY:

B-1 ●	Borings
P1 ▲	Percolation Test

└── Exploratory Number and Approximate Location

SITE PLAN

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
SAN DIEGO, CALIFORNIA



REFERENCE: Preliminary Grading Plan (dated 3/21/16) by Stuart Engineering

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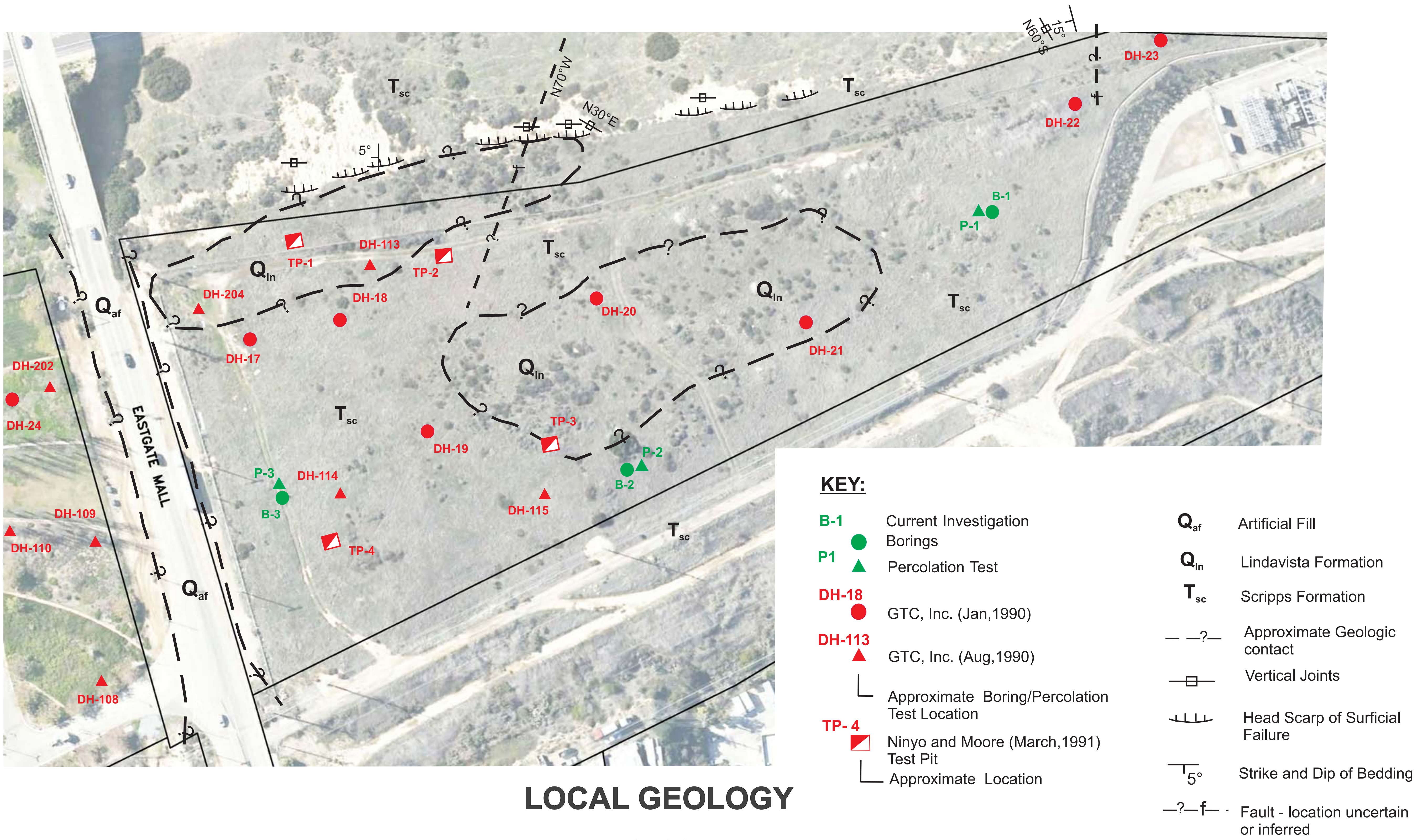
JOB G2015007-2.3 DATE 5/31/2016 BY SCK/VG ENGR REV



LOCAL GEOLOGY

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
SAN DIEGO, CALIFORNIA

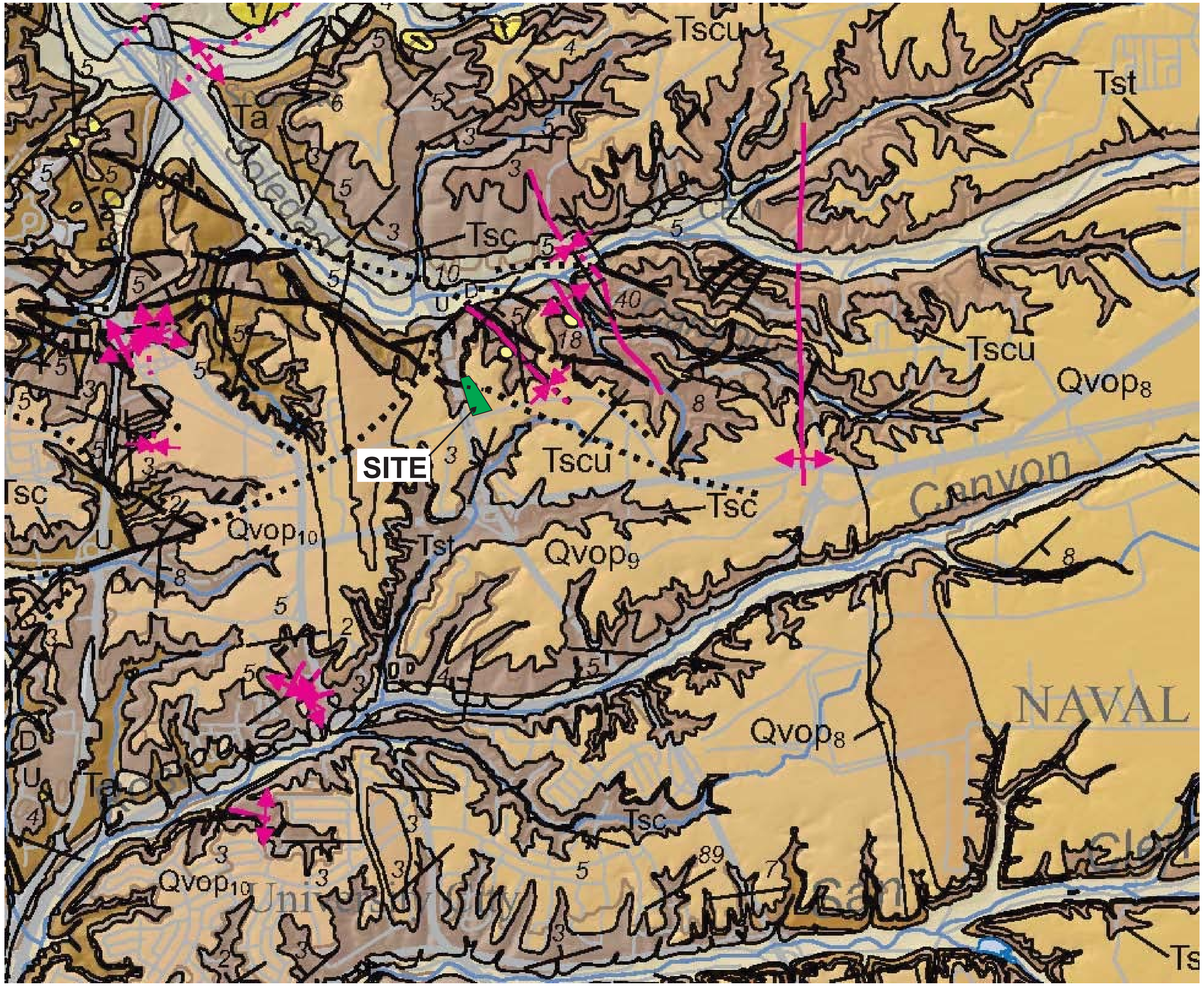
REFERENCE: Aerial Photography (undated) provided by Brown and Caldwell

K2 ENGINEERING, INC.

JOB G2015007-2.3 DATE 5/27/2016 BY SCK/VG ENGR SCK REV

GEOLOGIC MAP

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
SAN DIEGO, CALIFORNIA



N

KEY:

🟨	Q_{ls}	Landslide deposits
◩	$Q_{ln}/Q_{vop\ 8\text{-}10}$	Lindavista Formation/ Very Old Pleistocene Paralic deposits
◩	T_{sc}/T_{scu}	Scripps Formation/ Scripps Formation Upper Unit
U D		Fault - Solid where defined, dashed where inferred
		Anticline - Solid where defined, dashed where inferred
		Sincline - Solid where defined, dashed where inferred
5°		Strike and Dip

REFERENCE: Department of Conservation, California Geological Survey,
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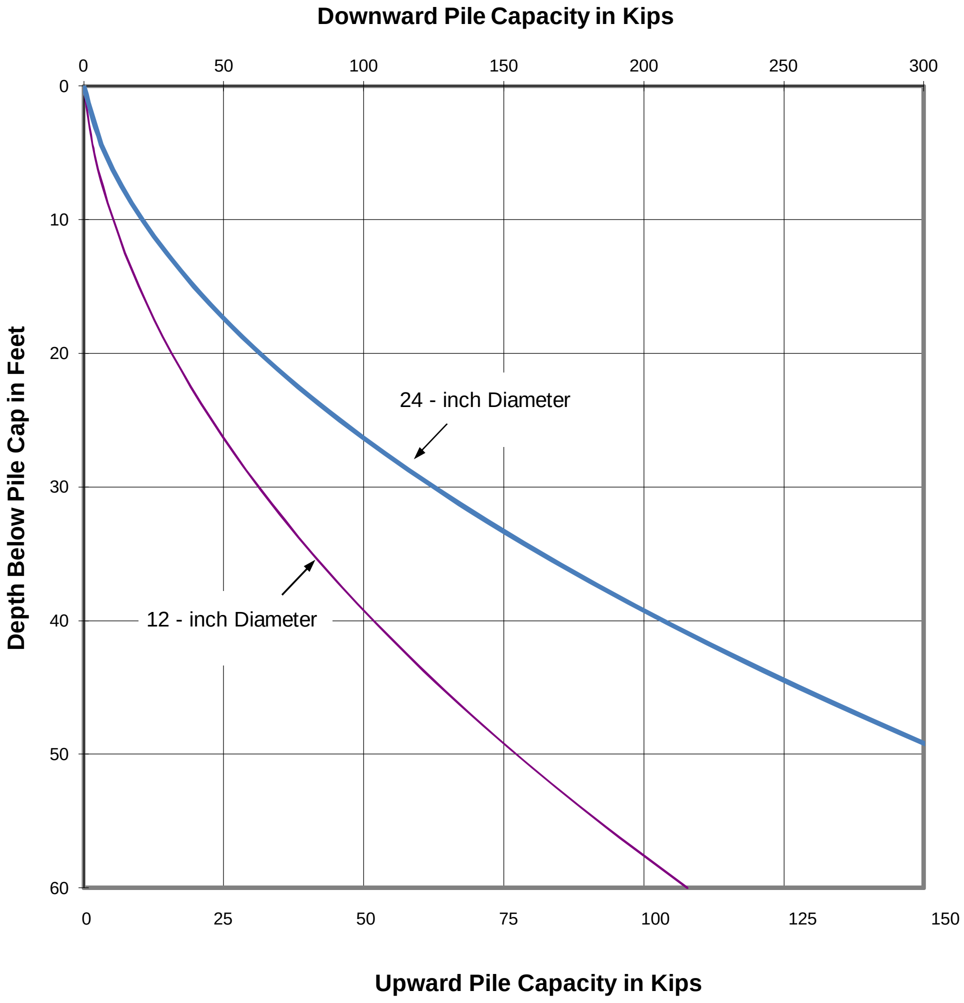


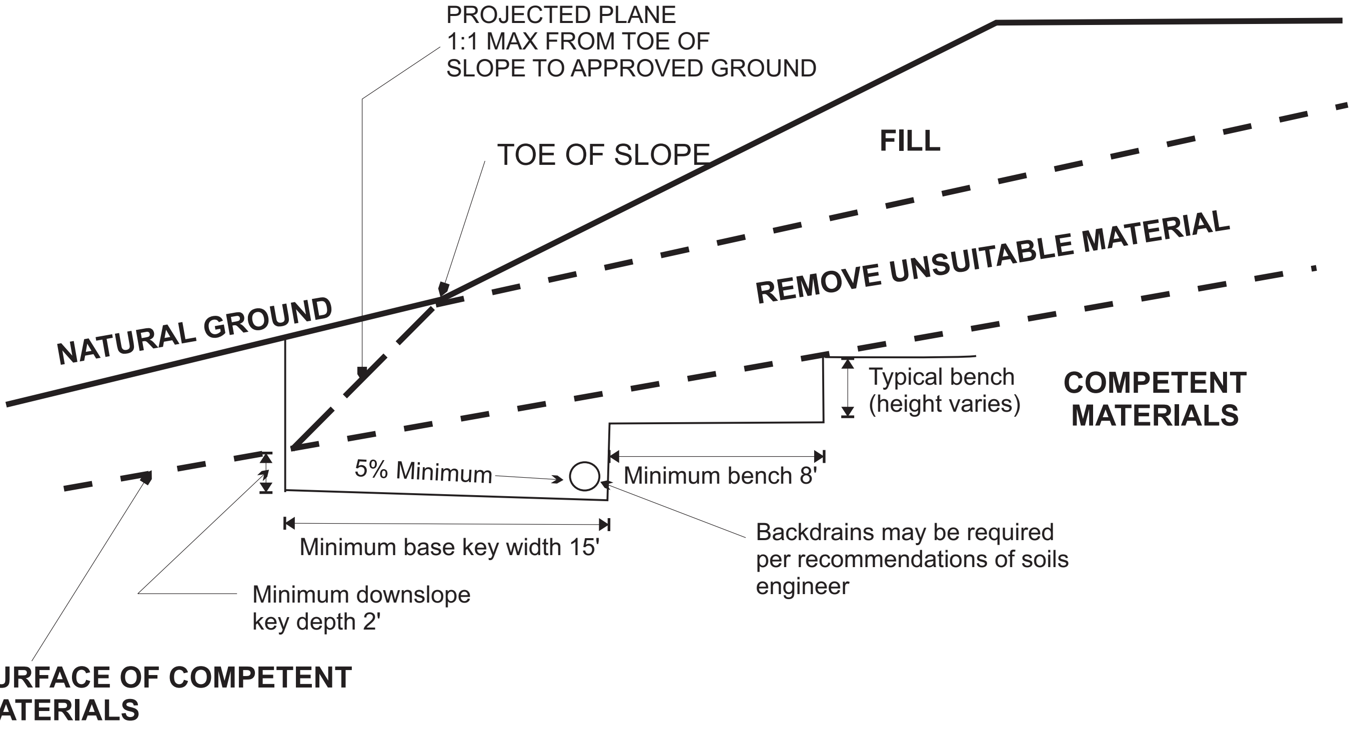
PILE CAPACITIES

NOTES:

1) The indicated values refer to the total dead plus live load. A one-third increase may be used for wind or seismic loads.

2) Piles in groups should be spaced at least 3 diameters on centers and should be drilled and filled alternately. The concrete should be allowed to set at least 8 hours before drilling the adjacent pile.

3) The values are based on the strength of the soils. The actual pile capacities may be less than those indicated and may be limited by the strength of the piles.



FILL SLOPE KEY

K2 ENGINEERING, INC.

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTS

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTS

FIELD EXPLORATIONS

The soil conditions beneath the site were explored by drilling three borings and three percolation test holes at the locations shown on Plates 2 and 3. The coordinates of the exploratory excavations are listed in Table A-1. The borings and test holes were drilled to depths of 3 to 31½ feet using 8-inch diameter continuous-flight and hollow-stem auger drilling equipment. After completion of the excavation, the borings were backfilled using bentonite grout. The percolation tests were backfilled using the excavated soils.

Table A-1,
Exploratory Excavations Coordinates

Boring/Percolation Number*	Latitude	Longitude
B-1/P-1	32°53.038’	117°12.023’
B-2/P-2	32°52.979’	117°11.969’
B-3/P3	32°52.899’	117°11.976’

**Percolation tests within 5 feet of borings. Coordinates are within the accuracy of the GPS used.*

The soils encountered were logged by our field engineer, who obtained bulk samples for laboratory observation and testing. A California-modified sampler was used to retrieve relatively undisturbed samples. This sampler consisted of a brass-ring-lined split-tube with an inside diameter of 2-1/2 inches and an outside diameter of 3 inches. The hammer used to drive the sampler weighed 140 pounds, and a drop of about 30 inches was used. The number of blows required to drive the sampler 12 inches is indicated on the logs. The logs of the borings are presented on Plates A-1.1 through A-1.3; the depths at which relatively undisturbed samples were obtained are indicated to the left of the logs.

The soils are classified in accordance with the Unified Soil Classification System described on Plate A-2.

LABORATORY TESTS

The field moisture content and dry density of the soils encountered were determined by performing tests on the relatively undisturbed samples. The results of the tests are shown to the left of the boring logs.

To aid in classifying the soils, three samples were tested to determine the percent passing the No. 200 sieve. The results are presented in the Boring Logs.

Tests to determine the Atterberg limits of one soil sample were performed in accordance with ASTM D2216. The results are presented in the boring logs.

Direct shear tests were performed on two undisturbed samples and one remolded sample compacted to 90% of the maximum dry density at near optimum moisture content. The tests were performed at various surcharge pressures after saturation. The peak point values determined from the direct shear tests are presented on Plate A-3, Direct Shear Test Data.

To evaluate the compressibility of the soils, confined consolidation tests were performed on two relatively undisturbed samples. Water was added to one of the samples during the tests to evaluate the effects of moisture on the compressibility. The test results are presented on Plates A-4.1 and A-4.2, Consolidation Test Data.

The optimum moisture content and maximum dry density of the soils were determined by performing a compaction test on a sample in accordance with ASTM D1557 method. The results of the test are presented on Plate A-5, Compaction Test Data.

To provide information for paving design, a Stabilometer (R-value) test was performed on a sample of the on-site soils. The test was performed in accordance with Standard 301 of the State of California Department of Transportation. The test results are presented on Plate A-6, R-Value Test Data.

The Expansion Index of the on-site soils was determined by testing three samples in accordance with ASTM D4829. The tests results are shown on Plate A-7, Expansion Index Test Data.

To determine the particle size distribution of the soils as an aid in classifying the soils, mechanical analyses were performed on two samples. The results of the mechanical analyses are presented on Plates A-8.1 and A-8.2, Particle Size Distribution.

To evaluate the corrosion potential of the on-site soils, two soil samples were tested at an analytical laboratory for pH, resistivity, sulfate and chloride content in accordance with the following standards.

Resistivity and pH – California Test 643
Soluble Chlorides – California Test 417
Soluble Sulfates – California Test 422

In addition the samples were tested for sulfides and the conductivity of soil extract determined. The test results are presented on Plate A-9, Corrosivity Test Data.

- oOo -

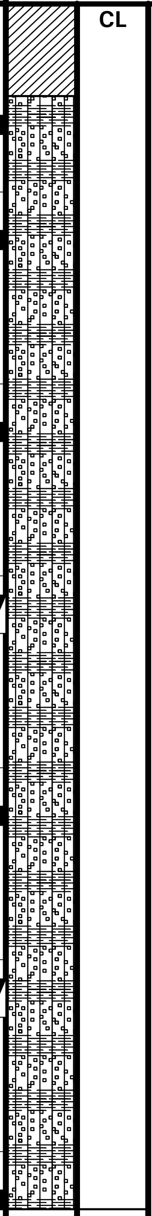
BORING B-1

DATE DRILLED: 4/19/16
EQUIPMENT USED: 8" Diameter Hollow-stem-auger

ELEVATION 373 **EAST** -117.2000 **NORTH** 32.8839

*ELEVATION (ft.)	DEPTH (ft.)	PID READINGS (ppm)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	**BLOWS/FOOT SPT "N" VALUES	SAMPLE LOC.
			18.8	103	66	■
			16.0	101	100	■
			15.6	112	100	■
			20.0		67	◩
			13.9	110	100	■
			16.7		92	◩
			15.2	112	100	■

Elevation scale: 370, 365, 360, 355, 350, 345, 340, 335
Depth scale: 5, 10, 15, 20, 25, 30, 35, 40



SILTY CLAY - medium stiff, some fine grained Sand, moist, dark brown

SCRIPPS FORMATION (Tsc)
Stiff to hard SILTSTONE, interbedded with fine grained, dense to very dense SILTY SANDSTONE, thinly bedded, weakly cemented, light orangish brown and greyish brown
93% Passing #200 sieve
Oxidized zones

<HOLE SQUEEZE>

Grey with reddish brown mottling
<ADDED WATER TO AID DRILLING>

94% Passing #200 sieve

NOTES: Water not encountered. No caving. Boring backfilled with bentonite grout.

Grey and light grey

<BOTTOM OF BORING AT 31-1/2 FEET>

*ELEVATIONS: Refer to datum of reference Site Plan. See Plate 2.1

**BLOWS/FOOT: Number of blows required to drive the sampler 12 inches using a 140 pound hammer

Note: The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditionsat other locations and times.

JOB G2015007-2.3 DATE 6/2/2016 F.T. VG BY SCK ENGR. SCK REV.

LOG OF BORING

K2 ENGINEERING, INC.

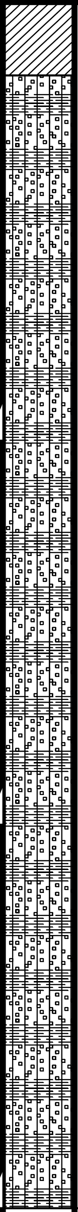
PLATE A-1.1

BORING B-2

DATE DRILLED: 4/19/16
EQUIPMENT USED: 8" Diameter Hollow-stem-auger

ELEVATION 373 **EAST** -117.1994 **NORTH** 32.8829

*ELEVATION (ft.)	DEPTH (ft.)	PID READINGS (ppm)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	**BLOWS/FOOT SPT "N" VALUES	SAMPLE LOC.
370			19.2	110	100	■
	5		15.8	108	100	■
365						
	10		18.6		68	◣
360						
	15		18.4	113	100	■
355						
	20		20.9		100	◣
350						
	25		15.6	106	100	■
345						
	30		20.1		92	◣
340						
	35					
335						
	40					



CL

SILTY CLAY - medium stiff, some fine grained Sand, moist, dark brown
(LL=50; PI=31)

SCRIPPS FORMATION (Tsc)
Stiff to hard SILTSTONE, interbedded with fine grained, dense to very dense SILTY SANDSTONE, thinly bedded, weakly cemented, light orangish brown and greyish brown
92% Passing #200 sieve
Light greyish brown

Grey with orangish-brown mottling

<ADDED WATER TO AID DRILLING>
93% Passing #200 sieve

89% Passing #200 sieve

<BOTTOM OF BORING AT 31-1/2 FEET>

NOTES: Groundwater not encountered. No caving.
Boring backfilled with bentonite grout.

Note: The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated.
It is not warranted to be representative of subsurface conditionsat other locations and times.

JOB G2015007-2.3 DATE 6/2/2016 F.T. VG BY SCK ENGR. SCK REV.

LOG OF BORING

K2 ENGINEERING, INC.

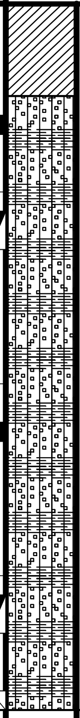
PLATE A-1.2

BORING B-3

DATE DRILLED: 4/19/16
EQUIPMENT USED: 8" Diameter Hollow-stem-auger

ELEVATION 375 EAST -117.1996 NORTH 32.8816

*ELEVATION (ft.)	DEPTH (ft.)	PID READINGS (ppm)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	**BLOWS/FOOT SPT "N" VALUES	SAMPLE LOC.
			15.2	106	87	■
370	5					
			14.5		91	◩
365	10					
			10.3	97	100	■
360	15					
			18.3		67	◩
					100	▧
355	20					
350	25					
345	30					
340	35					
335	40					



CL SILTY CLAY - medium stiff, dry to moist, dark brown

SCRIPPS FORMATION (Tsc)
Stiff to hard SILTSTONE, interbedded with fine grained, dense to very dense SILTY SANDSTONE, thinly bedded, weakly cemented, light greyish brown

<HOLE SQUEEZE>

<HARD DRILLING>

<BORING TERMINATED AT A DEPTH OF 18 FEET DUE TO HARD DRILLING AND SLOW PROGRESS>

NOTES: Water not encountered. No caving. Boring backfilled with bentonite grout.

Note: The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditionsat other locations and times.

JOB G2015007-2.3 DATE 6/2/2016 F.T. VG BY SCK ENGR. SCK REV.

LOG OF BORING

K2 ENGINEERING, INC.

PLATE A-1.3

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 20 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 seive size)	CLEAN GRAVELS (Little or no 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 FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 seive size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands, gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures.
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			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 plasticiy.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinaions of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

No. 200 No. 40 No.10 No. 4 3/4" 3" 12"

US STANDARD SIEVE SIZE

Key:

California Sampler

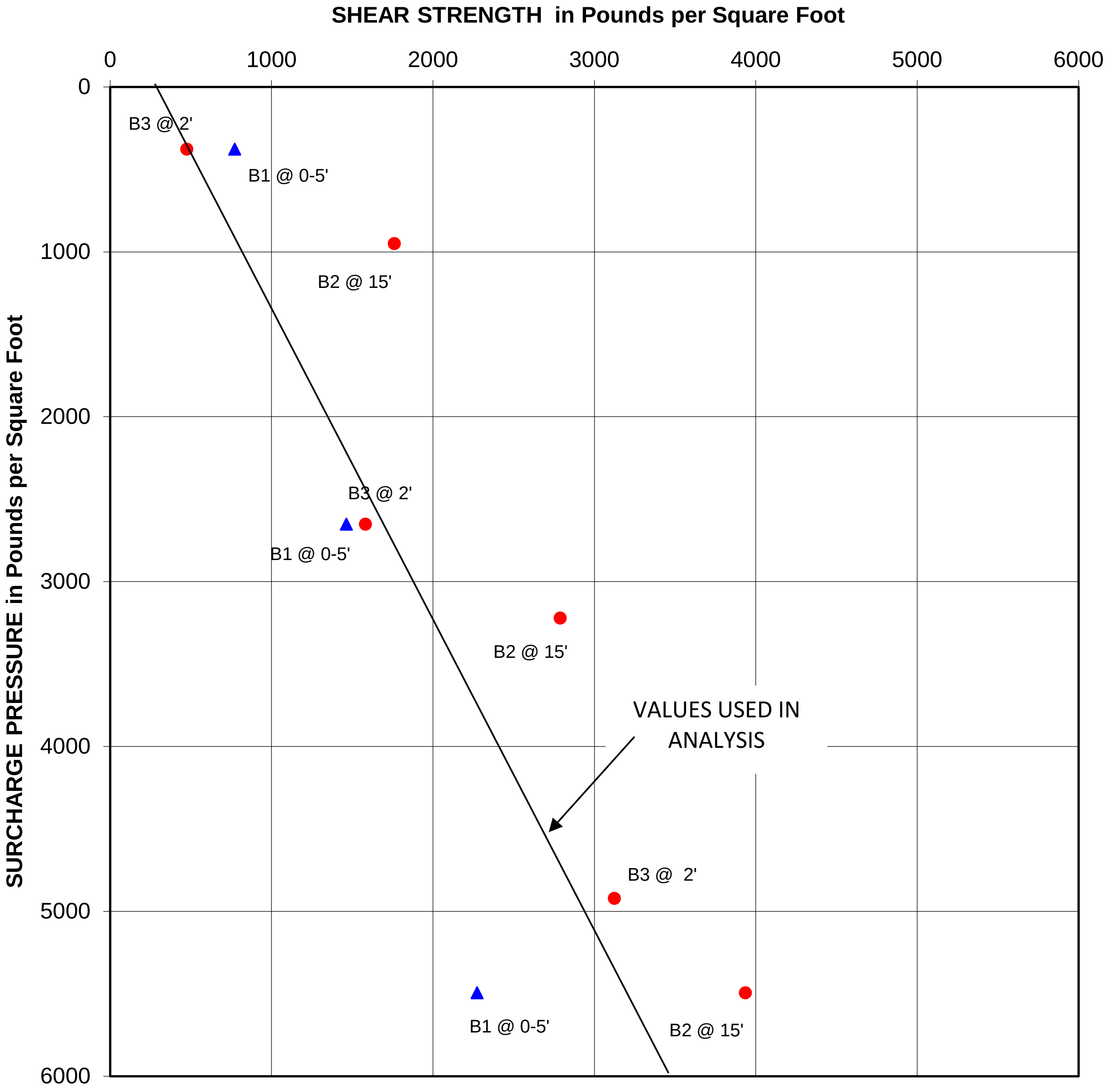
Standard Penetration

No Recovery

Continuous Auger Coring

HQ Coring

K2 ENGINEERING, INC.
GEOTECHNICAL CONSULTANTS



KEY:

● Relatively undisturbed samples

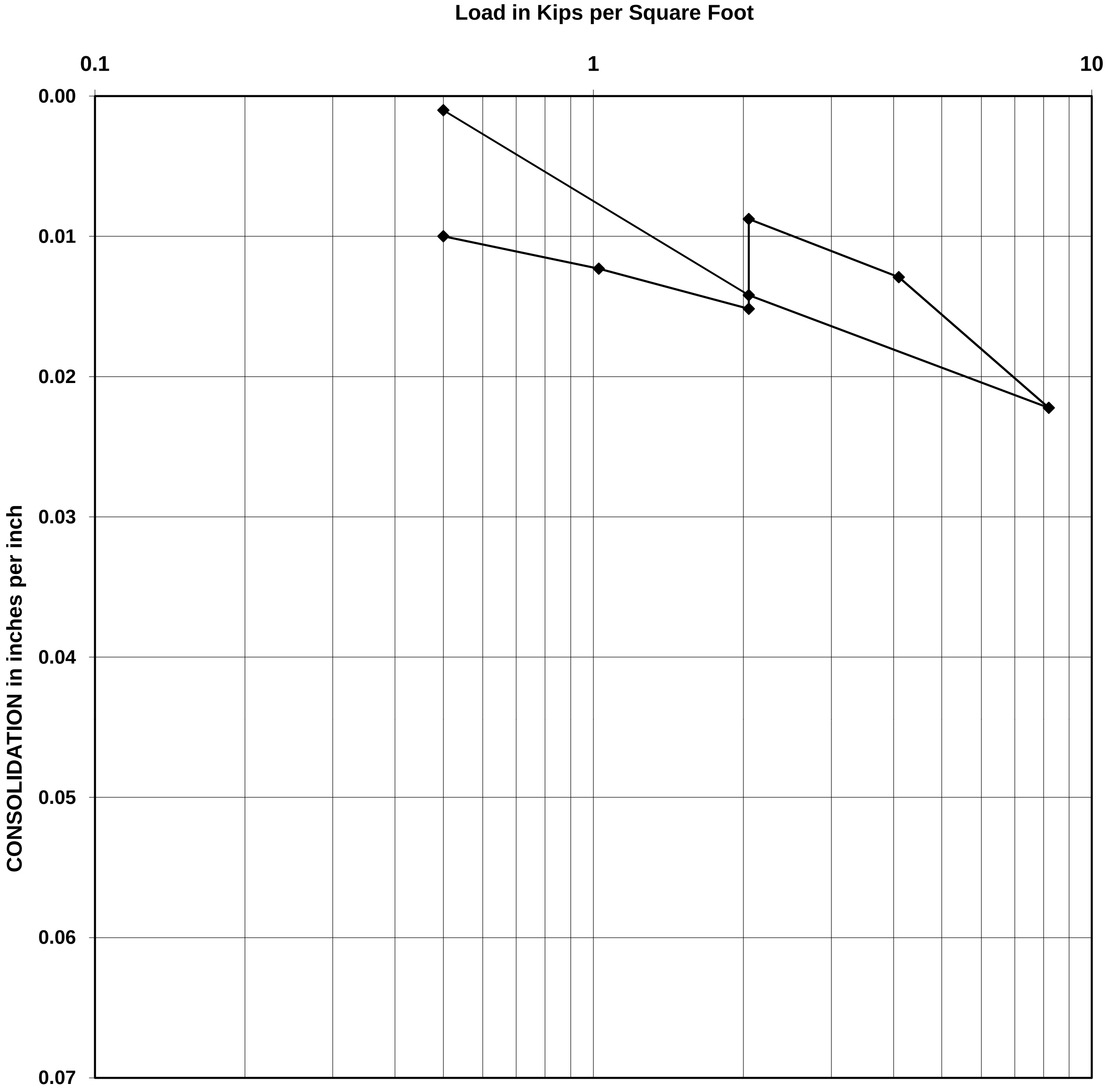
▲ Remolded samples compacted to 90%

└── Maximum values. All samples tested after saturation

SHEAR SUMMARY

Job No. G2015007-2.3

K2 ENGINEERING, INC.

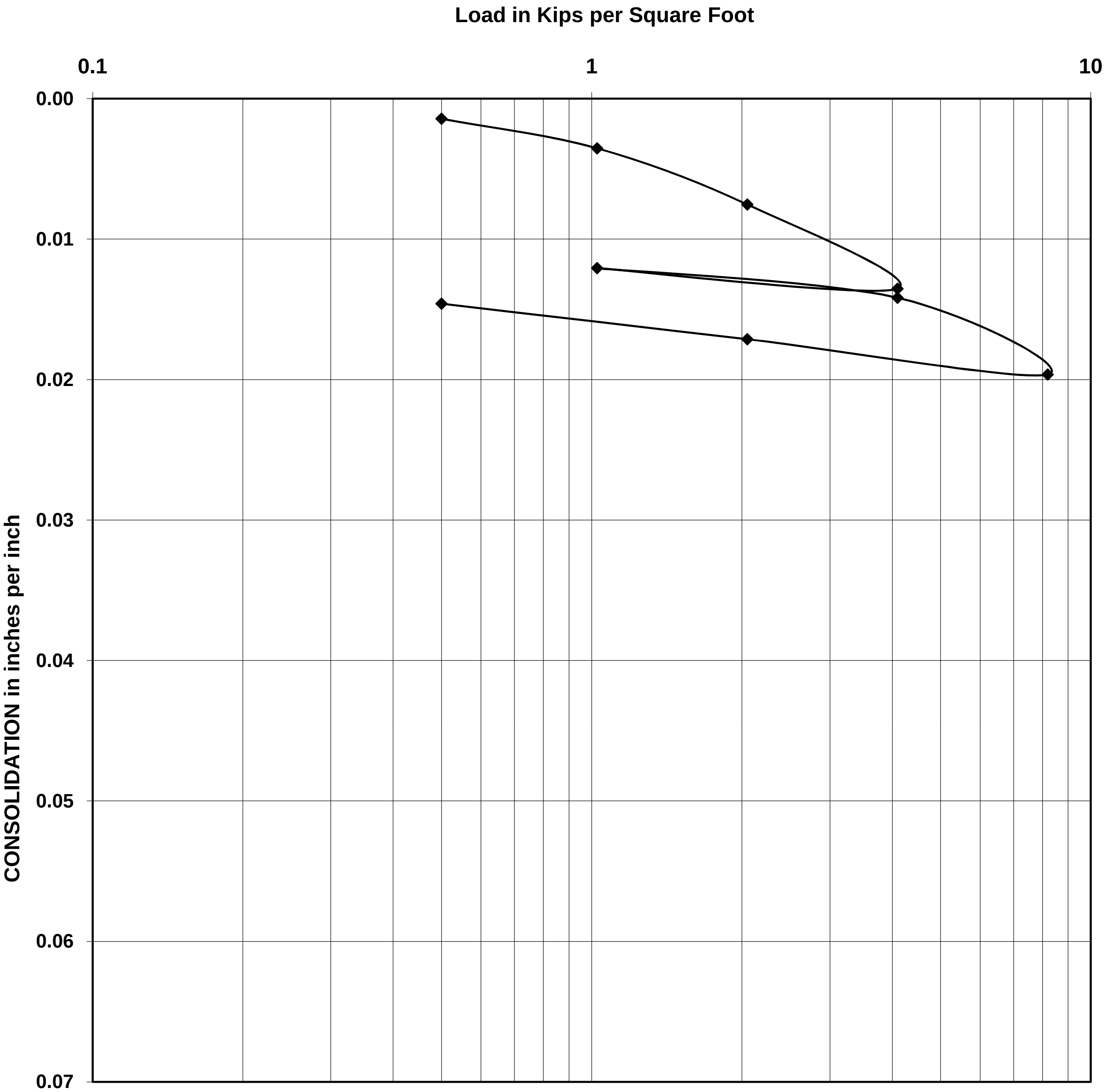


Note: Water added to the sample after consolidation under a load of 2.0 kips per square foot.

CONSOLIDATION TEST DATA

BORING	B-1	**SOIL TYPE**
DEPTH	5'	**SILTSTONE**
Job No.	G2015007-2.3	

K2 ENGINEERING, INC.



Note: Sample tested at field moisture content.

CONSOLIDATION TEST DATA

BORING	B-1	**SOIL TYPE**
DEPTH	20'	**SILTSTONE**
Job No.	G2015007-2.3	

K2 ENGINEERING, INC.

BORING NUMBER AND SAMPLE DEPTH	B2 @ 1 - 3'
SOIL TYPE	SILTY CLAY
MAXIMUM DRY DENSITY (lbs per cubic foot)	121
OPTIMUM MOISTURE CONTENT (% of dry weight)	12.7

COMPACTION TEST DATA

(ASTM D1557)

K2 ENGINEERING, INC.

Job No. G2015007-2.3

BORING NUMBER AND SAMPLE DEPTH	B2 @ 1 - 3'
SOIL TYPE	SILTY CLAY
R - VALUE	
by Exudation	5
by Expansion	9
at Equilibrium	5

R - VALUE TEST DATA

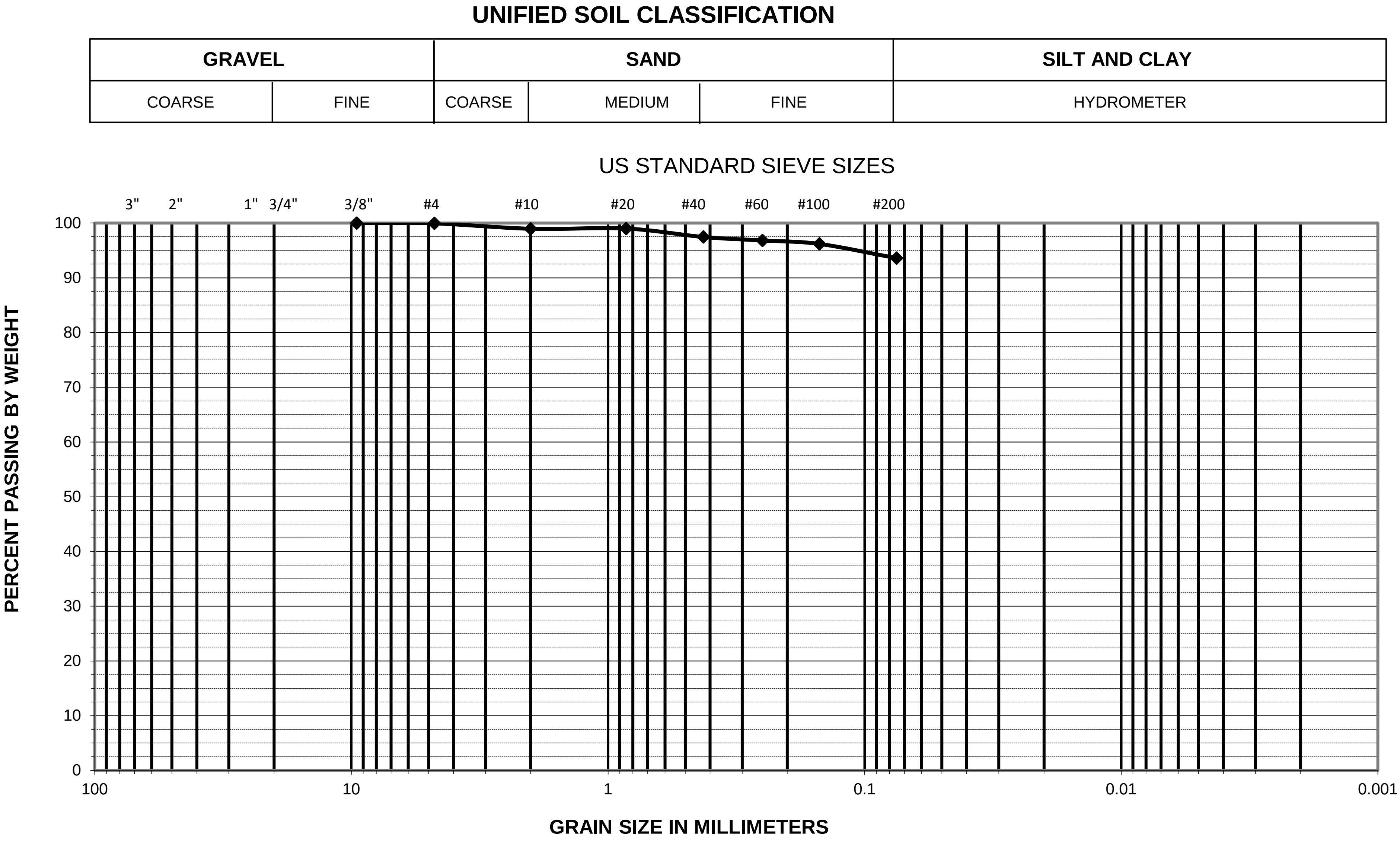
Job No. G2015007-2.3

ENGINEERING, INC.

BORING NUMBER AND SAMPLE DEPTH	B2 @ 1 - 3'	B2 @ 3 - 11' (COMPOSITE)	B3 @ 1 - 3'
SOIL TYPE	SILTY CLAY	SILTSTONE and SANDSTONE	SILTY CLAY
CONFINING PRESSURE (lbs per square foot)	144	144	144
FINAL MOISTURE CONTENT	24.5	18.7	26.7
DRY DENSITY (in pounds per cubic foot)	101	99	103
EXPANSION INDEX	113	85	99
EXPANSION POTENTIAL	HIGH	MEDIUM	HIGH

EXPANSION INDEX TEST DATA

(ASTM D4829)



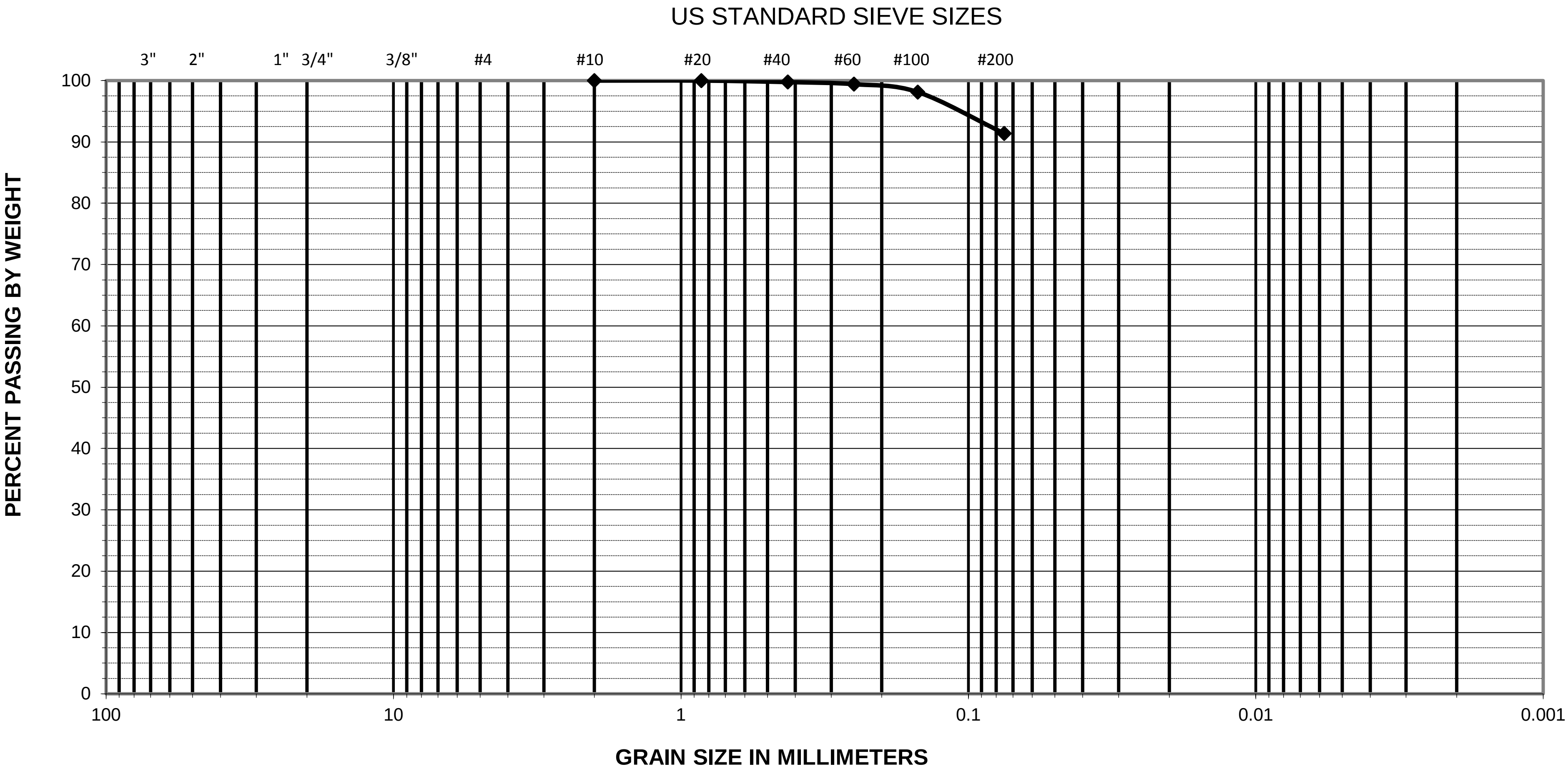
PARTICLE SIZE DISTRIBUTION

BORING	B1	**SOIL TYPE**
DEPTH	25'	**SILTSTONE**
Job No.		G2015007-2.3

K2 ENGINEERING, INC.

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	HYDROMETER



PARTICLE SIZE DISTRIBUTION

BORING	B2	**SOIL TYPE**
DEPTH	5'	**SILTSTONE**
Job No.		G2015007-2.3

K2 ENGINEERING, INC.

BORING NUMBER AND SAMPLE DEPTH	B-2 @ 1 - 3'	B-3 @ 10 - 15'	Caltrans Method
SOIL TYPE	SILTY CLAY	SILTSTONE	
pH	5.6	7.9	Caltrans 643
Resistivity (in ohms-cm)	190	190	Caltrans 643
Soluble Sulfate (%)	0.042	0.027	Caltrans 417
Soluble Chloride (%)	0.16	0.123	Caltrans 422
Sulfides (ppm)	0.2	0.7	SM 4500-S D, 22nd Ed, 2012
Electrical Conductance (EC) (micro ohms/cm)	6.8	6.7	USDA Handbook - Method 4

CORROSIVITY TEST DATA

Job No. G2015007-2.3

APPENDIX B

REPORTS BY OTHERS

PRELIMINARY GEOTECHNICAL RECONNAISSANCE
PROPOSED SLUDGE PROCESSING SITE
NORTH CITY – NORTH
CITY OF SAN DIEGO, CALIFORNIA

By: Geotechnical Consultants, Inc.

February, 1990

106051

LOG OF DRILL HOLE

Sludge Processing Sites
North City-North
METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: J. Thurber
CHECKED BY:

DRILL HOLE NO.: 17
DRILLING DATE: January 6, 1990
DATUM: City of San Diego
REFERENCE EL.: 380 Feet

REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	TORVANE (PSF)	ADDITIONAL TESTS
					"COLLUVIUM (Qc)" SANDY SILT (ML) brown, dry, soft.						
		1	72		""SCRIPPS FORMATION (Tsc)" SILTY SANDSTONE (R) light brown, damp, dense, sand is very fine to fine grained, locally oxidized, faintly laminated. Local volcanic gravel at 2 to 3 feet.						
		2	86		Very finely laminated, 2-3 degree dip.						
		3	50/ 5.5"		"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE/SANDSTONE (R) light gray and light brown, damp, stiff-dense, sand is very fine grained, minor clay.						
					Firm, slow drilling.						
		4	89		CLAYEY SILTSTONE (R) laminated gray and brown, laminations are subhorizontal.						
		5	50/ 6"		SILTSTONE/SANDSTONE (R) light brown, damp, dense, finely laminated.						
		6	50/ 6"		SANDSTONE (R) very fine grained, moderate amount of silt, locally oxidized.						
					Bottom of drill hole at 30 feet. No groundwater encountered. Drill hole backfilled and tamped.						

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012
PROJECT: Sludge Processing Sites
LOCATION: North City-North
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: M. Hosseini
CHECKED BY: J. Thurber

DRILL HOLE NO.: 18
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 380 Feet

ELEVATION (FEET)	DEPTH	DRILLING RATE	REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		TORVANE (PSF)	ADDITIONAL TESTS
											LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
380								"COLLUVIUM (Qc)" SILTY CLAY (CL) brown, dry, soft to medium stiff, scattered cobbles.						
	5			◢	1	63		"SCRIPPS FORMATION (Tsc)" SILTSTONE (R) light gray-brown, dry to damp, dense, thin claystone interlaminations. Interlaminated Siltstone/Claystone. Scattered hematite cementation. CLAYEY SILTSTONE (R) light gray-brown, damp, stiff.						
370	10			◢	2	89		Minor, thin, fine grained sand lenses.						
	15			◢	3	50/ 4"		Interlaminated CLAYEY SILTSTONE/SILTY CLAYSTONE (R) light gray/light brown, damp, stiff, laminations dip 3-4 degrees, locally oxidized, minor organics.						
360	20			◢	4	50/ 6"								
	25			◢	5	85/ 11"		CLAYEY SILTSTONE (R) light gray-brown, damp, stiff to medium hard, finely laminated-subhorizontal, minor iron oxide.						
350	30			◢	6	50/ 6"		Locally stiff, slow drilling. 2-3 inch Silty Sandstone layers, oxidized orange-brown, sand is very fine grained.						
	35			◢	7	88		CLAYEY SILTSTONE (R) light green-grey, damp, stiff, laminations subhorizontal, locally oxidized, minor organic matter, thin (less than 1/2 inch) medium grained sand layers.						
340	40			◢	8	86/ 11"		Interbedded Sandy Siltstone (R) and Clayey Siltstone (R) light gray, damp, stiff, sand is fine grained, 4-6 inch beds.						
								Bottom of drill hole at 40.5 feet. No groundwater encountered. Drill hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.2

106051

LOG OF DRILL HOLE

JOB NO.: S89012
PROJECT: Sludge Processing Sites
LOCATION: North City-North
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: M. Hosseini
CHECKED BY: J. Thurber

DRILL HOLE NO.: 19
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 380 Feet

ELEVATION (FEET)	DEPTH	DRILLING RATE	REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	TORVANE (PSF)	ADDITIONAL TESTS
380								"COLLUVIUM (Qc)" CLAYEY GRAVEL (Gc) brown, dry, dense, abundant cobbles.						
	5				1	51		"SCRIPPS FORMATION (Tsc)" SILTSTONE/CLAYSTONE (R) light gray-brown, dry to damp, laminated. Laminations are subhorizontal, locally oxidized, scattered organic-rich clay inclusions.						
370	10				2	52		SANDSTONE (R) very fine-grained, light gray, damp, dense (indurated), friable, uniform texture.						
	15				3	50/6"		Interlaminated SILTSTONE/CLAYSTONE (R) light gray/brown, damp, stiff (indurated), minor fine grained sand lenses.						
360	20				4	50/6"		SILTY SANDSTONE (R) light gray-brown, damp, dense, indurated, friable, locally oxidized. Sand is very fine grained. Streaks of tightly-cemented SILTSTONE (R) Very dense, well indurated SILTSTONE (R) bed, between 22 and 24 feet.						
	25				5	81		CLAYEY SILTSTONE (R) light gray-brown, damp, stiff (indurated), local oxidization, subhorizontal layers.						
350	30				6	89/11"		CLAYEY SILTSTONE (R) light gray-brown, damp, very stiff, locally hematite-stained along vertical micro-fractures. Scattered fine grained sand lenses. Very dense Siltstone layer at 32 feet.						
	35				7	89/11"		SILTSTONE (R) light grey/brown, damp, very firm, vertical and horizontal hematite stained fractures.						

Bottom of drill hole at 35.5 feet.
No groundwater encountered.
Drill hole backfilled and tamped.

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.3

106051

LOG OF DRILL HOLE

JOB NO.: S89012
PROJECT: Sludge Processing Sites
LOCATION: North City-North
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: J. Thurber
CHECKED BY:

DRILL HOLE NO.: 20
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 380 Feet

ELEVATION (FEET) DEPTH	DRILLING RATE	REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		TORVANE (PSF)	ADDITIONAL TESTS
										LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
380							"COLLUVIUM (Qc)/LINDAVISTA FORMATION (Qln)" SANDY SILT (ML) brown, dry, soft, scattered cobbles at surface to 2 feet.						
5				1	58		"SCRIPPS FORMATION (Tsc)" SILTSTONE (R) light brown, damp, stiff, locally medium hard, minor fine grained sand, thin (less than 1/4 inch) clay seams, damp, stiff, waxy.						
370 10				2	55		"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) light gray and brown, damp, stiff, finely laminated subhorizontal sandy layers are oxidized orange-brown.						
15				3	81		Laminations dip 2-3 degrees, minor to moderate amount of clay.						
360 20				4	72		Medium green-gray, damp, stiff to medium hard, laminated with numerous sand laminae, moderate amount of clay.						
							Bottom of drill hole at 20.5 feet. No groundwater encountered. Drill hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

Sludge Processing Sites
North City-North
METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: J. Thurber
CHECKED BY:

DRILL HOLE NO.: 21
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 380 Feet

REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		TORVANE (PSF)	ADDITIONAL TESTS
								LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
					"COLLUVIUM (Qc)/LINDAVISTA FORMATION (Qln)" SANDY SILT (ML) brown, dry, soft, scattered gravel and cobble.						
		1	53		"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) light gray-brown, damp, stiff, minor to moderate clay, brown clay layers with clay filling steep fractures, laminated, fissile.						
		2	80		Moderate amount of clay, 1/4 inch Sand-Silt laminae. Laminations dip 2-4 degrees, minor organic matter locally along laminations.						
		3	50/ 5.5"		SANDY CLAYEY SILTSTONE (R) light gray, damp, stiff to medium hard, sand is very fine grained, near horizontal lamination, minor oxidization locally.						
		4	75/ 11.5"		CLAYEY SILTSTONE (R) medium green gray, damp, stiff, laminations dip 3-4 degrees.						
					Bottom of drill at 20.5 feet. No groundwater encountered. Drill hole backfilled and tamped.						

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012
PROJECT: Sludge Processing Sites
LOCATION: North City-North
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: J. Thurber
CHECKED BY:

DRILL HOLE NO.: 22
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 350 Feet

ELEVATION (FEET)	DEPTH	DRILLING RATE	REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		TORVANE (PSF)	ADDITIONAL TESTS
											LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
350								"COLLUVIUM (Qc)" SANDY SILT (ML) brown, dry, soft.						
								"SCRIPPS FORMATION (Tsc)/POSSIBLE LANDSLIDE DEBRIS (Qls)"						
	5				1	46		CLAYEY SILTSTONE (R) medium gray, moderate amount of orange iron oxide, damp, stiff; moderate amount of clay, local steep micro-fractures, laminations dip 2-4 degrees.						
340	10				2	37		Light gray, minor iron oxide along laminations, fine grained sand laminae, sand fills pockets, burrows (less than 1/4 inch) laminations dip 5-6 degrees.						
	15				3	45		Carbonate cemented layer, hard. Gypsum fills laminae and open voids (up to 1/2 inch) below carbonate layer. Green-gray Clayey Siltstone (R), iron oxide locally.						
330	20				4	50/ 5.5"		Carbonate cemented layer over gypsum filled lamina (1/8 inch thick) over green-gray Siltstone (R).						
	25				5	55		Numerous gypsum filled laminations from less than 1/8 inch to 1 inch thick, in medium green-gray Clayey Siltstone (R), medium hard, fissile.						
								Bottom of drill hole at 25.5 feet. No groundwater encountered. Drill hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.6

LOG OF DRILL HOLE

JOB NO.: S89012
PROJECT: Sludge Processing Sites
LOCATION: North City-North
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: J. Thurber
CHECKED BY:

DRILL HOLE NO.: 23
DRILLING DATE: January 8, 1990
DATUM: City of San Diego
REFERENCE EL.: 320 Feet

ELEVATION (FEET) DEPTH	DRILLING RATE	REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		TORVANE (PSF)	ADDITIONAL TESTS
										LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
320							"SCRIPPS FORMATION (Tsc)/POSSIBLE LANDSLIDE DEBRIS (Qls)" CLAYEY SILTSTONE (R) gray-brown, damp, stiff, minor clay, moderate pervasive iron oxide, faintly laminated.						
5				1	46								
310 10					30/2"		Hard layer, calcium carbonate cement, 9 to 10 feet.						
				2	51		Siltstone, faintly laminated.						
15				3	61		CLAYEY SILTSTONE (R) gray green, damp, stiff, micro-fractures with brown iron oxide, gypsum fills steep fractures and pockets, less than 1/4 inch wide, locally calcareous.						
300 20				4	48		Locally moist, softer (medium stiff), green and brown (iron oxide), abundant thin (2-3 mm) gypsum laminae, laminations dip 2-4 degrees.						
							Bottom of drill hole at 20.5 feet. No groundwater encountered. Drill hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.7

106051

UNIFIED SOIL CLASSIFICATION SYSTEM

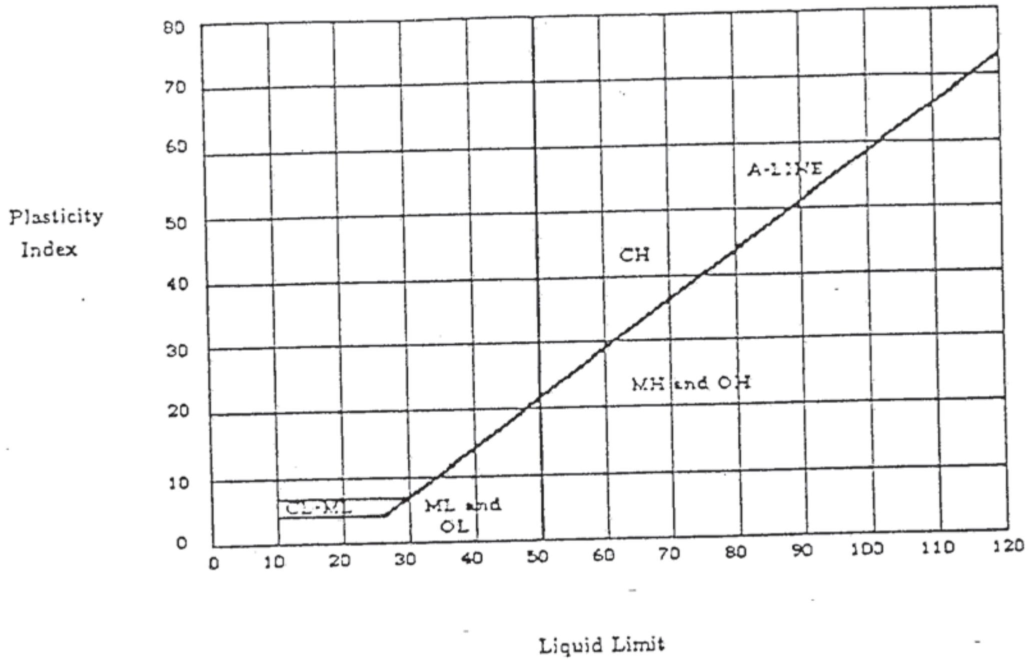
MAJOR DIVISION			GROUP SYMBOL	DESCRIPTION	GRAPHIC LOG
COARSE GRAINED SOILS Over 50% By Weight Coarser Than No. 200 Sieve Size	GRAVELLY SOILS OVER 50% OF COARSE FRACTION LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELLY SOILS LITTLE OR NO FINES	GW	WELL GRADED GRAVELS OR GRAVEL - SAND MIXTURES	▤
			GP	POORLY GRADED GRAVELS OR POORLY GRADED GRAVEL -SAND SILT MIXTURES	▤
		GRAVELLY SOILS WITH FINES OVER 12% FINES	GM	SILTY GRAVELS OR POORLY GRADED GRAVEL - SAND SILT MIXTURES	▤
			GC	CLAYEY GRAVELS OR POORLY GRADED GRAVEL - SAND - CLAY MIXTURES	▩
	SANDY SOILS OVER 50% OF COARSE FRACTION SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDY SOILS LITTLE OR NO FINES	SW	WELL GRADED SANDS OR GRAVELLY SANDS	░
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS	░
		SANDY SOILS WITH FINES OVER 12% FINES	SM	SILTY SANDS OR POORLY GRADED SAND - SILT MIXTURES	▥
			SC	CLAYEY SANDS OR POORLY GRADED SAND - CLAY MIXTURES	▨
FINE GRAINED SOILS Over 50% By Weight Finer Than No. 200 Sieve Size	SILTY AND CLAYEY SOILS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS SILTY/CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY	▥
			CL	INORGANIC CLAYS-LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTY OR LEAN CLAYS	▨
			OL	ORGANIC CLAYS OR ORGANIC SILTY CLAYS OF LOW PLASTICITY	▥
	SILTY AND CLAYEY SOILS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, OR ELASTIC SILTS	▥
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, OR FAT CLAYS	▨
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, OR ORGANIC SILTS	▤
HIGHLY ORGANIC SOILS			Pt	PEAT OR OTHER HIGHLY ORGANIC SOIL	▤

SAMPLE TYPES:

■	UNDISTURBED SLEEVE
☒	DISTURBED
☐	UNSUCCESSFUL ATTEMPT
◢	STANDARD PENETRATION
▼	WATER LEVEL
≡	WATER INFLOW

LEGEND TO LOGS

PLASTICITY CHART - Used for Classification of Fine Grained Soils



BLOW COUNT - The number of blows required to drive the indicated sampler the last 12 inches of an 18 inch drive. The notation 100/9 indicates only 9 inches of penetration were achieved in 100 blows. Hammer weights and drop heights are shown below:

Symbol	Driving Weight (pounds)	Drop Height (inches)
7		
(3)		
[6]		


Heavy Caving


Light Caving

ADDITIONAL TESTS -

UC : Unconfined Compression
TD : Triaxial Compression, Drained
TU : Triaxial Compression, Undrained
TDy: Triaxial Compression, Dynamic
PH : Hydrogen Ion Concentration
PA : Paleontologic, Analysis
GS : Grain Size Distribution

WP : Water Pressure
PMt: Pressuremeter
SE : Sand Equivalent
GJ : Goodman Jack
SP : Specific Gravity
CP : Compaction
C : Consolidation
DS : Direct Shear

PM : Permeability
EX : Expansion
RS : Resistivity
S : Swell
CL : Chloride
SU : Sulphate

GEOTECHNICAL INVESTIGATION
PROPOSED NORTH CITY SLUDGE PROCESSING FACILITIES AT I-805 AND EASTGATE MALL
CITY OF SAN DIEGO, CALIFORNIA

By: Geotechnical Consultants, Inc.

November, 1990

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 108
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 366 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
				1	37		"COLLUVIUM (Qc)" SANDY SILT (ML) red-brown, damp, hard and SILTY CLAY (CL) tan, damp, hard, slightly plastic.		12				
				2	82/ 10"		"SCRIPPS FORMATION (Tsc)" SILTSTONE (R) tan, damp, hard, moderate iron oxidation.		9				
	5												
360				3	76								
							7 feet: 1-1/2 feet concretion of well cemented Siltstone, very hard.						
	10			4	72/ 10"		Clayey siltstone, tan to light gray, damp, hard to very hard, fissile, moderate iron oxidation.		12				
	15			5	100/ 10"								
350													
	20			6	82		Laminated.						
	25			7	83			99	20				
340													
	30			8	86/ 11"								
							Bottom of drill hole at 30 feet. No groundwater encountered. Hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.8

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 109
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 373 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
										LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
370				1	41	▨	"FILL (af)" CLAYEY SILT (CL) tan, damp, hard, local roots, slightly weathered.		14	42	24		AL
			■	2	77	▤	"SCRIPPS FORMATION (Tsc)"		12				
	5		■	3	65/6"	▤	CLAYEY SILTSTONE (R) tan, damp, hard, moderate iron oxidation, fissile.		7				

Bottom of drill hole at 5.5 feet.
No groundwater encountered.
Hole backfilled and tamped.

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.9

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 110
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 367 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
			◪	1	22		"FILL (af)" CLAY (CH) dark brown, moist, very stiff, plastic, local roots.		22	63	25		AL
	5		■	2	74		"SCRIPPS FORMATION (Tsc)" SILTSTONE (R) tan to light gray, damp, hard, fissile, moderate iron oxidation.						RS,PH CL,SU
360													
	10		■	3	33		Clayey siltstone, light brown, damp, moderately hard, fissile, 1 foot interbed of fine grained.	94	18				
	15		◪	4	92		Becoming olive green, moist, plastic, laminated.						
350													
	20		■	5	69/6"				9				
							23 feet: 1 foot concretion of well cemented siltstone medium gray, damp, very hard.						
	25		■	6	92/ 11"		Sandy siltstone, brown, damp, hard, fine grained sands, local hematite.		7				
340													
	30		◪	7	84/ 11"		Fissile, moderate iron oxidation.						
	35		•	8	75/3"		1 foot concretion of well cemented siltstone. Refusal at 35 feet.						
							Bottom of drill hole at 35 feet. No groundwater encountered. Hole backfilled and tamped.						

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 113
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 380 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
380			◪	1	57	▨	"FILL (af)" CLAY (CH) dark brown, moist, hard, local pebbles.		15				
	5		■	2	47		"LINDAVISTA FORMATION (Qln)" CONGLOMERATE (R) gray granitic and purple volcanic cobbles (1/2-3 inches), very hard, subrounded in clayey silt matrix, red-brown, damp, poorly indurated.	101	20				
							"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) tan to light gray, damp, medium hard, moderate iron oxidation, fissile.						
370	10		■	3	81/ 11"			107	16				
	15		◪	4	81/ 10"								
			□	5									
360	20		■	6	98/ 10"								
	25		◪	7	98/ 11"								
350	30		■	8	82/ 11"		Uniform lithology.	101	19				
	35		■	9	93/ 11"								
340	40		◪	10	78/ 11"								
							Bottom of drill hole at 40.5 feet. No groundwater encountered. Hole backfilled and tamped.						

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 114
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 375 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
			☒	1		▤	"SCRIPPS FORMATION (Tsc)"						
			◪	2	71	▤	CLAYEY SILTSTONE (R) tan, damp, medium hard, moderate iron oxide, fissile.		11	40	22		AL
			☒			▤							
370	5		■	3	97	▤							RS,PH, CL,SU
	10		■	4	70/6"	▤	Sandy siltstone, tan, damp, hard, fine grained sand.		8				
			☒	5		▤							
360	15		■	6	70/6"	▤	Clayey siltstone, tan, dry, hard, fissile.		11				
	20		◪	7	50/6"	▤	Refusal at 21 feet.						
							Bottom of drill hole at 21 feet. No groundwater encountered. Hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.14

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Hollow Stem Auger, 7-inch

LOGGED BY: Y. Van
CHECKED BY: T. Huber

DRILL HOLE NO.: 115
DRILLING DATE: September 12, 1990
DATUM: See Plate 1
REFERENCE EL.: 373 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
370			◢	1	56		"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) light tan, damp, hard, fissile, local iron oxidation.		14				
	5		■	2	79			98	16				
			☒	3									
	10		■	4	64		Uniform lithology.	107	22				R-Value
			⊡	5									
360													
	15		◢	6	92		Abundant patches of manganese oxide.						
	20		■	7	93			104	18				
350													
	25		■	8	91		23 feet: 1 foot concretion of well cemented siltstone, very hard. 24 feet: 6-inch layer of sandy siltstone, red-brown, damp, hard, fine grained sand.						
	30		◢	9	98/ 11"								

Bottom of drill hole at 30.5 feet.
No groundwater encountered.
Hole backfilled and tamped.

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Rotary Bucket, 24-inch

LOGGED BY: Y. Van
CHECKED BY: J. Thurber

DRILL HOLE NO.: 202
DRILLING DATE: September 8, 1990
DATUM: See Plate 1
REFERENCE EL.: 376 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS		POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
										LIQUID LIMIT (%)	PLASTIC LIMIT (%)		
							"COLLUVIUM (Qc)" SANDY SILT (ML) tan, dry, firm.						
							"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) tan, dry to damp, medium hard.						
370	5		■	1	(2)		Moderately weathered, iron oxidation lined laminations.	100	15				GS,DS
	10		■	2	(6)		Fissile, abundant iron oxidation, local manganese oxide. @ 12 feet: Bed N82W 5NE	106	15				
360	15		■	3	(14)		Interbed of sandy siltstone, tan, damp, moderately indurated, moderately hard. Siltstone, light brown, damp, moderately hard, fissile, moderate manganese oxides, no visible fractures, local small concretions. @ 17-20 feet: Interbed of sandstone, brown, fine grained, moderately dense.	102	16				
	20		■	4	(9)		Slightly weathered, local iron oxidation. Concretion of well cemented sandstone, 6 inches thick. @ 20 feet: Bed N67W 6NE. Minor clay, laminated.	109	15				GS
			☒	5									
350	25		■	6	(11)		""SCRIPPS FORMATION (Tsc)" CLAYSTONE (R) brown, damp, poorly indurated, plastic. @ 26.2-27.5 feet: Concretion of well cemented sanstone, medium gray, dry, very hard, local hematite and manganese oxide, gypsum lined.	110	19				
	30		■	7	[14]		@ 27.5-28.5 feet: Interbed sanstone, red-brown, damp, friable, moderately hard, medium grained, abundant iron oxide. @ 29 feet: Contact N6W 2NE sanstone/claystone. 1 foot layer of claystone interbed. Olive green, damp, poorly indurated, weathered, abundant iron oxidation, slightly plastic. @ 30 feet: Contact N43E 3SE sanstone/claystone, interbed of claystone, brown, damp, poorly indurated, plastic.						
340	35		■	8	[21]		@ 35-46 feet: Massive clayey siltstone. Concretion of well cemented siltstone, 3 inches thick.	106	20				
	40												
330	45		■	9	[19]		"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) brown, damp, moderately hard, moderately indurated, fissile, laminated, abundant iron oxidation. @ 46-47 feet: Concretion of well cemented siltstone.	106	19				
	50												
							@ 54 feet: Bed N80W 12NE.						

LEGEND TO LOGS ON PLATE A-2

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Rotary Bucket, 24-inch

LOGGED BY: Y. Van
CHECKED BY: J. Thurber

DRILL HOLE NO.: 202
DRILLING DATE: September 8, 1990
DATUM: See Plate 1
REFERENCE EL.: 376 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
320	60		■	10	[13]	▤	"SCRIPPS FORMATION (Tsc)" CLAYEY SILTSTONE (R) brown, moderately hard, fissile, laminated.	104	20				
							Bottom of drill hole at 60 feet. No groundwater encountered. Hole backfilled and tamped.						

LEGEND TO LOGS ON PLATE A-2

106051

LOG OF DRILL HOLE

JOB NO.: S89012B
PROJECT: Sludge Dewatering Facility
LOCATION: North City, San Diego
DRILLING METHOD: Rotary Bucket, 24-inch

LOGGED BY: Y. Van
CHECKED BY: J. Thurber

DRILL HOLE NO.: 204
DRILLING DATE: September 10, 1990
DATUM: See Plate 1
REFERENCE EL.: 379 Feet

ELEVATION (FEET)	DEPTH (FEET)	DRILLING RATE REAL TIME/DEPTH	SAMPLE	SAMPLE NO.	BLOW COUNT (BLOWS PER FOOT)	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT (%)	ATTERBERG LIMITS PLASTIC LIMIT (%)	POCKET PENETROMETER (TSF)	ADDITIONAL TESTS
							"COLLUVIUM (Qc)" SANDY SILT (ML) tan, dry, soft.						
	5		■	1	(3)		"LINDAVISTA FORMATION (Qln)" CONGLOMERATE (R) granitic and volcanic cobbles in sandy siltstone matrix, brown, damp, moderately hard. @ 3-4 feet: Clay (CL) dark brown to black, moist, hard, plastic, local pebbles and roots, weathered. Contact: N52W2SW	92	9				GS
370	10		■	2	(8)		"SCRIPPS FORMATION (Tsc)" SILTSTONE (R) tan to brown, damp, moderately hard, abundant iron oxides, laminated, moderately indurated, fissile.	110	15				
	15		■	3	(18)								RS,PH, CL,SU
360	20		■	4	(14)		Interbed of sandy siltstone, golden brown, dry to damp, moderately hard, fine grained sand, laminated. 19.5 - 20.5 feet: Well cemented sandstone concretion. Siltstone, massive, minor fine grained sand, local indurated beds. Minor to moderate amounts of clay.	98	13				
	25						Weakly laminated.						
350	30		■	5	[5]			108	16	54	33		AL
	35						Moderate amounts of clay. 33.5 - 34.5 feet: 1/2 foot concretion of well cemented sandstone.						
340	40		■	6	[39]		40-41 feet: 1 ft. concretion of well cemented sandstone.						
							Bottom of drill hole at 41 feet. No groundwater encountered. Hole backfilled and tamped.						

SHEET 1 OF 1

LEGEND TO LOGS ON PLATE A-2

PLATE A-1.19

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISION			GROUP SYMBOL	DESCRIPTION	GRAPHIC LOG
COARSE GRAINED SOILS Over 50% By Weight Coarser Than No. 200 Sieve Size	GRAVELLY SOILS OVER 50% OF COARSE FRACTION LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELLY SOILS LITTLE OR NO FINES	GW	WELL GRADED GRAVELS OR GRAVEL - SAND MIXTURES	▤
			GP	POORLY GRADED GRAVELS OR GRAVEL - SAND MIXTURES	▤
		GRAVELLY SOILS WITH FINES OVER 12% FINES	GM	SILTY GRAVELS OR POORLY GRADED GRAVEL - SAND - SILT MIXTURES	▦
			GC	CLAYEY GRAVELS OR POORLY GRADED GRAVEL - SAND - CLAY MIXTURES	░
	SANDY SOILS OVER 50% OF COARSE FRACTION SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDY SOILS LITTLE OR NO FINES	SW	WELL GRADED SANDS OR GRAVELLY SANDS	▒
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS	▒
		SANDY SOILS WITH FINES OVER 12% FINES	SM	SILTY SANDS OR POORLY GRADED SAND - SILT MIXTURES	▥
			SC	CLAYEY SANDS OR POORLY GRADED SAND - CLAY MIXTURES	▨
FINE GRAINED SOILS Over 50% By Weight Finer Than No. 200 Sieve Size	SILTY AND CLAYEY SOILS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY	▥
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, OR LEAN CLAYS	░
			OL	ORGANIC CLAYS OR ORGANIC SILTY CLAYS OF LOW PLASTICITY	▥
	SILTY AND CLAYEY SOILS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, OR ELASTIC SILTS	▥
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, OR FAT CLAYS	▨
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, OR ORGANIC SILTS	▤
HIGHLY ORGANIC SOILS			Pt	PEAT OR OTHER HIGHLY ORGANIC SOIL	▤

SAMPLE TYPES:

▦	UNDISTURBED SLEEVE
☐	DISTURBED
☐	UNSUCCESSFUL ATTEMPT
◢	STANDARD PENETRATION

▼	WATER LEVEL
≡	WATER INFLOW

LEGEND TO LOGS

PRELIMINARY GEOTECHNICAL INVESTIGATION
EASTGATE MALL SITE
SITE DEVELOPMENT PROJECT
CITY OF SAN DIEGO, CALIFORNIA

By: Ninyo and Moore

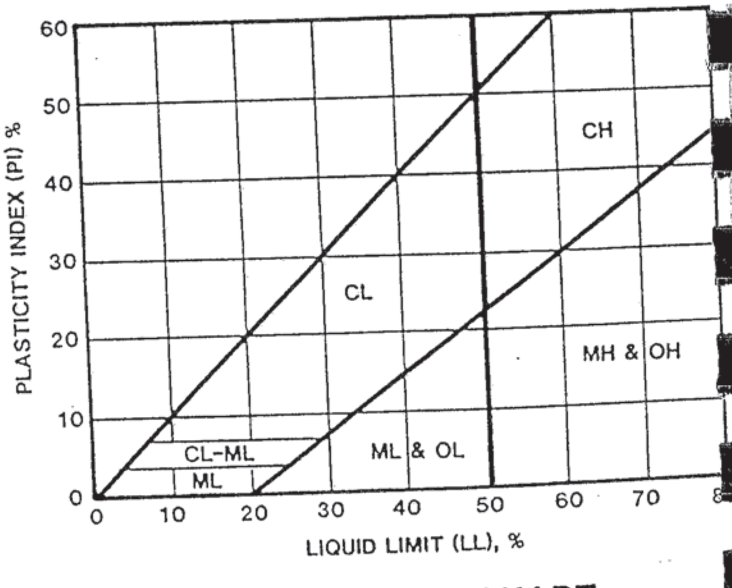
March 15, 1991

MAJOR DIVISIONS		SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > No. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > No. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < No. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < No. 200 sieve size)	SILTS & CLAYS Liquid Limit < 50	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		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 > 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

CLASSIFICATION CHART (Unified Soil Classification System)

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
Coarse	3" to 3/4"	76.2 to 19.1
Fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
Coarse	No. 4 to No. 10	4.76 to 2.00
Medium	No. 10 to No. 40	2.00 to 0.420
Fine	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

GRAIN SIZE CHART



PLASTICITY CHART

Ninyo & Moore | U.S.C.S. METHOD OF SOIL CLASSIFICATION

Ninyo & Moore

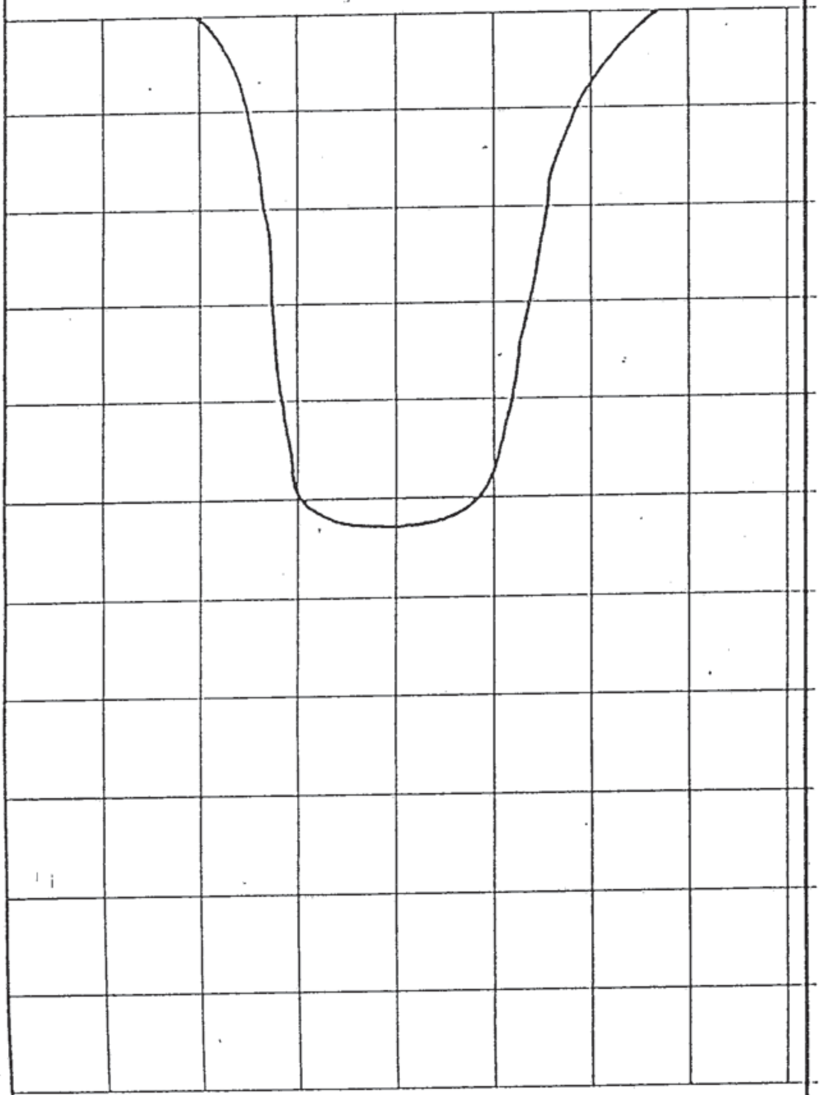
TEST PIT LOG

Explanation of Test Pit Log

PROJECT NO.
DATE

DATE EXCAVATED
TEST PIT NO.
GROUND ELEVATION
LOGGED BY
METHOD OF EXCAVATION
LOCATION

DEPTH (FEET)	SAMPLES: Bulk	SAMPLES: Driven	SAMPLES: Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
		■					Drive Tube density sample
			■				Sand Cone density sample
	■						Bulk sample
				♀			Seepage
				▼			Ground water table



SCALE

FIGURE

Ninyo & Moore

TEST PIT LOG

Eastgate Mall Site
North City Reclamation Plant and
Sludge Processing Facilities

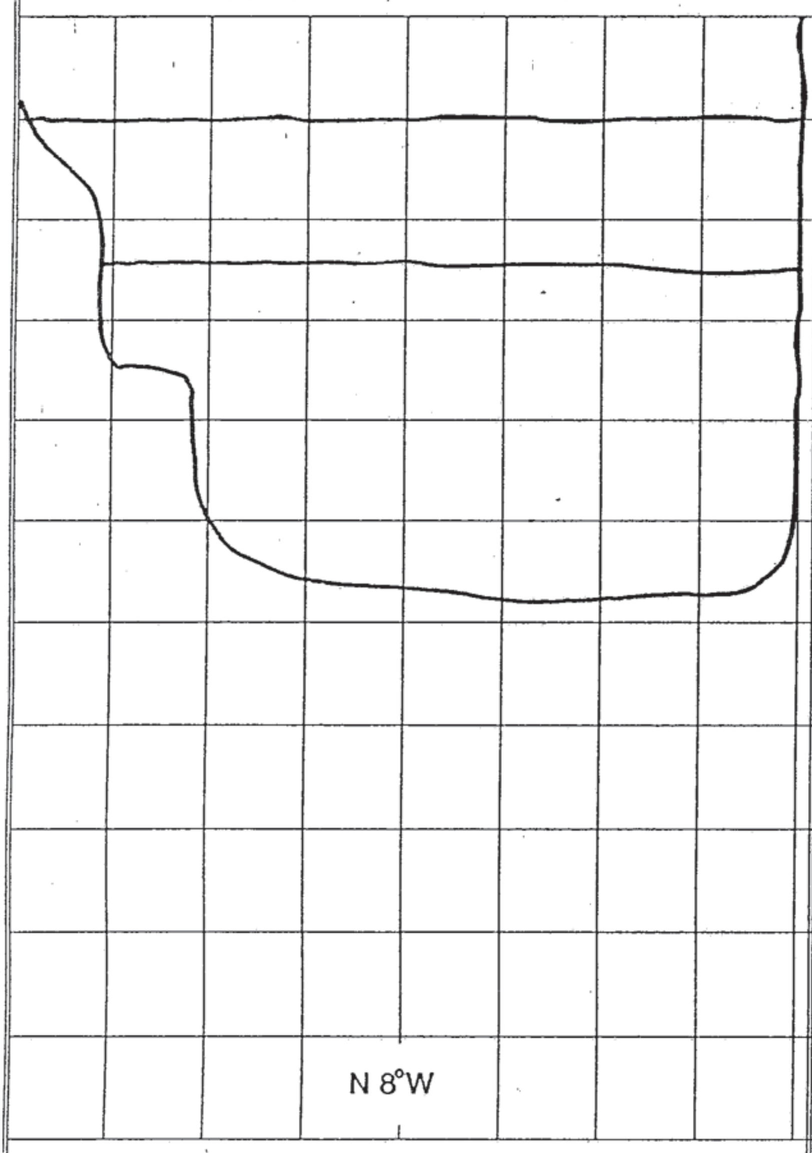
PROJECT NO. 101948-01
DATE 3/91

DATE EXCAVATED [illegible] TEST PIT NO. [illegible]

GROUND ELEVATION 380' +/- MSL LOGGED BY GJS

METHOD OF EXCAVATION JD-510 Backhoe

LOCATION See Plate One



SCALE 1" = 4'

DEPTH (FEET)	SAMPLE Bulk	SAMPLE Driven	SAMPLE Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	TOPSOIL/COLLUVIUM: Dark brown, damp, medium dense, clayey, silty SAND.
2							LINDAVISTA FORMATION: Yellow-brown, damp, dense to very dense, silty SAND with abundant rounded gravel and cobbles to 10 inches in diameter.
4							SCRIPPS FORMATION: Yellow-brown to light brown, damp, hard, SILTSTONE, with interbeds of gray, silty and yellow-brown, fine sand and light brown clay.
6	■						
8							
10							
12							Total Depth 11.5' No Caving No Ground Water Encountered Backfilled 2/8/91
14							
16							
18							
20							
22							

Ninyo & Moore

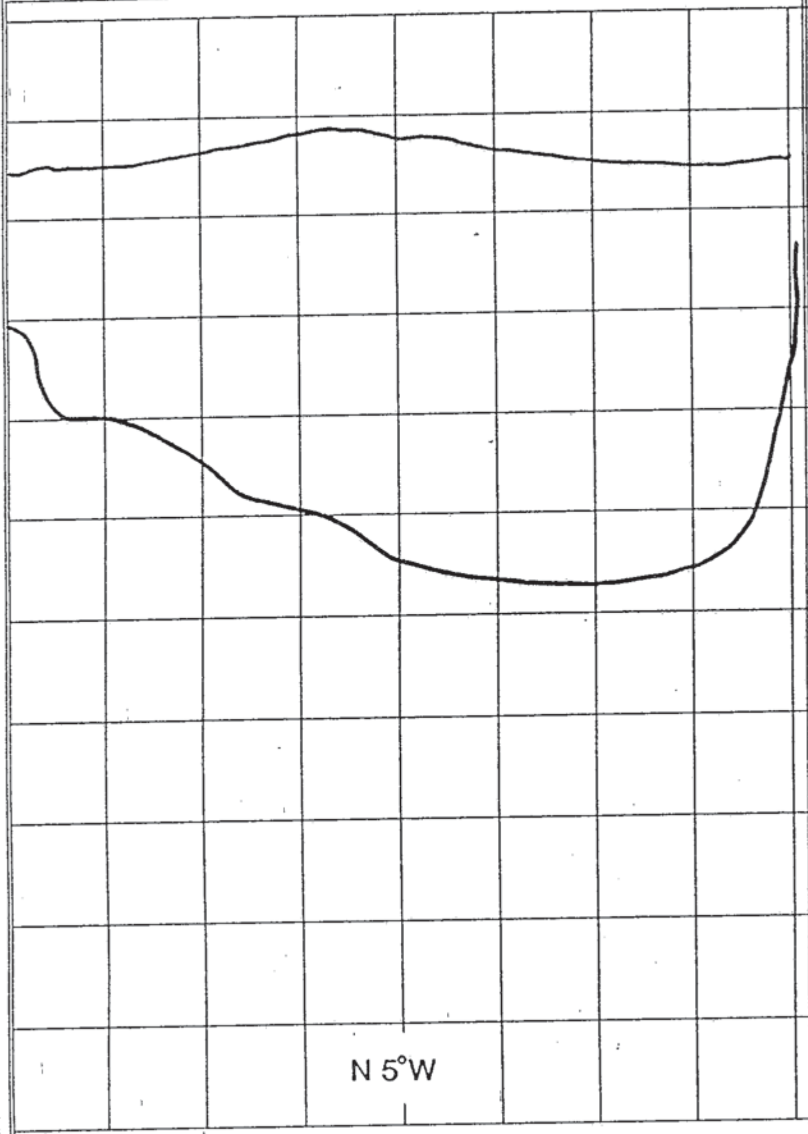
TEST PIT LOG

Eastgate Mall Site
North City Reclamation Plant and
Sludge Processing Facilities

PROJECT NO. 101948-01
DATE 3/91

DATE EXCAVATED 2/8/91
TEST PIT NO. TP-2
GROUND ELEVATION 380'+/- MSL
LOGGED BY GJS
METHOD OF EXCAVATION JD-510 Backhoe
LOCATION See Plate One

DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM-CL	TOPSOIL/COLLUVIUM: Dark brown, damp, medium dense, clayey, silty SAND and firm, sandy CLAY with occasional rounded gravel.
2							
4							SCRIPPS FORMATION: Yellow-brown to light brown, damp, stiff to hard, SILTSTONE with interbeds of yellow-brown, silty, fine sand and gray silt.
6	■						
8							
10							Total Depth 11.0' No Caving No Ground Water Encountered Backfilled 2/8/91
12							
14							
16							
18							
20							
22							



SCALE 1" = 4'

FIGURE B-15

Ninyo & Moore

TEST PIT LOG

Eastgate Mall Site
North City Reclamation Plant and
Sludge Processing Facilities

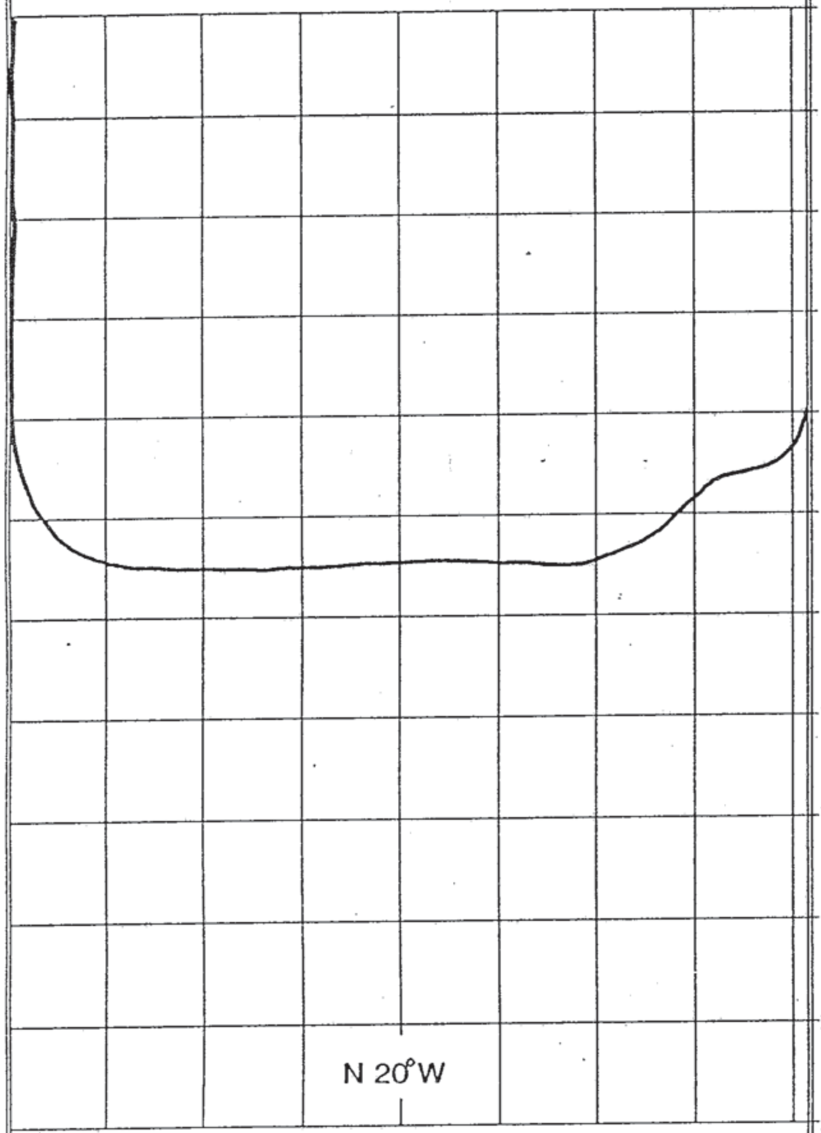
PROJECT NO.	DATE
101948-01	3/91

DATE EXCAVATED 2/8/91 TEST PIT NO. TP-3

GROUND ELEVATION 380'+/- MSL LOGGED BY GJS

METHOD OF EXCAVATION JD-510 Backhoe

LOCATION See Plate One



SCALE 1" = 4'

DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						CL-SM	TOPSOIL/COLLUVIUM: Dark brown, damp, firm to stiff, sandy CLAY and clayey, silty SAND.
2							LINDAVISTA FORMATION: Yellow-brown, damp, dense, silty, medium to fine SAND with rounded gravel and small cobbles. Not observed on south side of test pit.
4							
6							SCRIPPS FORMATION: Yellow-brown to light brown, damp, stiff to hard SILTSTONE with interbeds of gray siltstone and yellow-brown, silty, fine sand and brown clay.
8							
10							Total Depth 10.5' No Caving No Ground Water Encountered Backfilled 2/8/91
12							
14							
16							
18							
20							
22							

Ninyo & Moore

TEST PIT LOG

Eastgate Mall Site
North City Reclamation Plant and
Sludge Processing Facilities

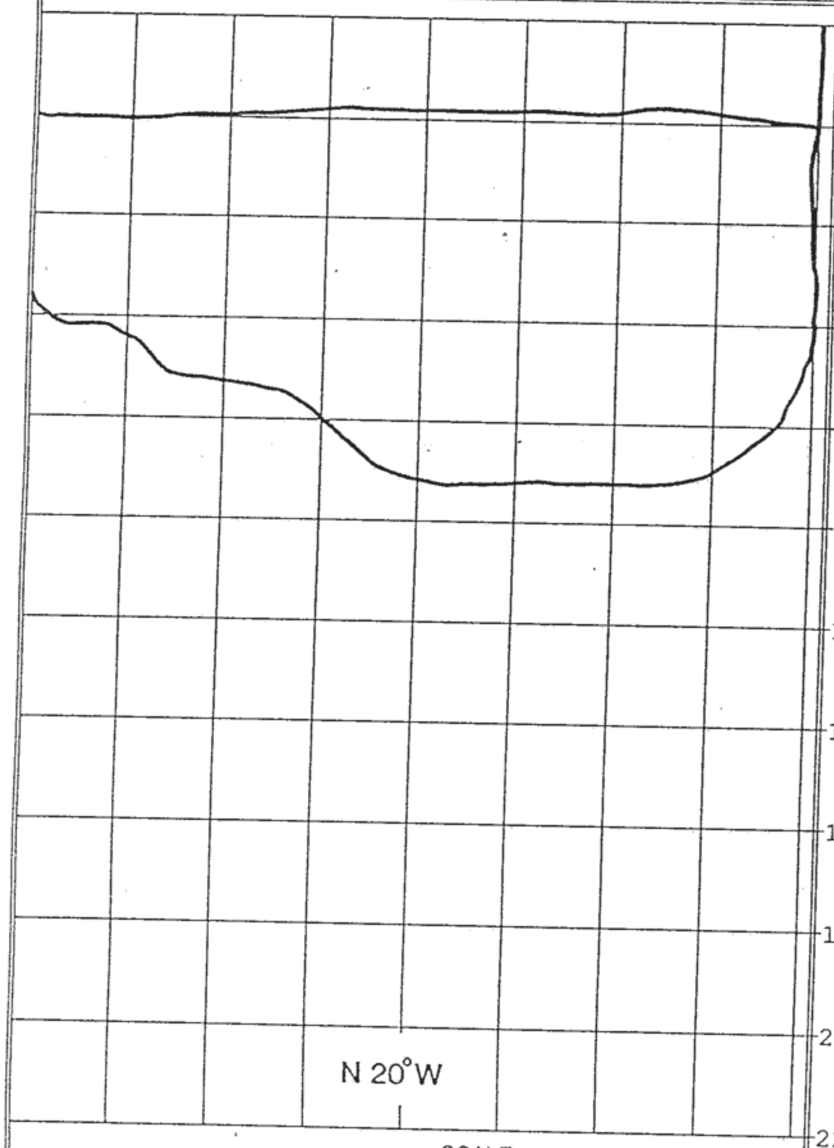
PROJECT NO.	DATE
101948-01	3/91

DATE EXCAVATED 2/8/91 TEST PIT NO. TP-4

GROUND ELEVATION 323'+/- MSL LOGGED BY GJS

METHOD OF EXCAVATION JD-510 Backhoe

LOCATION See Plate One



SCALE 1" = 4'

DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM-CL	TOPSOIL/COLLUVIUM: Dark brown, damp, medium dense to dense, clayey, silty SAND and firm to stiff, sandy CLAY.
2							SCRIPPS FORMATION: Yellow-brown, damp, very stiff, fine, sandy SILTSTONE.
4							
6							Yellow-brown to light brown, damp, hard SILTSTONE with interbeds of gray silt and yellow-brown, silty, fine sand.
8							
10							Total Depth 9.0' No Caving No Ground Water Encountered Backfilled 2/8/91
12							
14							
16							
18							
20							
22							

ENGINEERING, INC.
GEOTECHNICAL CONSULTANTS

August 26, 2016

MWH/Brown and Caldwell
Pure Water Team
9665 Chesapeake Drive, Suite 201
San Diego, California 92123

Transmitted via e-mail: LSkutecki@BrwnCald.com
MJayakumar@BrwnCald.com

Subject: **Addendum - Preliminary Geotechnical Investigation**
Predesign – North City Plant Upgrades
Proposed North City Advanced Water Purification Facility
San Diego, California
K2 Engineering Job No. G2015007-2.3

As requested, we are pleased to present this addendum letter for the proposed North City Advanced Water Purification Facility in San Diego, California. We previously performed a geotechnical investigation at the site and presented the results in a report dated June 2, 2016.

The information in this letter represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. *No other warranty, express or implied, is made as to the professional advice included in this letter*

As part of the preliminary geotechnical investigation, we perfomed percolation tests at the Advanced Water Purification Facility (AWPF) site and presented the results in the June 2, 2016 report. The infiltration rates obtained were classified and presented following the United States Department of Agriculture (USDA) criteria which does not provide a classification for infiltration rates of less than 0.06 inches per hour. The purpose of this letter is to provide a site classification following the guidelines of the City of San Diego.

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According to the City of San Diego guidelines, sites may be classified as follows:

> *Partial Infiltration:* => 0.01 in/hr. to 0.50 in/hr. (treatment control with DCV reduction; 9% to 40% depending on infiltration rate)
>
> *No Infiltration:* < 0.01 in/hr. (treatment control with 3% reduction through dispersion/evapotranspiration; 3% of impervious area within DMA)

The tests performed at the site indicate that the soils do not percolate, as such it may be classified as a *"No Infiltration"* site. The results of the tests are presented below.

Percolation Test Results

Test Number	Percolation Rate (min/inch)	Assumed Permeability (in/hr)(1)
P-1	Did not percolate	<0.01
P-2	Did not percolate	<0.01
P-3	Did not percolate	<0.01

- o0o -

We trust this letter provides you with the information you require at this time. Should you have any questions regarding the information presented, please do not hesitate to call.

Sincerely,

K2 ENGINEERING, INC.

Susana C. Kemmerrer

Susana Kemmerrer, RGE 2287
President

REGISTERED PROFESSIONAL ENGINEER
SUSANA KEMMERRER
No. 2287
Exp. 6/30/2017
GEOTECHNICAL
STATE OF CALIFORNIA

K2 ENGINEERING, INC.
GEOTECHNICAL CONSULTANTS

May 16, 2017

MWH/Brown and Caldwell
Pure Water Team
9665 Chesapeake Drive, Suite 201
San Diego, California 92123

Transmitted via e-mail: Jeff.Schulz@mwhglobal.com

Subject: **Addendum/Response to Comments -**
Predesign – North City Plant Upgrades
Proposed North City Advanced Water Purification Facility
San Diego, California
K2 Engineering Job No. G2015007-2.3

Dear Mr. Schulz:

We are pleased to present this supplemental letter to provide supplemental geotechnical information for the proposed North City Advanced Water Purification Facility in San Diego, California. In addition, this letter presents our response to the review comments by City of San Diego Development Services Department as provided in the e-mail dated May 4, 2017 by Ms. Keli Balo. For ease of reference, the review comments followed by our response(s) are presented below.

We previously performed a geotechnical investigation and presented the results in a report dated June 2, 2016. Based on the results of the investigation, preliminary recommendations for earthwork and for design of foundations and paving were provided. The supplemental information presented in this letter is based on the information gathered during our investigation and the review of published information. No additional laboratory tests were performed for this letter.

The information in this letter represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

***Comment 1**:* *Show location of the Torrey Pines fault on the site specific geologic map, if located within the project area.*

Response: The approximate location of the Torrey Pines fault was noted in Plate 3 in the report (Map Key symbol --?—f--). As requested, we have added the fault name to the map and it is presented in the attached Plate 3, Local Geology.

Comment 2: *If faulting is present within the project area, clarify the activity of faulting and how that was determined*

Response: As noted in the June 2, 2016 report, a strand of the Torrey Pines Fault has been mapped as crossing the site in a northwest to southeast direction. The Torrey Pines Fault has been classified "inactive, potentially active, presumed inactive or activity unknown" (City of San Diego Seismic Study Map No. 34). Studies by Ziony (1973) documents displacement along the Torrey Pines Fault of Eocene stratigraphy but not within the Quaternary Lindavista Formation.

Comment 3: *Clarify if the geologic structure of the site is favorable or adverse with respect to slope stability.*

Response: As discussed in Section 5.1.4 of the June 2, 2016 report, the geologic structure is favorable with respect to slope stability. Field observations of the west-facing slope indicated no adverse bedding. Furthermore, the site is located on generally flat to gently sloping terrain that have been identified as having minimal to moderate risk (City of San Diego Seismic Study, 2008) for slope failure.

Comment 4: *The project's geotechnical consultant indicates the site is subject to shallow slope instability. The consultant should provide recommendations to mitigate the potential for slope instability related to the proposed development*

Response: Preliminary recommendations to mitigate and control erosion were provided in Section 6.8 of the June 2, 2016 report.

Comment 5: *Clarify if the proposed project will destabilize or result in settlement of existing improvements.*

Response: No existing improvements were present at the AWPF site at the time we completed the preliminary investigation. As such, no impact on existing improvements is anticipated within the AWPF site. Furthermore, based on plans available at the time the report was written, it is our understanding that the proposed facilities will not be located in close proximity to improvements located on adjacent properties or easements.

We trust this letter provides you with the information you require at this time. Should you have any questions regarding the information presented, please contact the undersigned.

Sincerely,

K2 ENGINEERING, INC.

Susana C. Kemmerer

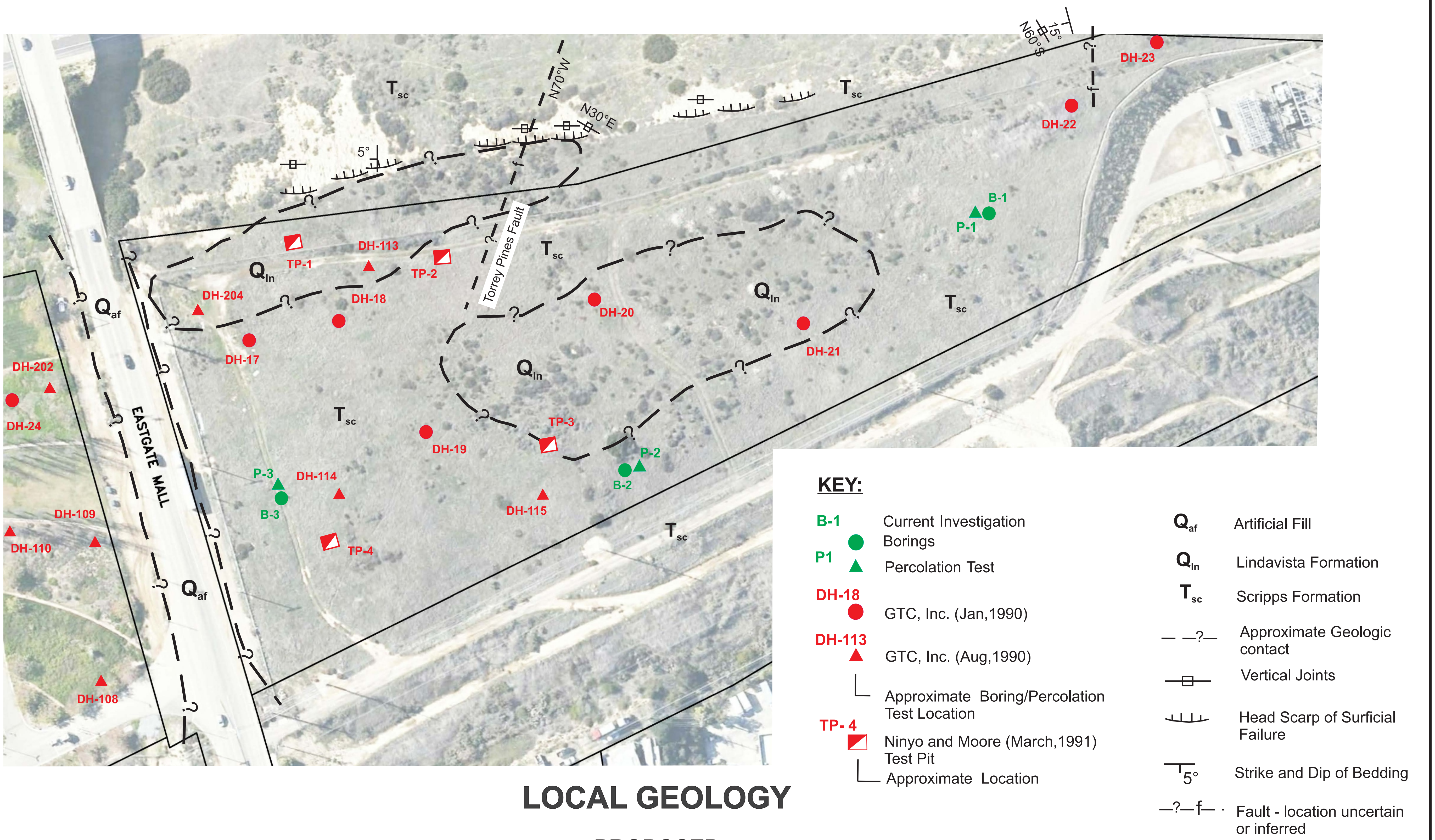
Susana Kemmerer, RGE 2282
President

REGISTERED PROFESSIONAL ENGINEER
SUSANA KEMMERRER
No. 2287
Exp. 6/30/2017
GEOTECHNICAL
STATE OF CALIFORNIA

Attachments: Plate 3 – Local Geology

Reports/MWH/G2015007-2.3 Supp Response to Comments 5-16-17.doc

JOB G2015007-2.3 DATE 5/31/2016 BY SCK/VG ENGR REV



LOCAL GEOLOGY

PROPOSED
NORTH CITY
ADVANCED WATER PURIFICATION FACILITY
SAN DIEGO, CALIFORNIA

0 50 100
SCALE

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REFERENCE: Aerial Photography (undated) provided by Brown and Caldwell

K2 ENGINEERING, INC.