GEOTECHNICAL INVESTIGATION REPORT

Bridge Widening
Cactus Road Over the Arizona Canal
51st Avenue & Cactus Road
Maricopa County, Arizona

SHB Job No. E83-97

Consulting Geotechnical Engineers

PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

901,004

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING

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August 22, 1983

Hoffman-Miller Engineers, Inc.
Consulting Engineers
3737 East Indian School Road
Suite 401
Phoenix, Arizona 85018

SHB Job No. E83-97

Attention: Lloyd W. Miller, P.E.

Re: Bridge Widening
Cactus Road Over the Arizona Canal
51st Avenue & Cactus Road
Maricopa County, Arizona

Gentlemen,

Our Geotechnical Investigation Report on the referenced project is herewith submitted. The report includes the results of test drilling, laboratory analysis and recommended criteria for foundation design.

Should any questions arise concerning this report, we would be pleased to discuss them with you.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By Norman H. Wetz
Norman H. Wetz, P.E.

Reviewed by George H. Beckwith
George H. Beckwith, P.E.

Registered Professional Engineer (Civil) Certificate No. 12176 Norman Harold Wetz Date Signed 8-22-83 Arizona, U.S.A.

Registered Professional Engineer (Civil) Certificate No. 2633 George Henry Beckwith Date Signed 8-22-83 Arizona, U.S.A.

Copies: Addressee (3)

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REPORT

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1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed bridge widening of existing Cactus Road over the Arizona Canal. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site to provide recommendations for foundation and abutment design.

2. PROPOSED CONSTRUCTION

Preliminary details of the proposed construction were provided by Lloyd W. Miller, P.E., of Hoffman-Miller Engineers, Inc.

It is understood that an existing bridge at Cactus Road over the Arizona Canal will be widened on either side to achieve a clear distance of 68 feet with an additional 5 feet for a sidewalk on one side. The existing bridge is 32 feet wide and is a two-span bridge with span lengths of 30 feet each.

Should details involved in final design vary significantly from those as outlined, this firm should be notified for review and possible revision of recommendations.

3. INVESTIGATION

3.1 Subsurface Exploration

Four exploratory borings were drilled to depths of about 30 to 36 feet below existing grade. Standard penetration testing and open-end drive sampling were performed at selected intervals in the borings. The results of the field investigation are presented in Appendix A, which includes a brief description of drilling and sampling equipment and procedures, a site plan showing the boring locations, and logs of the test borings. The field investigation was supervised by J. David Deatherage, P.E., staff engineer of this firm.

3.2 Laboratory Analysis

Moisture content determinations were made on selected tube samples recovered, while dry densities were determined for the 2.42-inch diameter open-end drive samples. The results of these tests are shown on the boring logs.

Grain-size analysis, Atterberg Limits and direct shear tests were performed on selected samples. The results of these tests are presented in Appendix B, along with a brief description of laboratory testing procedures.

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

As previously mentioned, there is an existing two-lane highway bridge over the Arizona Canal. The canal is lined with Gunite along the sides and bottom beneath the present bridge.

4.2 Geotechnical Profile

A surface layer of man-made fill consisting of silty sands extends about 2½ to 5 feet below existing grade. These soils are soft to firm. Sandy and silty clays, and clayey sands with lesser amounts of silty sands underlie the man-made fill and extend the full depths of the borings. These soils are soft to moderately firm near the surface and become very firm to hard with depth. Also, the materials are weakly cemented near the surface and become moderately to strongly cemented with depth.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

The existing structure has been in service for some time now and settlements probably have already taken place in the structure. It is concluded that the near surface soils are somewhat compressible and could lead to excessive differential settlements between the existing structure and the proposed new portions of the bridge. Therefore, in order to reduce differential settlements, it is recommended that footings be founded on materials at greater depth. Straight, machine-cleaned, drilled, cast-in-place concrete piers are recommended for the support of the structure. Design criteria for drilled piers are given in Section 5.2.

5.2 Straight, Machine-Cleaned, Drilled, Cast-In-Place Concrete Piers

5.2.1 Downward Loads

Structural loads may be supported on straight, machine-cleaned, drilled, cast-in-place concrete piers and should extend at least 15.0 feet below finished grade. Design Chart 1 in Appendix C provides safe downward capacities versus depth for drilled piers.

Capacities apply to full dead plus live loads. A one-third increase is recommended when considering wind or seismic forces. A minimum shaft diameter of 1.5 feet is recommended for drilled foundations.

5.2.2 Estimated Settlements

It is estimated that settlements of pier foundations designed and constructed in accordance with criteria presented herein will not exceed ½ inch.

Settlements can be expected to occur rapidly, with the major portion being complete at the end of construction.

5.2.3 Resistance to Lateral Loads

Methods based on the theory of a beam-on-an-elastic half-space are recommended for the computation of soil reactions, deflections, moments and shears at the design loads. In that it is anticipated that the foundations will meet the criteria for rigid piers, the Prakash method is recommended as one approach for use in analysis. This method is described in concise form on page 79 of Woodward, et al*. A K_L of 5,000 psi and an n of 0.15 are recommended for the application of this method.

Design Chart 2, shown in Appendix C, presents the estimated ultimate lateral soil bearing pressure versus depth. In order to insure an adequate factor of safety, it is recommended that the soil reactions determined by the Prakash method be limited to 50 percent of the ultimate soil bearing pressures shown on the chart.

Criteria given above apply to isolated piers spaced no closer than 3 diameters on center perpendicular to the line of thrust and 6 diameters on center parallel to the line of thrust.

5.2.4 Cleaning of Drilled Pier Excavations

Straight drilled pier excavations should be advanced

*Woodward, R.J., Gardner, W.S., and Greer, D.M., "Drilled Pier Foundations", McGraw-Hill Book Company, 1972.

with a single flight auger, or bucket auger bits, to the design depth. It should be verified by observations and measurement that excavations are open to that depth. The auger should be placed back in the holes and two additional passes made to clean loose material present in the bottom of the holes.

5.2.5 Placement of Concrete

Concrete should be placed through a hopper or other device approved by the geotechnical engineer so that it is channeled in such a manner to free fall and clear the walls of the excavation and reinforcing steel until it strikes the bottom. Adequate compaction will be achieved by free fall of the concrete up to the top 5.0 feet. The top 5.0 feet of concrete should be vibrated in order to achieve proper compaction. The concrete should be designed, from a strength standpoint, so that the slump during placement is in the range of 4 to 6 inches.

5.2.6 Quality Assurance & Construction

Continuous observations of the construction of drilled piers should be carried out by a representative of the geotechnical engineer. He should confirm proper diameter, depth and cleaning, and should also confirm the nature of the materials encountered in the pier excavations. Concrete placement should be continuously observed to confirm that it meets requirements.

A quality assurance report should be submitted on each pier stating, in writing, that all details have been confirmed and meet requirements.

It appears that drilled pier excavations can be advanced to the depths recommended with little or no caving. Therefore, concrete quantities are expected to be very near the neat volumes indicated by the plans.

5.3 Abutment Wall Design Criteria

Free draining granular backfill should be utilized behind the abutments and in roadway approach fills. This material should consist of sand and gravel and have no more than 8 percent passing the no. 200 sieve. This material should be nonplastic when tested in accordance with ASTM D422 and D423. Compaction of the fill should be at least 98 percent of maximum density as determined by ASTM D1557.

The earth pressures against abutments will depend upon the degree of restraint. Rigid, absolutely restrained abutments would be subjected to earth pressures represented by a hydrostatic load diagram of about 50 pounds per square foot per foot of depth. Rotation or lateral translation of the walls equal to about 0.001 times the height would reduce earth pressures to the active state of about 30 pounds per square foot per foot of depth.

Slight lateral translation equal to about 0.0005 times the height would result in an intermediate pressure diagram on the order of 40 pounds per square foot per foot of depth.

Drilling Equipment Truck-mounted CME-55 drill rigs powered with 4 or 6 cylinder Ford industrial engines are used in advancing test borings. The 4 cylinder and 6 cylinder engines are capable of delivering about 4,350 and 6,500 foot/pounds torque to the drill spindle, respectively. The spindle is advanced with twin hydraulic rams capable of exerting 12,000 pounds downward force. Drilling through soil or softer rock is performed with 6 1/2 O.D., 3 1/4 I.D. hollow stem auger or 4 1/2 inch continuous flight auger. Carbide insert teeth are normally used on the auger bits so they can often penetrate rock or very strongly cemented soils which require blasting or very heavy equipment for excavation. Where refusal is experienced in auger drilling, the holes are sometimes advanced with tricone gear bits and NX rods using water or air as a drilling fluid. Where auger and tricone gear bits cannot be used to advance the hole due to cobbles or caving conditions, the ODEX (overburden drilling with the eccentric method) is used. A percussion down-the-hole hammer underreams the hole and 5 inch steel casing is introduced into the hole during drilling. The drill bit is eccentric and can be removed from the center of the casing to allow sampling of the material below the bit penetration depth.

Sampling Procedures Dynamically driven tube samples are usually obtained at selected intervals in the borings by the ASTM D1586 procedure. In many cases, 2" O.D., 1 3/8" I.D. samplers are used to obtain the standard penetration resistance. "Undisturbed" samples of firmer soils are often obtained with 3" O.D. samplers lined with 2.42" I.D. brass rings. The driving energy is generally recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the samplers in 6 inch increments. However, in stratified soils, driving resistance is sometimes recorded in 2 or 3 inch increments so that soil changes and the presence of scattered gravel or cemented layers can be readily detected and the realistic penetration values obtained for consideration in design. These values are expressed in blows per foot on the logs. "Undisturbed" sampling of softer soils is sometimes performed with thin walled Shelby tubes (ASTM D1587). Where samples of rock are required, they are obtained by NX diamond core drilling (ASTM D2113). Tube samples are labeled and placed in watertight containers to maintain field moisture contents for testing. When necessary for testing, larger bulk samples are taken from auger cuttings.

Continuous Penetration Tests Continuous penetration tests are performed by driving a 2" O.D. blunt nosed penetrometer adjacent to or in the bottom of borings. The penetrometer is attached to 1 5/8" O.D. drill rods to provide clearance to minimize side friction so that penetration values are as nearly as possible a measure of end resistance. Penetration values are recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the penetrometer in one foot increments or less.

Boring Records Drilling operations are directed by our field engineer or geologist who examines soil recovery and prepares boring logs. Soils are visually classified in accordance with the Unified Soil Classification System (ASTM D2487) with appropriate group symbols being shown on the logs.

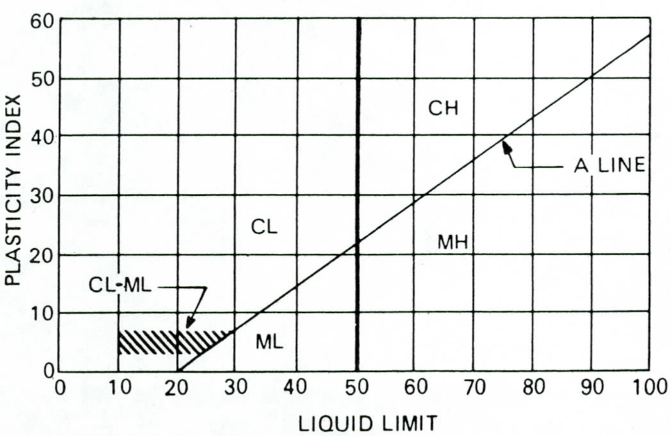
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified by the Unified Soil Classification system on the boring logs presented in this report. Grain-size analysis and Atterberg Limits Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, see "The Unified Soil Classification System" Corp of Engineers, US Army Technical Memorandum No. 3-357 (Revised April 1960) or ASTM Designation: D2487-66T.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 seive)			SW	Well graded sands, gravelly sands.
					SP	Poorly graded sands, gravelly sands.
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS LIMITS PLOT BELOW "A" LINE & HATCHED ZONE ON PLASTICITY CHART	SILTS OF LOW PLASTICITY (Liquid Limit Less Than 50)			ML	Inorganic silts, clayey silts with slight plasticity.
		SILTS OF HIGH PLASTICITY (Liquid Limit More Than 50)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
	CLAYS LIMITS PLOT ABOVE "A" LINE & HATCHED ZONE ON PLASTICITY CHART	CLAYS OF LOW PLASTICITY (Liquid Limit Less Than 50)			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		CLAYS OF HIGH PLASTICITY (Liquid Limit More Than 50)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity.

NOTE: Coarse grained soils with between 5% & 12% passing the No. 200 sieve and fine grained soils with limits plotting in the hatched zone on the plasticity chart to have double symbol.

PLASTICITY CHART



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Cobbles	Above 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to ¾ in.
Fine gravel	¾ in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

TERMINOLOGY USED TO DESCRIBE THE RELATIVE DENSITY, CONSISTENCY OR FIRMNESS OF SOILS

The terminology used on the boring logs to describe the relative density, consistency or firmness of soils relative to the standard penetration resistance is presented below. The standard penetration resistance (N) in blows per foot is obtained by the ASTM D1586 procedure using 2" O.D., 1 3/8" I.D. samplers.

1. Relative Density. Terms for description of relative density of cohesionless, uncemented sands and sand-gravel mixtures.

N	Relative Density
0-4	Very loose
5-10	Loose
11-30	Medium dense
31-50	Dense
50+	Very dense

2. Relative Consistency. Terms for description of clays which are saturated or near saturation.

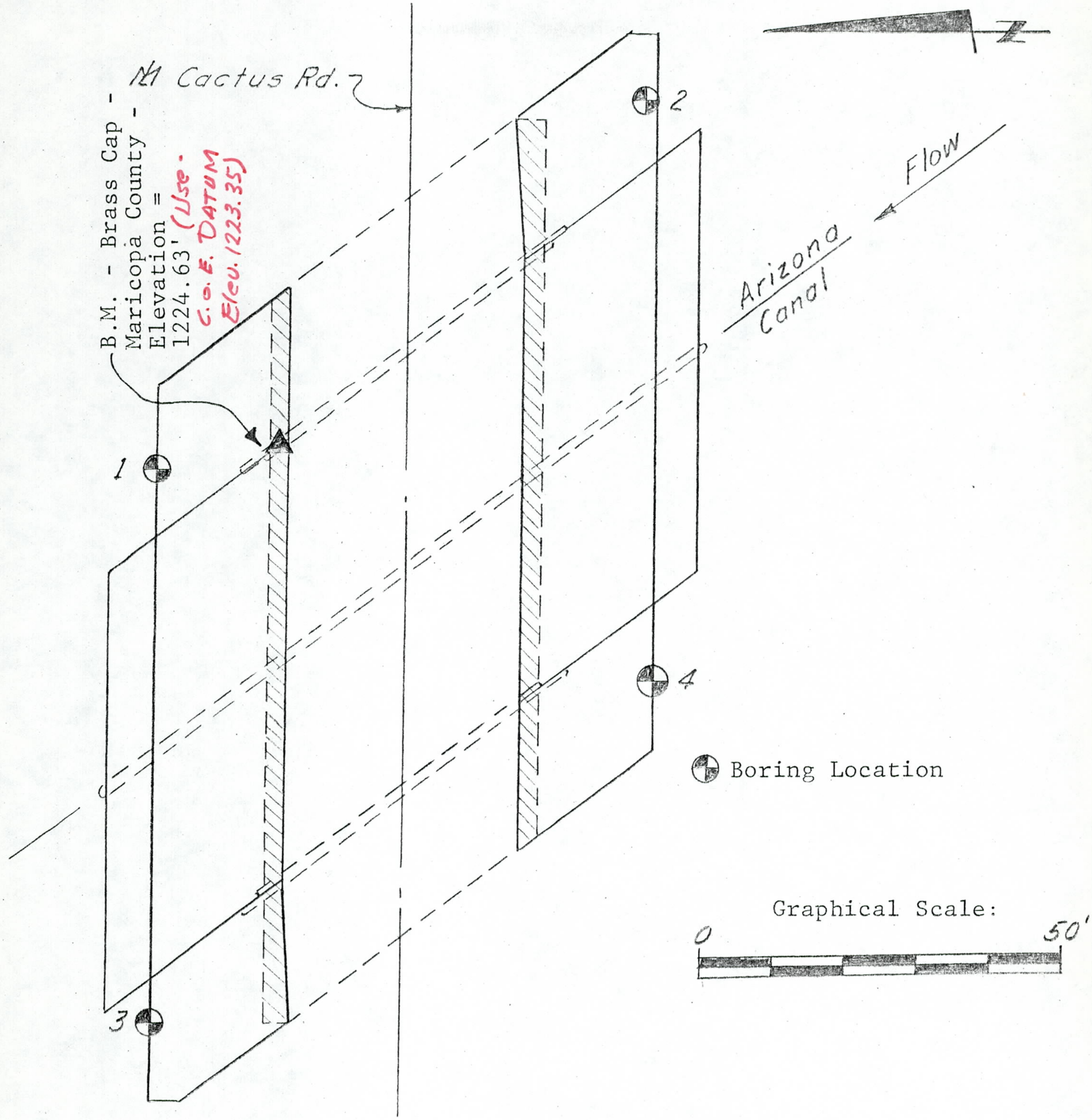
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
16-30	Very stiff	Readily indented with thumbnail.
30+	Hard	Indented only with difficulty by thumbnail.

3. Relative Firmness. Terms for description of partially saturated and/or cemented soils which commonly occur in the Southwest including clays, cemented granular materials, silts and silty and clayey granular soils.

N	Relative Firmness
0-4	Very soft
5-8	Soft
9-15	Moderately firm
16-30	Firm
31-50	Very firm
50+	Hard

SITE PLAN

SHOWING LOCATIONS OF TEST BORINGS



Reference Drawing: "Location Plan - Cactus Rd. Bridge Widening Over Arizona Canal" by Hoffman-Miller Engineers, Phoenix, Arizona, undated

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51st Avenue & Cactus Road
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CONSULTING SOIL AND FOUNDATION ENGINEERS
PHOENIX • TUCSON • ALBUQUERQUE

PROJECT Bridge Widening Cactus Road Over the Arizona Canal

LOG OF TEST BORING NO. 1

JOB NO. E83-97 DATE 8-2-83

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 1223.07'
DATUM See Site Plan

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	8			SM-SP		Man-made FILL
									slightly moist moderately firm	SILTY SAND, predominantly fine, low plasticity, brown
5			X	S	9			CH	moist moderately firm	SANDY CLAY, medium to high plasticity, brown
10			X	S	31				moist firm to hard	SANDY CLAY, some gravel, weakly to moderately cemented, medium plasticity, light brown to brown
15			X	S	24					
20			X	S	46			CL		
25			X	S	80					
30			X	S	59					
35			X	S	28			SM	moist firm	SILTY SAND, well graded, subrounded, weakly cemented, low plasticity, light brown
40										Stopped auger at 34'6" Stopped sampler at 36'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
U – 3" O.D. 2.42" I.D. tube sample.
T – 3" O.D. thin-walled Shelby tube.

SERGENT, HAUSKINS & BECKWITH
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PROJECT Bridge Widening Cactus Road Over the Arizona Canal

JOB NO. E83-97 DATE 8-2-83

LOG OF TEST BORING NO. 2

RIG TYPE CME-55

BORING TYPE $6\frac{1}{2}$" Hollow Stem Auger

SURFACE ELEV. 1223.49'

DATUM See Site Plan

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	18			SP-SM	dry firm	Man-made FILL SILTY SAND, predominantly fine, low plasticity, brown
5			X	S	6			CL	very moist soft	SILTY CLAY, medium plasticity, brown
10			X	S	21			CL	moist firm	SANDY CLAY, low plasticity, light brown to brown
15			X	S	35			CL	moist very firm to hard	SANDY CLAY, low plasticity, light brown note: weakly to moderately cemented
20			X	S	68					
25			X	S	80					
30			X	S	60			SC	moist hard	CLAYEY SAND, some gravel, subrounded, low plasticity, light brown
35										Auger refused at 32'6"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Bridge Widening Cactus Road Over the Arizona Canal

LOG OF TEST BORING NO. 3

JOB NO. E83-97 DATE 8-2-83

RIG TYPE CME-55

BORING TYPE $6\frac{1}{2}$" Hollow Stem Auger

SURFACE ELEV. 1223.00'

DATUM See Site Plan

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0			X	S	27			
5			X	S	17			SC
10			X	U	31	104	13	
15			X	S	48			
20			X	U	72	95	19	
25			X	S	59			CL
30			X	S	47			
35			X	S	68			
40								

REMARKS	VISUAL CLASSIFICATION
slightly moist firm	CLAYEY SAND, some gravel, well graded, low plasticity, brown
slightly moist very firm to hard	SANDY CLAY, weakly to moderately cemented, low to medium plasticity, brown to light brown
	Stopped auger at 34'6" Stopped sampler at 36'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT Bridge Widening Cactus Road Over the Arizona Canal

JOB NO. E83-97 DATE 8-2-83

LOG OF TEST BORING NO. 4

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1222.98'

DATUM See Site Plan

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	23 (no recovery)			SM	slightly moist firm	Man-made FILL SILTY SAND, some gravel, well graded, subrounded, low plasticity, brown
5			X	S	11			CL	slightly moist moderately firm	SANDY CLAY, some gravel, low plasticity, brown
10			X	S	28			SC	slightly moist firm	CLAYEY SAND, some gravel, well graded, low plasticity, brown
15			X	S	35		19	CH	slightly moist very firm	SANDY CLAY, moderately to weakly cemented, medium to high plasticity, brown
20			X	S	50					
25			X	S	85			CL	slightly moist hard	SANDY CLAY, moderately cemented, medium plasticity, tan to brown
30			X	S	50/4"			SC	slightly moist hard	CLAYEY SAND, some gravel, predominantly fine, low plasticity, brown
35										Stopped auger at 29'6" Sampler refused at 30'4"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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LABORATORY TESTING PROCEDURES

Consolidation Tests Soiltest or Clockhouse apparatus of the "floating-ring" type are employed for the one-dimensional consolidation tests. They are designed to receive one inch high 2.5 inch O.D. brass liner rings with soil specimens as secured in the field. Procedures for the tests generally are those outlined in ASTM D2435. Loads are applied in several increments to the upper surface of the test specimen and the resulting deformations are recorded at selected time intervals for each increment. For soils which are essentially saturated, each increment of load is maintained until the deformation versus log of time curve indicates completion of primary consolidation. For partially saturated soils, each increment of load is maintained until the rate of deformation is equal or less than 1/10,000 inch per hour. Applied loads are such that each new increment is equal to the total previously applied loading. Porous stones are placed in contact with the top and bottom of the specimens to permit free addition or expulsion of water. For partially saturated soils, the tests are normally performed at in situ moisture conditions until consolidation is complete under stresses approximately equal to those which will be imposed by the combined overburden and foundation loads. The samples are then submerged to show the effect of moisture increase and the tests continued under higher loadings. Generally, the tests are continued to about twice the anticipated curve due to overburden and structural loads with a rebound curve then being established by releasing loads.

Expansion Tests The same type of consolidometer apparatus described above is used in expansion testing. Undisturbed samples contained in brass liner rings are placed in the consolidometers, subjected to appropriate surcharge loads and submerged. The loads are maintained until the expansion versus log of time curve indicates the completion of "primary swell".

Direct Shear Tests Direct shear tests are run using a Clockhouse or Soiltest apparatus of the strain-control of approximately 0.05 inches per minute. The machine is designed to receive one of the one inch high 2.42 inch diameter specimens obtained by tube sampling. Generally, each sample is sheared under a normal load equivalent to the effective overburden pressure at the point of sampling. In some instances, samples are sheared at several normal loads to obtain the cohesion and angle of internal friction. When necessary, samples are saturated and/or consolidated before shearing in order to approximate the anticipated controlling field loading conditions.

TABULATION OF TEST RESULTS

Job No. E83-97

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#200 / .75"	#100 / 1"	#50 / 1.5"	#40 / 2"	#30 / 2.5"	#16 / 3"	#10 / 3.5"	#8 / 4"	#4 / 6"	.25" / 8"	.375" / 10"	.5" / 12"	LAB NO
					SIEVE ANALYSIS-ACCUM % PASSING												
4	14.5'-16'	CH	54	29	74.7	81	86	88	90	93	95	95	97	98	99	100	3-97-27

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ENGINEERING ANALYSIS • PHYSICAL TESTING • QUALITY CONTROL • FIELD EXPLORATION

REPORT ON LABORATORY TESTS

DATE ____________

PROJECT Bridge Widening Cactus Road over the Arizona Canal JOB NO. E83-97

LOCATION 51st Avenue & Cactus Road; Maricopa County LAB NO. 3-97-18 & 20

SAMPLE See Below - One point test @ sample depths noted

In Situ DIRECT SHEAR TESTS

(#18) In Situ - Point No. 1 (= ± 1.00 KSF) #3 @ 9½'-10½'

Initial Moisture Content	12.8 %
Dry Density (PCF)	103.5
Submerged	
Final Moisture Content	- %
Maximum Vertical Deformation @ T Max.	(+)0.021 Inches
Shearing Stress, T Max.	1.59 KSF

(#20) In Situ - Point No. 2 (= ± 2.06 KSF) #3 @ 19½'-20½'

Initial Moisture Content	19.1 %
Dry Density (PCF)	95.1
Submerged	
Final Moisture Content	- %
Maximum Vertical Deformation @ T Max.	(+)0.015 Inches
Shearing Stress, T Max.	3.88 KSF

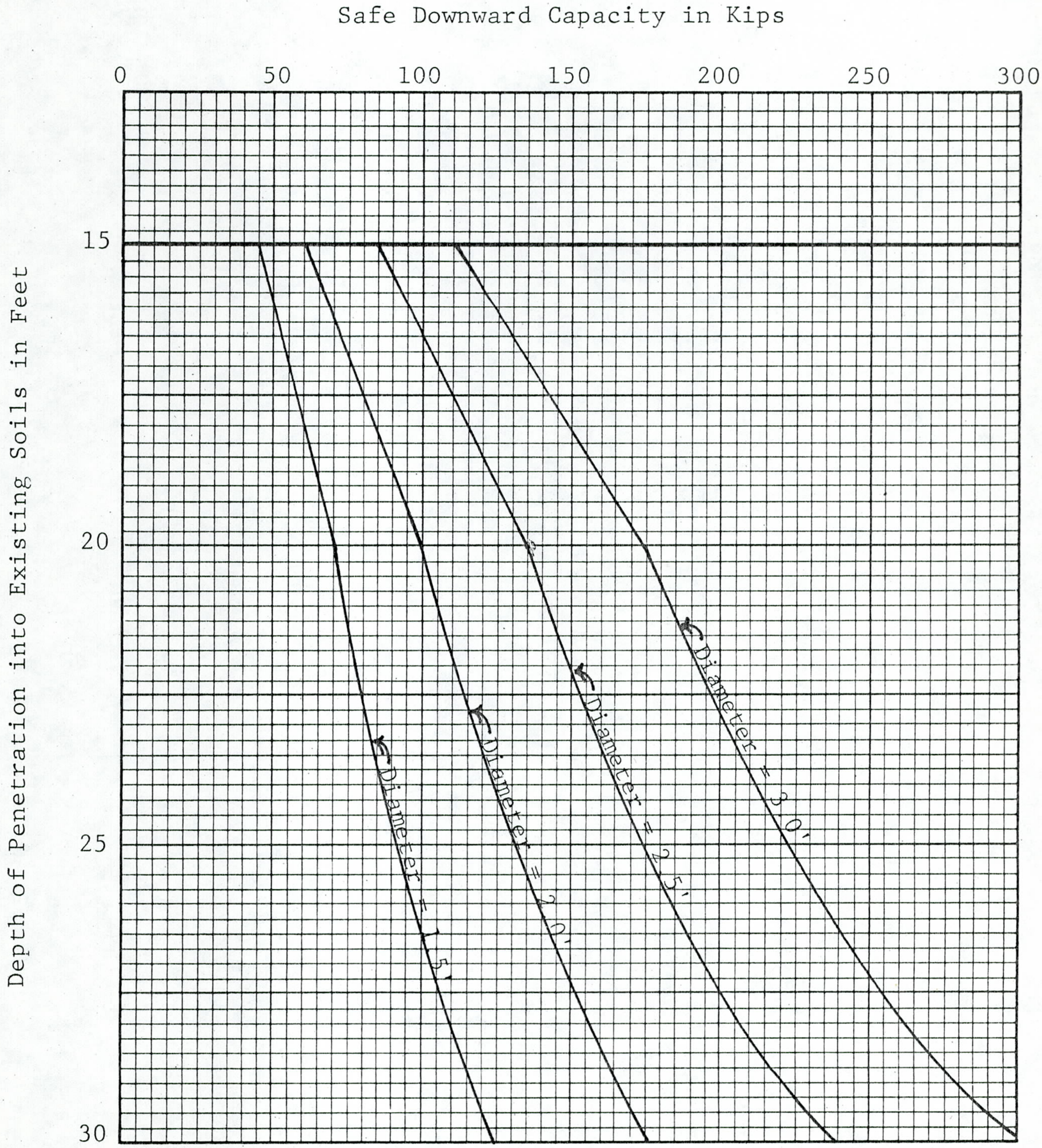
In Situ - Point No. 3 (= ± ______ KSF)

Initial Moisture Content	______ %
Dry Density (PCF)	______
Submerged	
Final Moisture Content	______ %
Maximum Vertical Deformation @ T Max.	______ Inches
Shearing Stress, T Max.	______ KSF

PROJECT Bridge Widening
Cactus Road Over the Arizona Canal JOB NO. E83-97

DESIGN CHART 1

SAFE DOWNWARD CAPACITIES OF STRAIGHT,
DRILLED, CAST-IN-PLACE CONCRETE PIERS



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PROJECT Bridge Widening Cactus Road Over the Arizona Canal JOB NO. E83-97

DESIGN CHART 2

ULTIMATE LATERAL SOIL BEARING PRESSURE
AGAINST DRILLED, CAST-IN-PLACE CONCRETE PIERS