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GEOTECHNICAL INVESTIGATION REPORT
Bridge Over the ACDC
29th Avenue & Arizona Canal
Phoenix, Arizona

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Date Signed 6-12-84
ARIZONA, U.S.A.

SHB Job No. E84-94

JUN 15 1984

Consulting Geotechnical Engineers
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SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY

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June 13, 1984

DMJM/Adam, Hamlyn, Anderson, Inc.
4751 North 15th Street
Phoenix, Arizona 85014

SHB Job No. E84-94

Attention: Doyle L. Wiste, P.E.

Re: Bridge Over the ACDC
29th Avenue & Arizona Canal
Phoenix, Arizona

Gentlemen:

Our Geotechnical Investigation Report on the referenced project is herewith submitted. The report includes the results of test drilling and 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, P.E.

Registered Professional Engineer CERTIFICATE NO. 12176 NORMAN HAROLD Date Signed 6-12-84 ARIZONA, U.S.A.

Reviewed by _____
Lawrence A. Hansen, Ph.D., P.E.

Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 6-12-84 ARIZONA, U.S.A.

Copies: Addressee (3)

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SHB Job No. E84-94

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Bridge Over the ACDC
29th Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E84-94

1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed 29th Avenue Bridge over the Arizona Canal Diversion Channel located in Phoenix, Arizona. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site to provide recommendations for foundation design and abutment support.

2. PROJECT DESCRIPTION

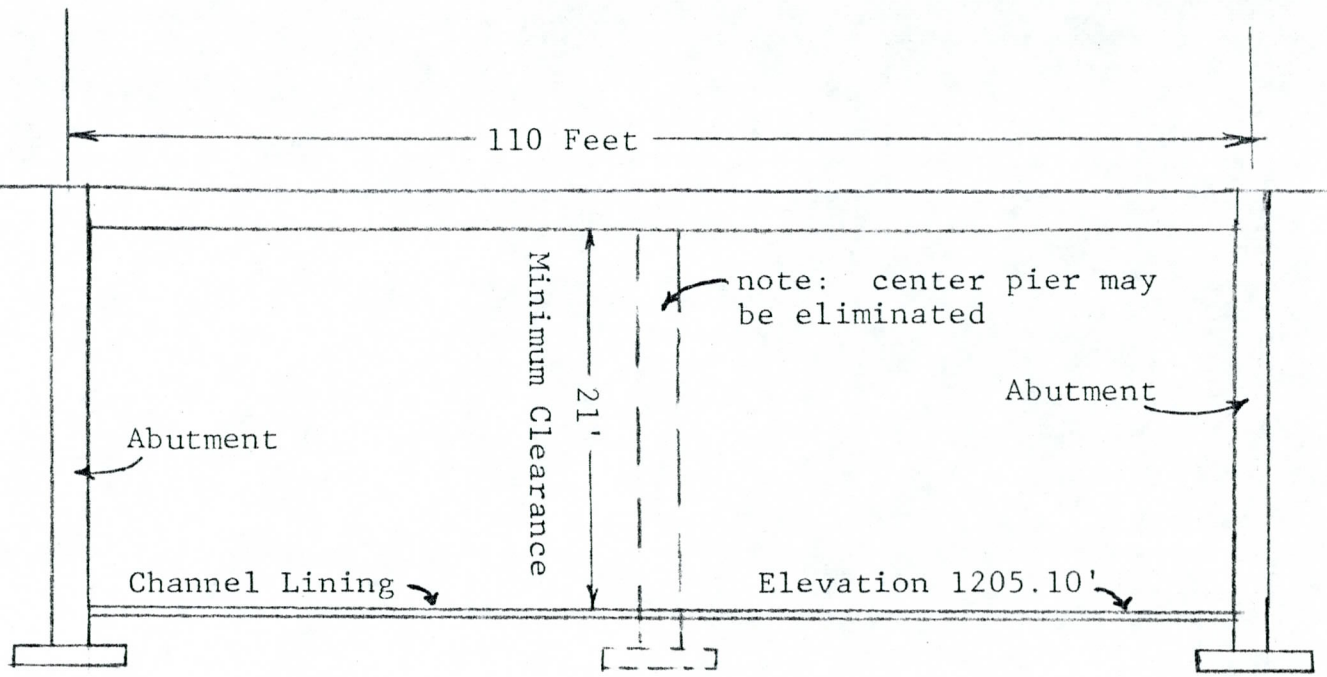
Preliminary details of the proposed construction were provided by Doyle L. Wiste, P.E., of DMJM/Adam, Hamlyn, Anderson, Inc.

It is understood that a bridge approximately 120 feet long and 50 feet wide is planned. The invert elevation of the Arizona Canal Diversion Channel at the centerline of the bridge will be 1205.10 feet (Corps of Engineeers datum). The channel will be concrete lined and rectangular in shape with a 110-foot wide bottom, as shown in Figure 1. The design flow rate is approximately 27,000 cubic feet per second at a depth of approximately 18 feet and a velocity of approximately 14 feet per second.

Should details involved in final design vary significantly from those as outlined, this firm should be

FIGURE 1

Section Along Centerline of 29th Avenue



Note to Scale

Bridge Over the ACDC
29th Avenue & Arizona Canal
Phoenix, Arizona
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notified for review and possible revision of recommendations.

3. INVESTIGATION

3.1 Subsurface Exploration

Three exploratory borings were drilled to depths of 23 to 50 feet below existing grade. The borings were performed using 6 5/8-inch O.D. hollow stem auger. Standard penetration testing and open-end drive sampling were performed at 5-foot intervals in the borings. All boreholes were maintained full of water during standard penetration testing.

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 Suang Cheng, staff engineer of this firm.

3.2 Laboratory Analysis

Moisture content determinations were made on selected samples recovered, while dry density was determined for a selected 2.42-inch drive sample. The results of these tests are shown on the boring logs.

Grain-size analysis, Atterberg Limits and consolidation

tests were performed on selected samples. The results of these tests are presented in Appendix B, along with a brief description of soil mechanics testing procedures.

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

The proposed bridge is located along 29th Avenue between the Arizona Canal located to the south and Metro Parkway located to the north. Twenty-ninth Avenue is a paved, four-lane roadway that is approximately 50 feet wide and has a curb and gutter. The site area is relatively level and void of vegetative cover. A box culvert is located on the south end of the proposed bridge site. It is approximately 6 by 7 feet in dimension and is located about 14 feet below grade.

4.2 Geotechnical Profile

The subsurface soils consist of a surface layer of sandy and silty clays, with lesser deposits of sandy silts, that extends about 13½ to 19 feet below existing grade. These soils are of low to medium plasticity and were found to be soft to firm. It is suspected that the upper few feet of this material is man-made fill that is related to the 29th Avenue road construction. Granular deposits consisting of sand and gravel with varying amounts of silt and clay underlie the surface stratum,

extending the full depths of the borings. These soils vary from medium dense to very dense, and are generally very dense to hard below elevation 1250.

4.3 Soil Moisture & Groundwater Conditions

No free groundwater was encountered in the borings and soil moisture contents were relatively low throughout the depth of the investigation. It is our opinion that in situ moisture conditions are somewhat drier than those reported. Due to maintaining the borehole full of water during standard penetration testing, increased moisture contents likely resulted. This is apparently the result of water being forced between the sample and the inner wall of the sampler.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

Spread-type footings are recommended for the support of the bridge structure. Footings should bear upon the dense sand and sand/gravel mixtures that will be exposed during construction. The concrete channel lining will protect the near surface soils from scour. Design criteria for spread-type footings are presented in Section 5.2.

Drilled pier foundations would also provide safe support of the structure, however, construction problems due to

anticipated caving and sloughing of the cleaner sands and sand/gravel mixtures should be anticipated with this approach. Design criteria for drilled pier foundations can be supplied, if desired.

5.2 Spread-Type Footings

5.2.1 Downward Loads

Spread-type footings will provide safe support of the structure. Safe soil bearing pressure for a footing bearing at a maximum elevation of 1202, which is approximately 3.0 feet below channel invert finished grade, is 13.5 ksf. Minimum footing depth does not consider scour, as erosion protection is assumed.

5.2.2 Resistance to Lateral Loads

A passive earth pressure of 300 pounds per square foot per foot of depth against footings and walls is recommended for the resistance of lateral loads. A coefficient of friction of 0.45 is recommended for computation of the lateral resistance between the base of foundations and the soil.

5.2.3 Estimated Settlements

Estimated settlements are presented in charts provided in Appendix C. Settlements are plotted as inches of settlement per kip of vertical load.

5.3 Abutment Wall Design Criteria

Free draining granular backfill should be utilized behind the abutments and roadway approach fills. This material should consist of sand and gravel, and have no more than 12 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 95 percent of maximum density as determined by ASTM D1557.

The earth pressures against the abutments would 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 the 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 intermediate pressure diagram on the order of 40 pounds per square foot per foot of depth.

TEST DRILLING EQUIPMENT & PROCEDURES

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.

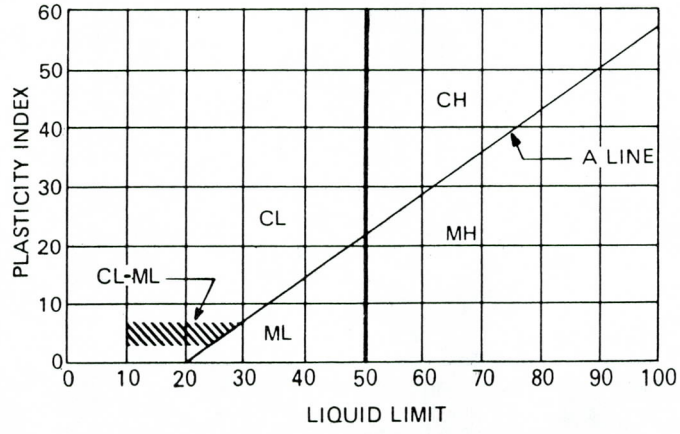
UNIFIED SOIL CLASSIFICATION SYSTEM

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.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve) — 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.
		SANDS WITH FINES (More than 12% passes No. 200 sieve) — 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.

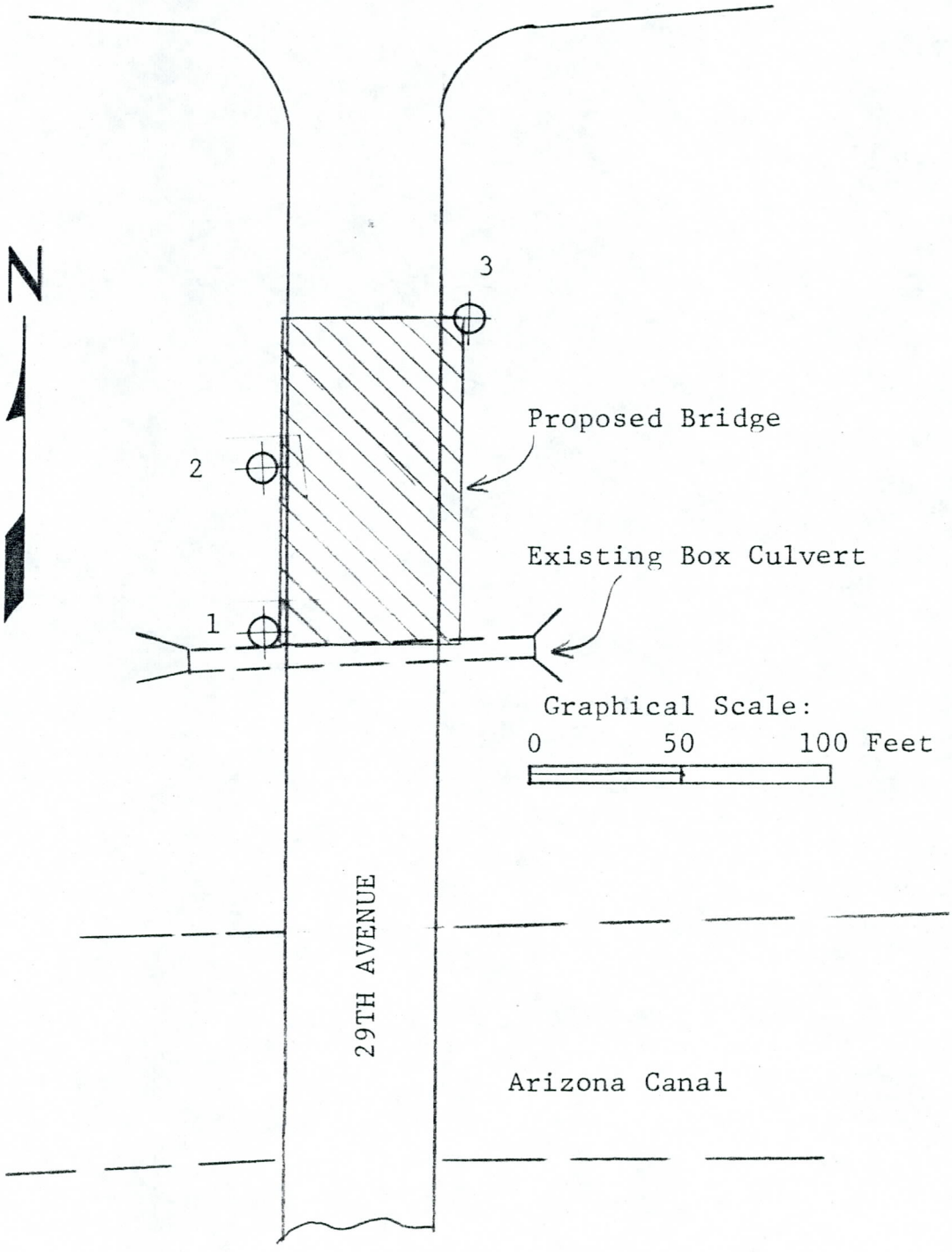
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
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: "Preprinted Site Plan" by DMJM/Adam, Hamlyn, Anderson, Inc., Phoenix, Arizona, dated May 10, 1984

Bridge Over the ACDC
29th Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E84-94

PROJECT Bridge Over the ACDC 29th Avenue & Arizona Canal

JOB NO. E84-94 DATE 5-18-84

LOG OF TEST BORING NO. 1

RIG TYPE CME-75
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 1233.96'
DATUM Corps of Engineers

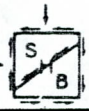
Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				S	37			CL	dry to slightly moist firm	SILTY CLAY, low to medium plasticity, brown
5				S	18			CL	slightly moist to moist firm	SANDY CLAY, some gravel, medium plasticity, brown
10				S	19				moist firm	CLAY, low to medium plasticity, dark brown
15				S	22		18	CL		
20				U	64	121	11		slightly moist hard	CLAYEY SAND & GRAVEL, occasional cobble, well graded, subangular, low to medium plasticity, brown note: thin silt lens at 34'
25				S	50/3"					
								GC		
30				S	50/5"					
35				S	50/4½"					
40										Auger refused at 37'

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.



A-5

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PROJECT Bridge Over the ACDC 29th Avenue & Arizona Canal

JOB NO. E84-94 DATE 5-18-84

LOG OF TEST BORING NO. 2

RIG TYPE CME-75
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 1232.95'
DATUM Corps of Engineers

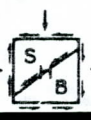
Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	S	8		15	CL	slightly moist to moist soft to moderately firm	SANDY CLAY, low plasticity, brown
5			☒	S	16		21			
10			☒	S	8		15	CL	moist soft	SANDY CLAY, low plasticity, brown
15			☒	S	22		21	ML	moist firm	SANDY SILT, low plasticity, brown
20			☒	S	30		10		slightly moist medium dense to very dense	SAND, some gravel, trace of silt, well graded, nonplastic, dark brown note: thin silt lens at 30'
25			☒	S	50/3"		17			
30			☒	S	68		8	SW-SM		
35			☒	S	95		10			
40			☒	S	50/5½"		8		slightly moist hard	SILTY SAND & GRAVEL, some clay, well graded, subangular, low plasticity, brown
45			☒	S	50/3"		8	GM-GC		Stopped auger at 49'6" Sampler refused at 49'9"
50			☒	S	50/3"		10			

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.



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PROJECT Bridge Over the ACDC 29th Avenue & Arizona Canal

JOB NO. E84-94 DATE 5-21-84

LOG OF TEST BORING NO. 3

RIG TYPE CME-75
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 1232.07'
DATUM Corps of Engineers

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	20				slightly moist; firm to soft	SANDY CLAY, low plasticity, brown
5			X	S	8		17	CL		
10			X	S	10			CL	moist; moderately firm	SANDY CLAY, low plasticity, light brown
15			X	S	11			SW-SM	slightly moist; medium dense	SAND & GRAVEL, some silt, well graded, subangular, nonplastic, brown
20			X	S	58		19	MH	moist; very firm	SANDY SILT, medium to high plasticity, light brown
				S	50/2½"			GC	slightly moist; hard	CLAYEY SAND & GRAVEL, some cobbles, well graded, subangular, moderately lime cemented, low plasticity, light brown
25										Auger refused at 23' Sampler refused at 23'2½"

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.

A-7

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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.

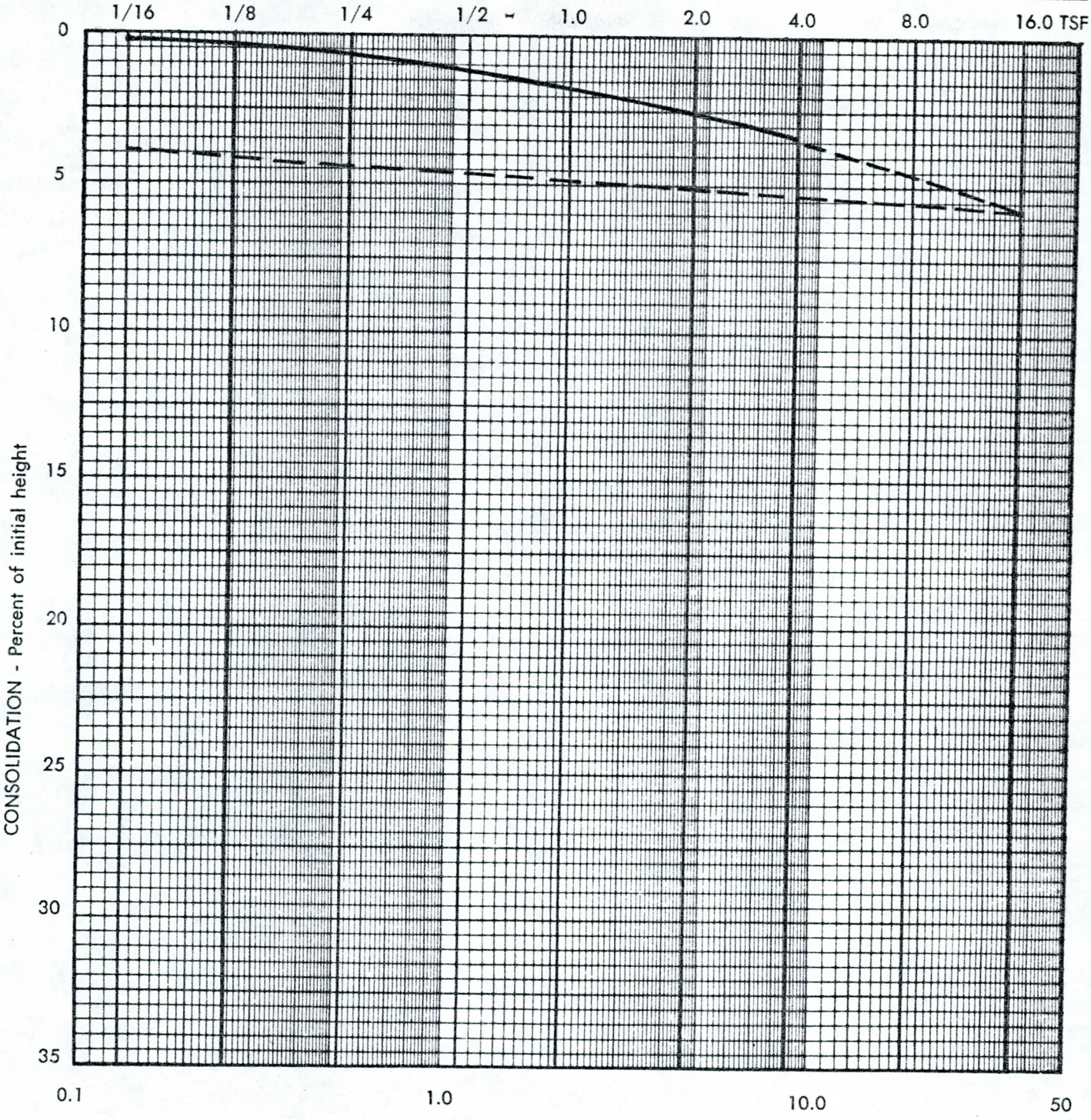
TABULATION OF TEST RESULTS

Job No. E84-94
W/O 1

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	SIEVE ANALYSIS-ACCUM % PASSING												LAB NO
					#200	#100	#50	#40	#30	#16	#10	#8	#4	.25"	.375"	.5"	
					.75"	1"	1.5"	2"	2.5"	3"	3.5"	4"	6"	8"	10"	12"	
BORING #2	10'--	CL	26	8	50.8	62	73	77	81	88	92	93	96	98	100		4-94-3
BORING #2	30'--	SW-SM	-	NP	11.2	13	18	22	29	43	53	57	68	71	76	78	
					86	91	100										4-94-7
BORING #1	15'--	CL	31	12	68.5	73	77	78	80	84	87	88	91	93	94	94	
					94	100											4-94-15
BORING #3	5'--	CL	26	8	72.4	78	86	89	92	95	97	98	100				4-94-21
BORING #3	20'--	MH	54	24	69.5	75	78	80	81	83	86	87	92	94	99	100	4-94-24

CONSOLIDATION TEST WORKSHEET

PROJECT Bridge Over the ACDC JOB NO. E84-94



CURVE	SAMPLE	INITIAL DRY DENSITY LBS./CU. FT.	MOISTURE CONTENT % DRY WEIGHT		UNIFIED SOIL CLASSIFICATION
			INITIAL	FINAL	
16	Sample #1 @ 20'	121.08	11.24	10.36	

SOIL MOISTURE CONDITION

——— INSITU

– – – – SUBMERGED

SERGENT, HAUSKINS & BECKWITH

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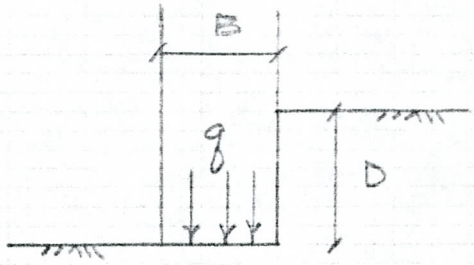
SAFE CAPACITY CALCULATIONS
FOR SPREAD TYPE FOOTINGS

ASSUME: $\phi = 38°$, $C = 0$, $\gamma_t = 125$ PCF

FROM NAVFAC DM7.2, 1982 p 131



1. CONTINUOUS FOOTINGS

$$q_{ULT} = \gamma D N_q + \frac{\gamma B}{2} N_\gamma$$

ASSUME: $D = 3.0'$ $N_q = 60$
$B = 4.0'$ $N_\gamma = 90$

$$q_{ULT} = 125 \cdot 3 \cdot 60 + \frac{125 \cdot 4}{2} \cdot 90$$
$$= 45000 \text{ PSF}$$

$$q_{SAFE} = q_u/3 = \underline{15000 \text{ PSF}}$$

2. SQUARE OR RECTANGULAR FOOTINGS

$$q_{ULT} = \gamma D N_q + 0.4 \gamma B N_\gamma$$

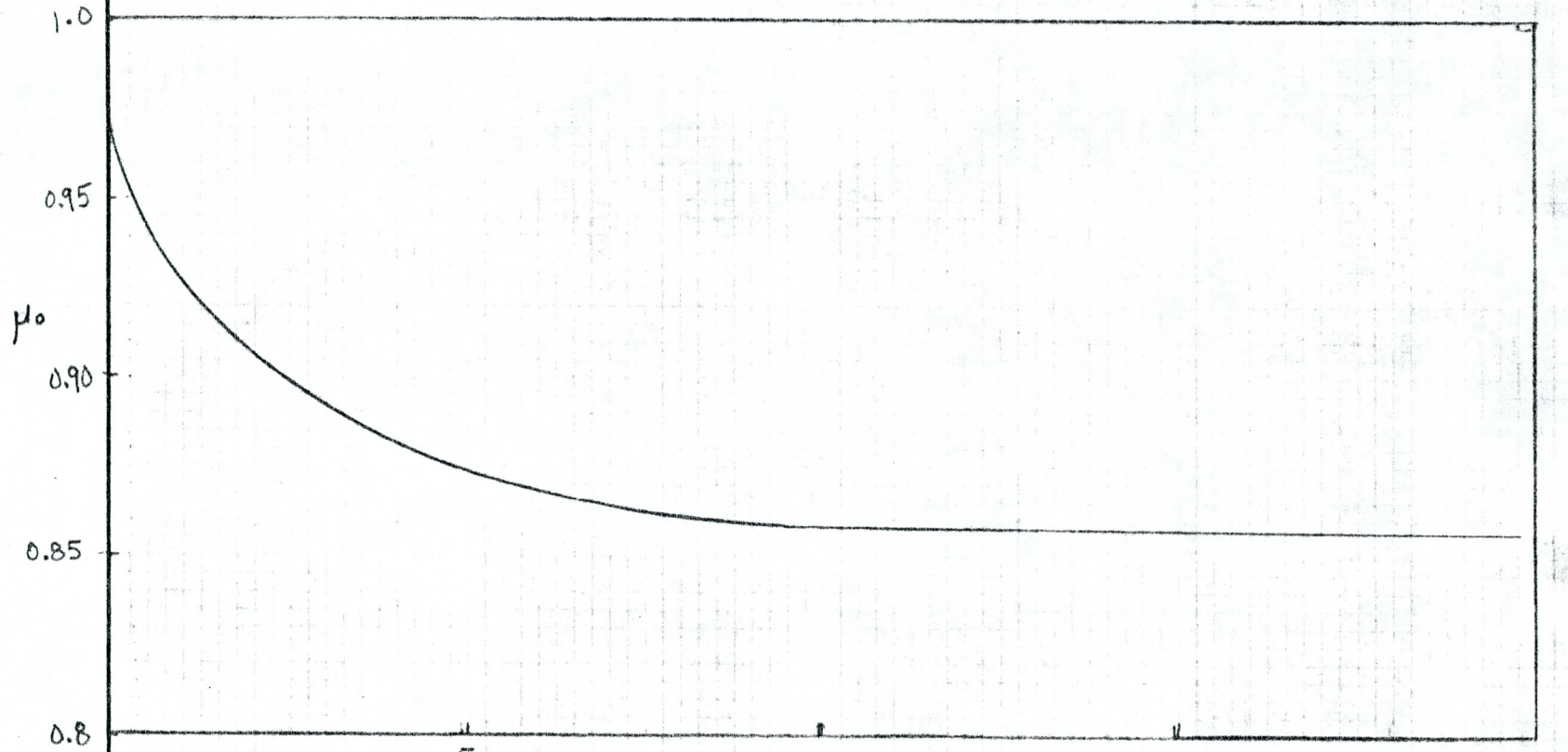
ASSUME: $D = 3.0'$ $N_q = 60$
$B = 4.0'$ $N_\gamma = 90$

$$q_{ULT} = 125 \cdot 3 \cdot 60 + 0.4 \cdot 125 \cdot 4 \cdot 90$$
$$= 40500 \text{ PSF}$$

$$q_s = \frac{40500}{3} = \underline{13500 \text{ PSF}}$$ THEREFORE, USE $q_s = 13500$ PSF.

FIGURE C-1

Variation of μ_o with D/B



D= Depth of Footing Below Finished Grade

B= Footing Width

SETTLEMENT ANALYSIS

FROM: CHRISTIAN, J.T. & CARRIER, W.D.
"JANBU, BJERRUM & KJAERNSLI'S CHART REINTERPRETED".

$$\delta = \mu_0 \mu_1 (q B / E_s)$$

WHERE:

δ = SETTLEMENT

q = AVERAGE BEARING PRESSURE

B = FOOTING WIDTH

E_s = SOIL MODULUS OF ELASTICITY = 1800 psf

μ_0, μ_1 = DIMENSIONLESS PARAMETERS DEPENDENT ON GEOMETRY (SEE FIG. C-1 FOR μ_0)

D = DEPTH OF FOOTING EMBEDMENT = 3.0'

1. CONTINUOUS FOOTING

FOOTING WIDTH FEET	NORMALIZED SETTLEMENT IN/KIP
1.33	0.000Z
1.5	0.000Z
2.0	0.000Z
3.0	0.000Z
4.0	0.000Z
6.0	0.000Z

NOTE: SETTLEMENT WILL BE DEPENDENT ON LOAD. SOME VARIATIONS OF SETTLEMENT CAN BE EXPECTED FOR FOOTINGS 1.33 TO 6.0 FEET WIDE, BUT VARIATIONS WILL BE INSIGNIFICANT FOR THE SOIL CONDITIONS.

2. SQUARE FOOTINGS

FOOTING WIDTH FEET	NORMALIZED SETTLEMENT INCH / KIP
2.0'	0.0021
3.0'	0.0014
4.0'	0.0011
5.0'	0.0009
10.0'	0.0004