GEOTECHNICAL INVESTIGATION REPORT

ACDC Canal Bridge

Maryland Avenue & Arizona Canal

Phoenix, Arizona

SHB Job No. E89-68
Report No. 1

Registered Professional Engineer
Certificate No. 12176
NORMAN HAROLD WETZ
Date Signed 7/13/89
ARIZONA, U.S.A.

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: ACDC Canal Bridge

LOG OF TEST BORING NO. 1

JOB NO.: E89-68 DATE: 6-21-89 & 6-22-89

RIG TYPE: CME-55
BORING TYPE: 6 5/8" Hollow Stem Auger
SURFACE ELEV.:
DATUM:

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification
0				S	5-7-7		0	GP-GM
				S	8-11-18		3	
5				S	15-29-25		4	GC
10				S	50/5"		14	GP-GM
15				S	50/5"		9	
20				S	50/5"		7	
				S	29-28 50/5"		4	SM-GM
30				S	50/3"		10	
35				S	50/5"		8	
40				S	50/1"		8	
45				S	50/3"		10	GM-SM
				S	50/4"		3	
50								

REMARKS	VISUAL CLASSIFICATION
silghtly moist moderately firm to firm	MAN-MADE FILL **SILTY SAND & GRAVEL**, some cobbles, predominantly fine to medium grained sand, gap graded gravels, predominantly rounded, nonplastic, light brown
slightly moist to saturated hard	**CLAYEY SAND & GRAVEL**, trace of cobbles, gap graded sand, predominantly fine grained gravel, subangular to angular, weakly to moderately lime cemented, low plasticity, grayish-brown note: considerable cobbles below 6 1/2'
hard slightly moist to saturated	**SILTY SAND, GRAVEL & COBBLES**, trace of clay, gap graded sand, predominantly fine grained gravel, subangular to angular, moderately to strongly lime cemented, low plasticity, brown note: weakly to moderately lime cemented below 20' note: occasional small boulders 1' to 1 1/2' in diameter
slightly moist to moist hard	**SILTY SAND & GRAVEL**, considerable cobbles, some clay, occasional small boulders (1' to 1 1/2' in diameter), gap graded sand, predominantly coarse grained gravel, subangular to angular, weakly to moderately lime cemented, low plasticity, light brown note: relatively harder drilling below 32' with moderately to strongly lime cemented in layers note: gravels predominantly schist below 40' note: possible breccia below 32'

GROUNDWATER

	DEPTH	HOUR	DATE
▽		none	
▼			

SAMPLE TYPE
A - Drill 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.

PROJECT ACDC Canal Bridge

LOG OF TEST BORING NO. 1

JOB NO. E89-68 DATE 6-21-89 & 6-22-89

RIG TYPE CME-55
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
50									slightly moist	**SILTY SAND & GRAVEL**, considerable cobbles, occasional small boulders 1' to 1 1/2' in diameter, gap graded sand, predominantly coarse grained gravel, subrounded to angular, weakly to moderately lime cemented, low plasticity, light brown
				S	50/3"				hard	
55								GM-SM		
60				S	50/2"		8			note: moderately to strongly lime cemented below 60'
										note: possible breccia
65				S	50/2"		9			
70				S	50/1"		6			
75				S	50/1/2"		4			Auger refused at 74' Sampler refused at 74 1/2'
80										
85										
90										
95										
100										

GROUNDWATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Drill 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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CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX · ALBUQUERQUE · SANTA FE
SALT LAKE CITY · EL PASO · TUCSON · RENO/SPARKS

PROJECT: ACDC Canal Bridge

LOG OF TEST BORING NO. 2

JOB NO.: E89-68 DATE: 6-23-89

RIG TYPE: CME-55
BORING TYPE: 6 5/8" HSA & 4 1/2"CFA
SURFACE ELEV.:
DATUM:

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification
0								
1				S	20-26-24		2	GP-GM
5				S	50/1"			
7								GM-GC
10				S	50/4"		4	
15				S	50/4"			
20				S	50/3"			GM
25				S	50/4"			
30				S	50/1"			
35				S	50/0"			
40				S	50/0" NR			
45								
50								

REMARKS	VISUAL CLASSIFICATION
slightly moist very firm	Possible MAN-MADE FILL **SILTY SAND, GRAVEL & COBBLES**, predominantly fine to medium grained sand, predominantly coarse grained gravel, subangular to round, some lime cementation, nonplastic, light brown to tan
slightly moist hard	**SILTY SAND & GRAVEL**, some cobbles & clay, gap graded sand, well graded gravel, angular to subangular, weakly to moderately lime cemented, low plasticity, light reddish-brown note: considerable cobbles & some small boulders 1' to 1 1/2' in diameter below 7' note: possibly graded to silty gravel from 8' to 12' note: stopped at 15'(HSA refused at 15') 6/23, started with CFA at 6/26
slightly moist hard	**SILTY SAND & GRAVEL**, considerable cobbles, occasional small boulders from 1' to 1 1/2' in diameter, predominantly fine to medium grained sand, predominantly fine grained gravel, subrounded to angular, moderately lime cemented, low plasticity, light brown to tan note: possible breccia below 34' note: used 3" gearbit from 34' to 39'
	Auger refused at 43' sampler refused at 39 1/2'

GROUNDWATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Drill 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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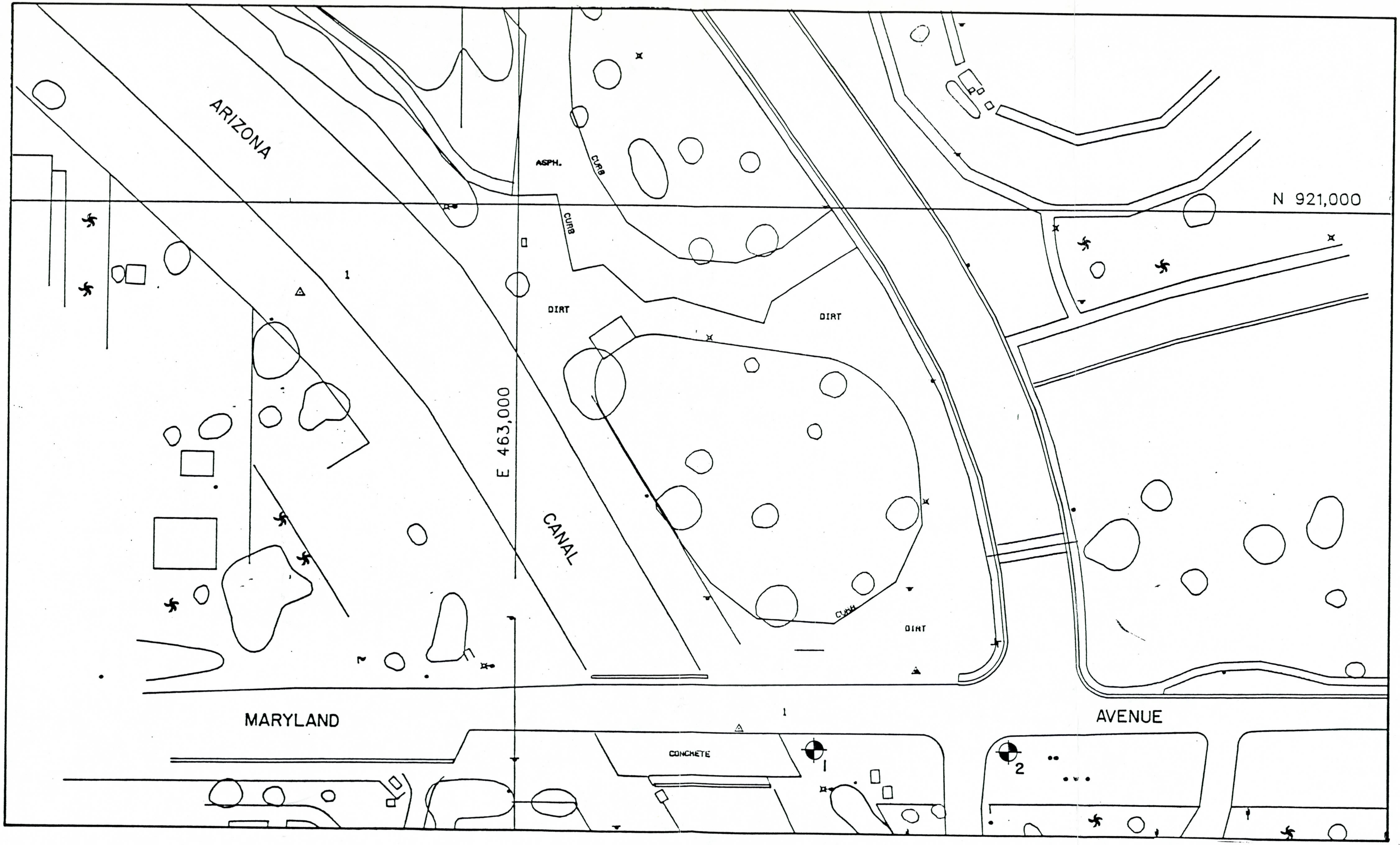

0 25' 50'
SCALE: 1" = 50'-0"

FIGURE 2

SITE PLAN
SHOWING LOCATION of BORINGS

ACDC Canal Bridge
Maryland Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E89-68
Report No. 1

A-4

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TABULATION OF TEST RESULTS

Job No. E89-68
W/O 1

SIEVE ANALYSIS-ACCUM % PASSING

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#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"	LAB NO
1	6"-2'		NA	NA	9.9	14	17	20	23	29	34	36	42	45	49	53	73	100											9-68-1
1	24'6"-25'11"	SM	NV	NP	16	19	23	25	29	37	45	49	61	64	70	73	92	100											9-68-7
1	49'6"-54'9"	SM	NV	NP	17	21	26	29	33	44	51	55	67	70	78	81	90	100											9-68-12/13
2	6"-2'	GM	NV	NP	17	22	27	31	34	41	46	48	55	58	64	72	73	93	100										9-68-18
2	9'6"-14'10"	SM-SC	23	7	23	27	31	34	37	46	53	56	68	72	82	84	100												9-68-20/21
3	4'6"-6'	SM	23	3	36	42	46	48	51	57	63	65	71	74	78	82	85	85	100										9-68-27
4	6"-2'	GM	21	1	30	36	40	42	44	50	54	56	63	67	70	74	80	80	100										9-68-31
5	4'6"-6'	SM	24	3	28	33	36	38	41	47	55	58	69	75	83	92	100												9-68-36

SERGENT, HAUSKINS & BECKWITH

TABULATION OF TEST RESULTS

Job No. E89-68
W/O 2

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"	
2	0-2'		NA	NA	17	23	31	37	42	52	59	61	67	71	77	82	
					89	95	99	100									9-68-40

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS
SOIL & FOUNDATION ENGINEERING • ENGINEERING GEOLOGY • HYDROGEOLOGY
MATERIALS ENGINEERING • MATERIALS TESTING • ENVIRONMENTAL SERVICES

July 13, 1989

T.Y. Lin International
Emerson Court
Suite 175
1817 North 7th Street
Phoenix, Arizona 85006

SHB Job No. E89-68
Report No. 1

Attention: Mitchell D. Smith, P.E.
Project Manager

Re: ACDC Canal Bridge
Maryland 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, laboratory analysis and recommended criteria for approach roadway pavement design and foundation design. Under separate cover, an engineering report for the relocation of utilities will be forthcoming.

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

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By [signature]
Gary N. Sheppard, Staff Engineer

Reviewed by [signature]
Norman H. Wetz, P.E.

[Stamp: Registered Professional Engineer (Civil) Certificate No. 12176 NORMAN HAROLD WETZ Date Signed 7/13/89 ARIZONA U.S.A.]

Copies: Addressee (3)

TABLE OF CONTENTS Page

Registered Professional Engineer
CERTIFICATE NO. 12176
NORMAN HAROLD WETZ
(CIVIL)
Date Signed 7/13/89
ARIZONA, U.S.A.

SHB Job No. E89-68
Report No. 1

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
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1. INTRODUCTION

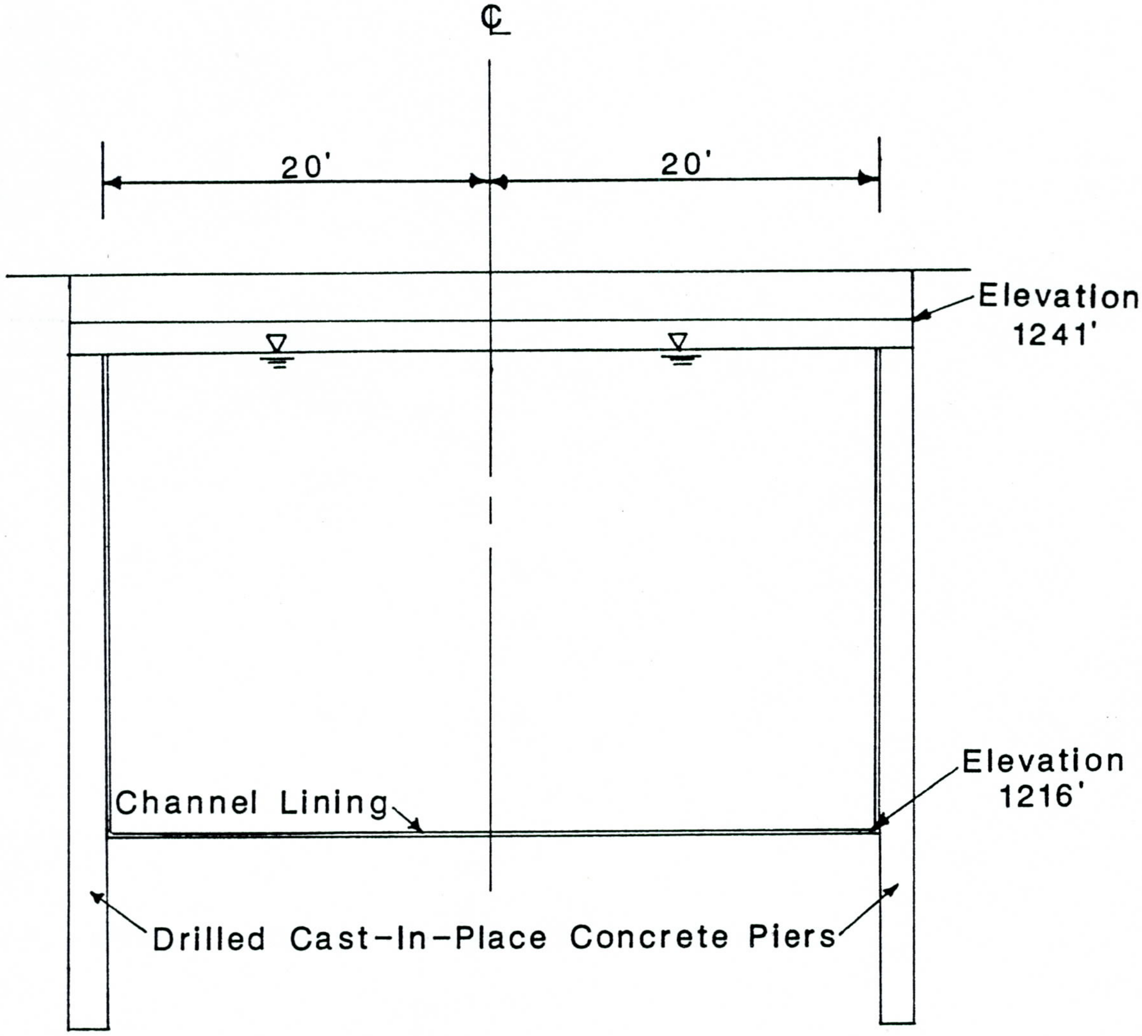
This report is submitted pursuant to a geotechnical investigation made by this firm at the site of the proposed Maryland Avenue Bridge over the Arizona Canal diversion channel (ACDC) located in Phoenix, Arizona. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site to provide recommendations for approach roadway pavements, site grading, excavations and foundation design.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided by Mitchell D. Smith, P.E., Project Manager of T.Y. Lin International.

It is understood that a bridge approximately 46 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 approximately 1,216 feet (Corps of Engineers datum). The channel will be concrete lined and rectangular in shape with a 40-foot wide bottom, as shown in Figure 1. The design flow rate at Maryland Avenue is approximately 8,300 cubic feet per second at a depth of approximately 24 feet and a velocity of approximately 9 feet per second.

Approaches to the bridge will likely consist of a concrete apron between the Arizona Canal and the ACDC



Not To Scale

FIGURE 1

TYPICAL DIVERSION CHANNEL SECTION

ACDC Canal Bridge
Maryland Avenue & Arizona Canal
Phoenix, Arizona
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bridge. Asphaltic concrete flexible pavement will be used on the east end of the ACDC bridge and will taper eastward to match with Maryland Avenue.

If the final design details vary significantly from those outlined herein, this firm should be notified for review and possible revision of recommendations.

3. INVESTIGATION

3.1 Subsurface Exploration

Two exploratory borings, one at each abutment location, were drilled to depths of 43 and 74 feet below existing grade. The borings were performed using our truck-mounted CME-55 drill rig advancing a 6 5/8-inch O.D. hollow stem auger, 4-inch continuous flight auger or 3-inch gear bit. Standard penetration testing was performed at 5-foot intervals in the borings. The bore-holes 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 Keith H. Dahlen, E.I.T., staff engineer of this firm.

3.2 Laboratory Analysis.

Moisture content determinations were made on selected samples recovered from the borings. The results of these tests are shown on the boring logs.

Grain-size analysis, Atterberg limits and an R-value determination were performed on selected samples. The results of these tests along with a brief description of soil mechanics testing procedures are presented in Appendix B.

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

The site is relatively level. The Arizona Canal runs in a north-south direction adjacent to the proposed ACDC canal. The surrounding area is primarily residential and a city park is located just north of Maryland Avenue. The western portion of the park will be removed for construction of the canal. Vegetation consists mainly of a moderate growth of trees and grass in the park. A few trees are also located along the south side of Maryland Avenue.

4.2 Geotechnical Profile

The subsoils underlying the site consist predominantly of clayey to silty sand, gravel and cobbles with occasional boulders that extend to the full depth of the borings. These soils are weakly to strongly cemented with calcium carbonate, and exhibit low to medium plasticity. Although the soils are very firm to hard for the full depth of the borings, an increase in hardness was observed in the borings below depths of about 32 to 34 feet. Materials below these depths may be classified as a relatively soft to very soft Breccia rock unit.

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 investigation. It is our opinion that in situ moisture conditions are somewhat drier than those reported because the boreholes were maintained full of water during standard penetration testing. The high moisture contents apparently are the result of water being forced between the sample and the inner wall of the sampler.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

Drilled pier foundations bearing a minimum of 10 feet below finished grade of the canal are recommended for the abutments. Design criteria for drilled piers are presented in Section 5.2.

Spread-type footings can be used as an alternative to the cast-in-place, drilled concrete piers at the abutment locations. The concrete channel lining will protect the near-surface soils from scour. Design criteria for spread-type footings can be provided if desired.

5.2 Cast-In-Place Concrete Piers

5.2.1 Estimated Settlements

Straight-sided, drilled, cast-in-place concrete piers bearing a minimum of 10 feet below the final canal invert elevation are recommended for the support of the foundation loads involved. Settlement and bearing capacity calculations were completed in accordance with criteria set forth in NAVFAC DM-7.2, Foundations and Earth Structure Design Manual 7.2, May, 1982. Calculation sheets are presented in Appendix C of this report.

According to the NAVFAC document, settlement, as opposed to bearing capacity, controls the design of drilled piers larger than 2 feet in diameter. Settlements of pier foundations designed and constructed in accordance with criteria presented herein can be estimated using design tables presented in Appendix C. Settlement charts were developed for both the end-bearing and side shear cases using elastic theory. Settlements are presented in terms of inches of settlement per kip of vertical load. Thus, using the charts, the settlement can be quickly estimated for straight-sided piers incorporating the pier diameter and the pier tip elevation.

5.2.2 Resistance to Lateral Loads

The design for lateral loads should be in accordance with procedures detailed by Broms (1965, 1964a, 1964b)*. The soil should be modeled as both cohesive and cohesionless, with the lower allowable lateral load from these procedures to be used for design. Based on our experience with the site soils, conservative strength parameters recommended for use in computing the ultimate lateral resistance are = 35 degrees and c_u = 1,000 pounds per square foot. The passive earth pressure coefficient for the cohesionless case is 3.0. The in situ unit weight of the soil can be taken as 120 pounds per cubic foot.

Implementation of Brom's procedures also requires a coefficient of horizontal subgrade reaction, k_h. For the cohesive case, a value of k_hD = 460 pounds per square inch, independent of depth, is recommended (D = diameter of drilled pier). Thus, for a 24-inch diameter pier, k_h = 19 pounds per cubic inch. For

*References are listed at the end of the report.

the cohesionless case, k_h varies with depth in accordance with the relationship

$$k_h = n_h \, (z/D)$$

where z is depth below finished grade and D is the pier diameter. In using this relationship, a value of n_h = 60 pounds per cubic inch is recommended. These values are in conformance with values suggested by Broms (1964a, 1964b). Values of the coefficient of subgrade reaction should be reduced by a factor of 2 for analysis of seismic loading conditions.

Criteria provided 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.

5.2.3 Cleaning of Drilled Pier Excavations

Straight, drilled pier excavations should be advanced with a single flight auger, or bucket auger bits, to the design depth. It should be verified by inspection and measurement that excavations are open to that depth. Loose material present in the bottom of the holes should be cleaned using the auger or other equipment so that no more than 2 inches of loose material is present after cleaning.

5.2.4 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.

5.2.5 Inspection & Construction

Continuous inspection of the construction of drilled piers should be carried out by the geotechnical engineer. The inspector should verify diameter, depth, cleaning, and the nature of the materials encountered in the pier excavations. Concrete placement should be continuously observed by the inspector to ensure that it meets specified requirements. An inspection report should be submitted on each pier which should state in writing that all details have been inspected and meet required specifications. All drilled piers should be observed by the geotechnical engineer's representative for verification of cleaning and contact with proper bearing material.

It appears that straight-sided, drilled pier excavations can be advanced to the depths recommended with little or no caving. Since caving is expected to be minimal, concrete quantities may be very near the neat volume indicated by the plans. If workmen or inspectors are required to enter the pier excavations, casing will be necessary for their safety.

5.3 Abutment Wall Design Criteria

The earth pressures against the abutments will depend upon the degree of restraint. Although it is anticipated that the soils will stand vertically during construction, a certain amount of sloughing can be expected behind the walls over the life of the project. The drilled piers should be designed to withstand forces equivalent to a hydrostatic pressure of 30 pounds per cubic foot. The walls should be designed to withstand forces equivalent to a hydrostatic pressure of 15 pounds per cubic foot.

It is understood that the bridge abutments will be constructed prior to the excavation of the channel and the concrete walls will be cast neat against the drilled piers and soils in the vertical walls of the channel excavation. This, in turn, will require that the vertical side walls of the channel remain stable during construction of the channel. A stability analysis has been performed to determine whether the channel excavation can generally be made without the need for lagging between the caissons. Details of the analyses, including example calculations and assumptions regarding soil properties, are presented in Appendix C. Two potential failure modes were considered. The lowest safety factors for the Case 1 and Case 2 failure modes are 2.07 and 1.71, respectively for a pier spacing of 9.0 feet center to center.

In our opinion, the analyses presented demonstrates that lagging will not be required to provide stability for the channel side walls. However, this recommendation remains subject to the construction qualifications stated in Section 5.2 of this report.

5.4 APPROACH ROADWAY PAVEMENT DESIGN

5.4.1 Pavement Design Methodology

Pavement design analysis for the approach roadways was based on the simplified American Association of State Highway and Transportation Officials (AASHTO) design method of flexible pavements (Mamlouk, 1988) utilizing an R-value test performed by this firm and traffic projections provided by the City of Phoenix Traffic Engineering Department. This method has been adopted by the City of Phoenix and the Arizona Department of Transportation (ADOT). Recommended approach roadway pavement sections are presented in Section 5.5.

5.4.2 Traffic Analysis

Information concerning anticipated traffic flows on Maryland Avenue was provided by J. Donald Herp, P.E., Deputy Street Transportation Director of the City of Phoenix Traffic Engineering Department. Current traffic is estimated to be a total of 5,000 vehicles per day. Traffic volumes for the year 2010 are also estimated to be 5,000 vehicles per day, therefore this value was chosen to represent the 20-year mean average daily traffic (ADT) for pavement design.

Other parameters assumed for the analysis include:

- Serviceability Index = 2.5
- Reliability Level = 90 percent
- Standard Deviation = 0.4
- Percentage of Trucks = 5 percent

The first three parameters are standard City of Phoenix design parameters and are also consistent with ADOT design philosophy. Design traffic calculations for the 18-kip equivalent axle loading, presented in Appendix C, determined a 20-year mean value of applications for the design lane.

5.4.3 Design R-Value

The R-value determination for the subgrade sample was 79. This value is relatively high because of the low clay content and large percentage of coarse grained sands and gravels in the soils. An R-value of 79 roughly equates to a soil resilient modulus of 18,000 psi.

5.5 Pavement Design Recommendations

Based on the parameters presented in Section 5.4, the required weighted structural number is 1.7. The minimum thicknesses of asphaltic concrete (AC) and aggregate base coarse (ABC) specified by the City of Phoenix for nonmajor streets are 2 inches and 4 inches, respectively. This would provide a section with a structural number of 1.42. Inclusion of an additional 1 inch of AC or 2 inches of ABC would increase the structural number to the required value. The design section recommended is 3 inches AC over 4 inches ABC. This section, rather than 2 inches AC over 6 inches ABC, was selected because it will provide for less maintenance over the 20-year design life.

5.6 Materials Quality, Construction Requirements & Asphaltic Concrete

5.6.1 Materials Quality & Construction Requirements

The materials quality and construction requirements should conform to the following sections of the current "Uniform Standard Specifications of Public Works

Construction" sponsored and prepared by the Maricopa Association of Governments (MAG) and current City of Phoenix supplements:

Item	Section(s)
Fill Construction	211
Subgrade Preparation	301
Untreated Base	310 & 702.2
Bituminous Prime Coat	315
Asphaltic Concrete	321 & 710

All base coarse and select materials should be compacted to a density of at least 100 percent of maximum dry density per ASTM D698. All subgrade materials beneath streets, curbs, gutters and sidewalks should be compacted to at least 95 percent of maximum dry density in accordance with ASTM D698. Moisture content during compaction should be maintained within 2 percent of optimum moisture content.

5.6.2 Asphaltic Concrete

A type C-3/4 mineral aggregate or approved alternative should be utilized. The job mix formula should be established using the Marshall Method of mix design, with design parameters determined by ASTM D1559. The following criteria should be used in the mix design:

Number of blows, each end of specimen - 75
Stability, pounds - 1,800 minimum

Flow, units of 0.01 inch - 8 to 18 percent

Percent air voids - 3 to 5

Percent voids in mineral aggregate - 14 minimum

The stripping potential of the job mix design formulation should be determined by performance of ASTM D1075. The type and quantity of antistrip additive, if required, should be assessed to meet local agency specification requirements.

REFERENCES

Broms, B.B., 1964a, Lateral Resistance of Piles in Cohesive Soils, ASCE, JSMFD, Volume 90, No. SM2, March, pp. 27-63.

Broms, B.B., 1964b, Lateral Resistance of Piles in Cohesionless Soils, ASCE, JSMFD, Volume 90, No. SM3, May, pp. 123-156.

Broms, B.B., 1965, Design of Laterally Loaded Piles, ASCE, JSMFD, Volume 90, No. SM3, May, pp. 79-99.

Mamlouk, Michael S., 1988, Simplified AASHTO Design Method of Flexible Pavements for City of Phoenix, June, 42 pages.

SERGENT, HAUSKINS, & BECKWITH
SERGENT, HAUSKINS, & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS

R VALUE CALCULATION
ASTM D-2844

JOB NO. E89-68

W.O.NO. 2

PROJECT: ACDC BRIDGE

LAB NO. 40

LOCATION: #2 @ 0 TO 2'

DATE 6/6/89

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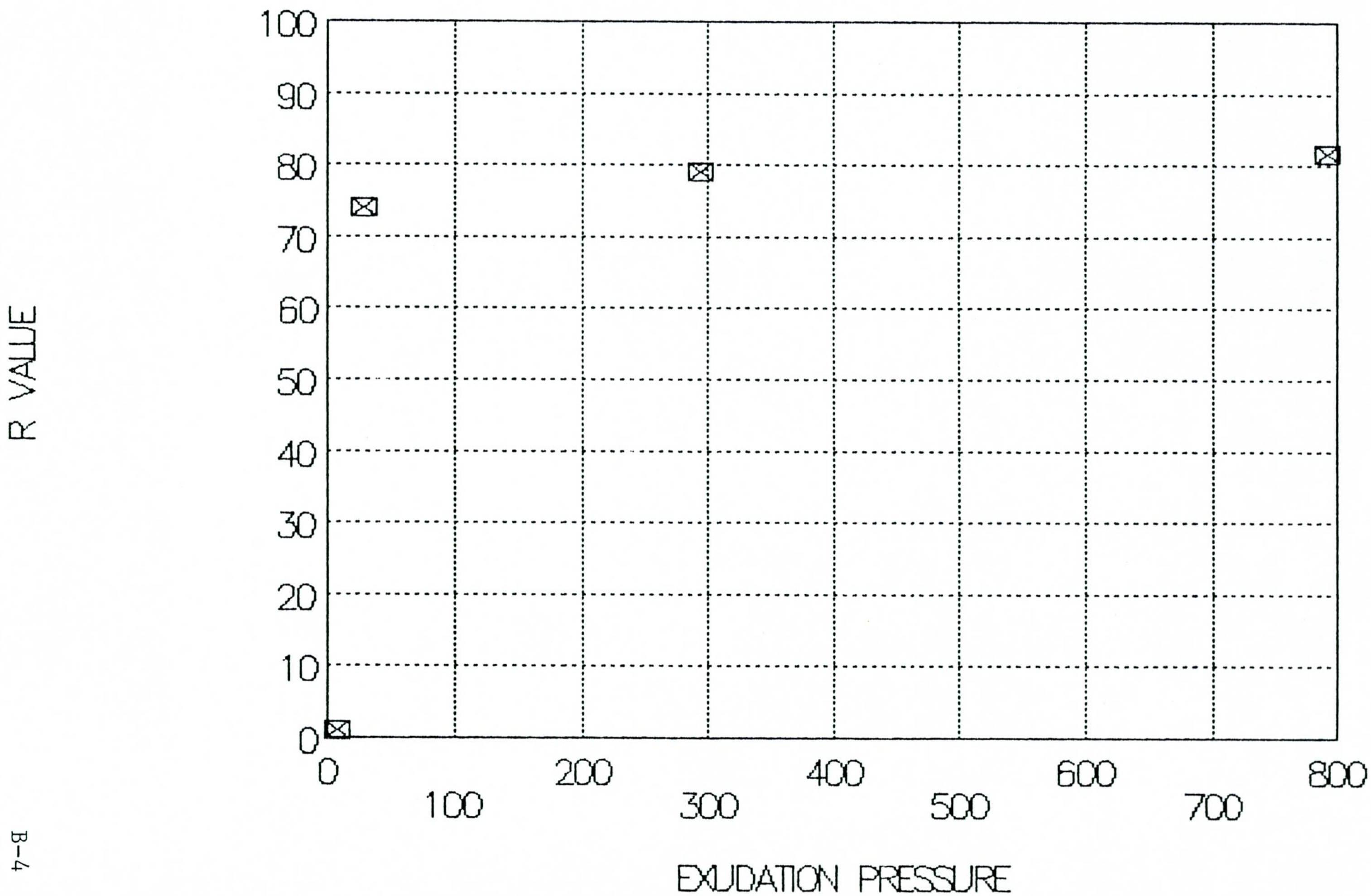
SERGENT, HAUSKINS, & BECKWITH

SPECIMEN I. D.	A	B	C
Moisture Content	6.9%	8.6%	7.8%
Compaction Foot PSI	25	0	0
Specimen Height, inches	2.50	2.50	2.54
Dry Density, PCF	130.9	128.5	130.2
Ph @ 1000 lb	12	16	13
Ph @2000 lb	19	28	22
Displacement	4.17	4.17	4.19
Expansion Pressure PSI	0.0	0.0	0.0
Exudation Pressure PSI	796	28	294
R Value	82	74	79
R Value at 300 PSI =	79		

R VALUE

#2 @ 0 TO 2'



DESIGN OF STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS

FOR DRILLED PIERS GREATER THAN 24-INCHES IN DIAMETER, SETTLEMENT CONTROLS DESIGN

CONSIDERING END-BEARING ONLY:

$$\text{SETTLEMENT} = Q_p \left(\frac{L}{AE_p} + \frac{C_p}{Bq_o} \right)$$

WHERE:

Q_p = POINT LOAD AT PIER TIP (KIPS)

L = PILE LENGTH (FT.)

A = PILE CROSS SECTIONAL AREA (FT^2)

E_p = MODULUS OF ELASTICITY OF PIER
FOR CONCRETE $E_p = 3 \times 10^6$ psi
$= 4.32 \times 10^6$ KSF wrong → 0.432×10^6 KSF

C_p = EMPIRICAL COEFFICIENT
FOR STIFF CLAY, $C_p = 0.03$

B = PIER DIAMETER (FT)

q_o = ULTIMATE END-BEARING CAPACITY (KSF)

CALCULATION OF q_o:

$$q_o = \frac{1}{3}\left(\frac{0.4 \bar{N} D}{B} \right) \text{ FOR DRILLED PIERS}$$

WHERE

$\bar{N} = C_N \cdot N$

$C_N = 0.77 \log_{10} \frac{20}{p}$

p = EFFECTIVE OVERBURDEN STRESS AT PIER TIP (KSF)

D = PIER EMBEDMENT DEPTH (FT)

B = PIER DIAMETER AT TIP (FT)

TABLE C-1

CALCULATION OF $\overline{N}$

$$\overline{N} = N \cdot 0.77 \log \frac{20}{p}$$

wrong - see corrections dated 8/89

DEPTH (FT)	N	p (KSF)	C_N	$\overline{N}$
10	80	1.2	0.94	75
20	80	2.4	0.71	57
30	80	3.6	0.57	46
40	80	4.8	0.48	38
50	80	6.0	0.40	32
60	80	7.2	0.34	27
70	80	8.4	0.29	23

TABLE C-2

CALCULATION OF q_o

$$q_o = \frac{1}{3}\left(\frac{0.4\overline{N}D}{B}\right) \text{ (KSF)}$$

DEPTH (FT)	B=2.0	B=2.5	B=3.0	B=3.5	B=4.0	B=4.5	B=5.0
10	50	40	33	29	25	22	20
20	76	61	51	43	38	34	30
30	92	74	61	53	46	41	37
40	101	81	68	58	51	45	41
50	107	85	71	61	53	47	43
60	108	86	72	62	54	48	43
70	107	86	72	61	54	48	43

SHB
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TABLE C-3

NORMALIZED SETTLEMENT IN INCHES/KIP VERTICAL LOAD FOR ABUTMENT PIERS (FINISHED GRADE = 1216) ASSUMING END BEARING ONLY

PIER TIP ELEVATION (FT)	DEPTH BELOW FINISHED GRADE (FT)	SETTLEMENT, INCHES/KIP						
		B = 2.0	B = 2.5	B = 3.0	B = 3.5	B = 4.0	B = 4.5	B = 5.0
1,206	10	0.0036	0.0036	0.0036	0.0036	0.0036	0.0036	0.0036
1,196	20	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024	0.0024
1,186	30	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020	0.0020
1,176	40	0.0018	0.0018	0.0018	0.0018	0.0018	0.0018	0.0018
1,166	50	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017
1,156	60	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017
1,146	70	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017

CONSIDERING SIDE SHEAR ONLY:

$$\text{SETTLEMENT} = Q_S \left(\frac{\alpha_s L}{A E_p} + \frac{C_s}{D q_o} \right)$$

WHERE:

Q_S = SHAFT FRICTION LOAD (KIPS)

L = PIER LENGTH (FT.)

A = PIER CROSSECTIONAL AREA (FT^2)

E_p = MODULUS OF ELASTICITY OF PIER FOR CONCRETE $E_p = 3 \times 10^6$ PSI $= 4.32 \times 10^6$ KSF

$C_s = (0.93 + 0.16 \, D/B) \, C_p$

D = DEPTH OF EMBEDMENT (FT.)

q_o = ULTIMATE END-BEARING CAPACITY (KSF)

α_s = 0.5 FOR PARABOLIC DISTRIBUTION

TABLE C-4

CALCULATION OF C_s

$$C_s = (0.93 + 0.16 \, D/B) \, C_p$$

PIER TIP ELEV. (FT.)	EMBED. DEPTH (FT)	B=2.0	B=2.5	B=3.0	B=3.5	B=4.0	B=4.5	B=5.0
1,206	10	0.05	0.05	0.04	0.04	0.04	0.04	0.04
1,196	20	0.08	0.07	0.06	0.06	0.05	0.05	0.05
1,186	30	0.10	0.09	0.08	0.07	0.06	0.06	0.06
1,176	40	0.12	0.10	0.09	0.08	0.08	0.07	0.07
1,166	50	0.15	0.12	0.11	0.10	0.09	0.08	0.08
1,156	60	0.17	0.14	0.12	0.11	0.10	0.09	0.09
1,146	70	0.20	0.16	0.14	0.12	0.11	0.10	0.10

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TABLE C-5

NORMALIZED SETTLEMENT IN INCHES/KIP VERTICAL LOAD FOR ABUTMENT PIERS (FINISHED GRADE = 1,216) ASSUMING SIDE-SHEAR ONLY

PIER TIP ELEVATION (FT)	DEPTH BELOW FINISHED GRADE (FT)	SETTLEMENT, INCHES/KIP						
		B = 2.0	B = 2.5	B = 3.0	B = 3.5	B = 4.0	B = 4.5	B = 5.0
1,206	10	0.0012	0.0015	0.0015	0.0017	0.0019	0.0022	0.0024
1,196	20	0.0006	0.0007	0.0007	0.0008	0.0008	0.0009	0.0010
1,186	30	0.0005	0.0005	0.0005	0.0005	0.0005	0.0006	0.0007
1,176	40	0.0004	0.0004	0.0004	0.0004	0.0005	0.0005	0.0005
1,166	50	0.0003	0.0004	0.0004	0.0004	0.0004	0.0004	0.0005
1,156	60	0.0003	0.0003	0.0004	0.0004	0.0004	0.0004	0.0004
1,146	70	0.0003	0.0003	0.0003	0.0004	0.0004	0.0004	0.0004

Stability Analysis
(Soil Between Piers)

BASIC ASSUMPTIONS

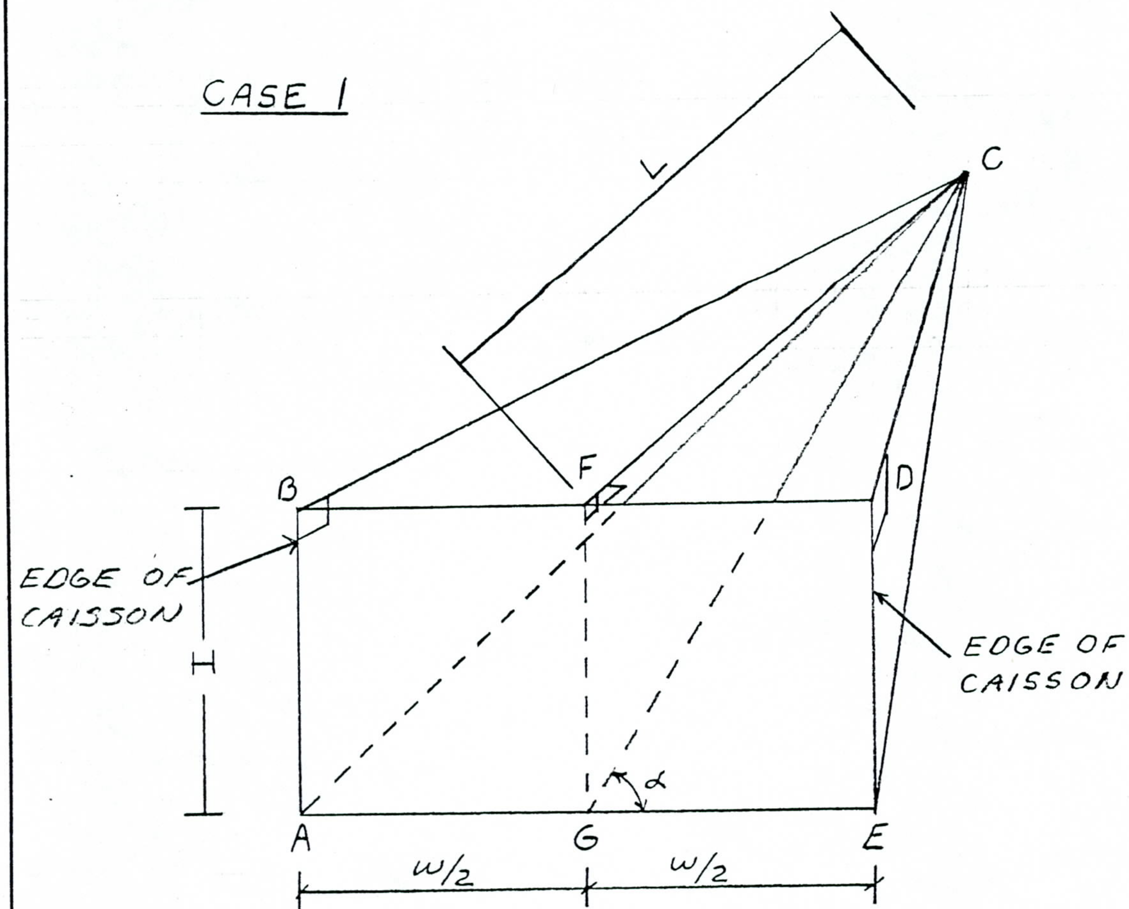
$\gamma = 120$ PCF

$\phi = 30°$

$c = 300$ PSF

$w = 6$ ft.

CASE 1



$\alpha = \tan^{-1}(H/L)$

LINE $\overline{CF} = L$

LINE $\overline{BC} = \sqrt{L^2 + (w/2)^2}$

LINE $\overline{CG} = \sqrt{L^2 + H^2}$

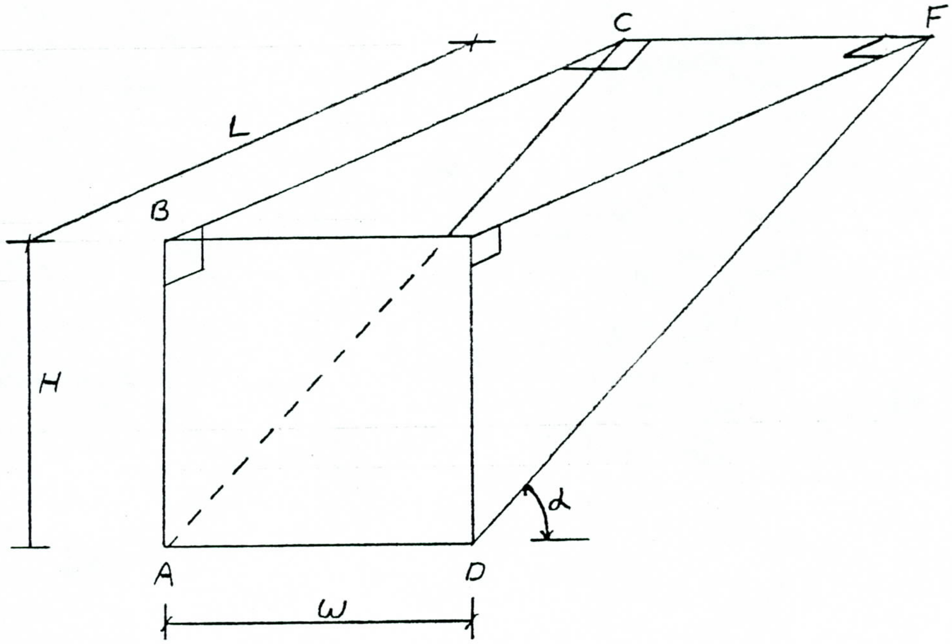
AREA $\overline{ABC} = \frac{1}{2}(H)(\overline{BC})$

AREA $\overline{ACE} = \frac{1}{2}(w)(\overline{CG})$

$\varpi = \frac{1}{3}(H)(L)(w)\gamma$

$$F.S. = \frac{[2(\overline{ABC}) + \overline{ACE}]C + \varpi \cos\alpha \tan\phi}{\varpi \sin\alpha}$$

Stability Analysis
(soil between Piers)
CASE 2



LINE $\overline{AC} = H^2 + L^2$

$\alpha = \tan^{-1}(H/L)$

AREA $\overline{ABC} = \frac{1}{2}(H)(L)$

AREA $\overline{ACFD} = w(\overline{AC})$

$\overline{W} = (\overline{ABC})(w)(\gamma)$

$$F.S. = \frac{[2(\overline{ABC}) + \overline{ACFD}]C + \overline{W}\cos\alpha\tan\phi}{\overline{W}\sin\alpha}$$

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Project ACDC CANAL BRIDGE

Job No: E89-68

Computed by: GNS Ckd. by: NAW

Date 7/10/89

STABILITY ANALYSIS

EXAMPLE CALCULATIONS

CASE 1

CONSIDER $H = 10'$, $L = 6'$, $W = 6'$

$\alpha = \tan^{-1}(H/L) = \tan^{-1}(10/6) = 59.0°$

$\overline{CF} = L = 6'$

$\overline{BC} = [L^2 + (W/2)^2]^{1/2} = [6^2 + (6/2)^2]^{1/2} = 6.71'$

$\overline{CG} = (L^2 + H^2)^{1/2} = (6^2 + 10^2)^{1/2} = 11.66'$

$\overline{ABC} = 1/2\, H\, \overline{BC} = 1/2(10)(6.71) = 33.54'^{\square}$

$\overline{ACE} = 1/2\, W\, \overline{CG} = 1/2(6)(11.66) = 34.98'^{\square}$

$\overline{W} = 1/3\, HLW\gamma = 1/3(10)(6)(6)120 = 14,400\text{ lb}$

$F.S. = [(2\overline{ABC} + \overline{ACE})c + \overline{W}\cos\alpha\tan\phi] / \overline{W}\sin\alpha$

$= \{[2(33.54) + 34.98]300 + [14,400\cos 59°\tan 30°]\} / 14,400\sin 59°$

$= (30,618 + 4,282) / 12,343 = \underline{\underline{2.83}}$

CASE 2

CONSIDER $H = 20'$, $L = 8'$, $W = 6'$

$\alpha = \tan^{-1}(H/L) = \tan^{-1}(20/8) = 68.2°$

$\overline{AC} = (H^2 + L^2)^{1/2} = (20^2 + 8^2)^{1/2} = 21.54'$

$\overline{ABC} = 1/2\, HL = 1/2(20)(8) = 80.0'^{\square}$

$\overline{ACFD} = W\,\overline{AC} = 6(21.54) = 129.24'^{\square}$

$\overline{W} = \overline{ABC}\, W\gamma = (80)(6)(120) = 57,600\text{ lb.}$

$F.S. = [(2\overline{ABC} + \overline{ACFD})c + \overline{W}\cos\alpha\tan\phi] / \overline{W}\sin\alpha$

$= \{[2(80) + 129.24]300 + 57,600\cos 68.2°\tan 30° / 57,600\sin 68.2°$

$= (86,772 + 12,350) / 53,481 = \underline{\underline{1.85}}$

CASE 1 ANALYSIS RESULTS

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
5.00	0.50	84.29	0.60	15.27
5.00	1.00	78.69	1.20	8.05
5.00	1.50	73.30	1.80	5.82
5.00	2.00	68.20	2.40	4.83
5.00	2.50	63.43	3.00	4.35
5.00	3.00	59.04	3.60	4.11
5.00	3.50	55.01	4.20	4.01
5.00	4.00	51.34	4.80	4.00
5.00	4.50	48.01	5.40	4.05
5.00	5.00	45.00	6.00	4.14
5.00	5.50	42.27	6.60	4.26
5.00	6.00	39.81	7.20	4.40
5.00	6.50	37.57	7.80	4.56
5.00	7.00	35.54	8.40	4.73
5.00	7.50	33.69	9.00	4.92
5.00	8.00	32.00	9.60	5.11
5.00	8.50	30.47	10.20	5.31
5.00	9.00	29.05	10.80	5.52
5.00	9.50	27.76	11.40	5.73
5.00	10.00	26.57	12.00	5.95

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
10.00	1.00	84.29	2.40	7.82
10.00	2.00	78.69	4.80	4.36
10.00	3.00	73.30	7.20	3.38
10.00	4.00	68.20	9.60	3.00
10.00	5.00	63.43	12.00	2.86
10.00	6.00	59.04	14.40	2.83
10.00	7.00	55.01	16.80	2.86
10.00	8.00	51.34	19.20	2.94
10.00	9.00	48.01	21.60	3.05
10.00	10.00	45.00	24.00	3.17
10.00	11.00	42.27	26.40	3.32
10.00	12.00	39.81	28.80	3.47
10.00	13.00	37.57	31.20	3.63
10.00	14.00	35.54	33.60	3.80
10.00	15.00	33.69	36.00	3.98
10.00	16.00	32.00	38.40	4.16
10.00	17.00	30.47	40.80	4.34
10.00	18.00	29.05	43.20	4.53
10.00	19.00	27.76	45.60	4.72
10.00	20.00	26.57	48.00	4.92

CASE 1 ANALYSIS RESULTS cont.

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
15.00	1.50	84.29	5.40	5.39
15.00	3.00	78.69	10.80	3.22
15.00	4.50	73.30	16.20	2.65
15.00	6.00	68.20	21.60	2.46
15.00	7.50	63.44	27.00	2.42
15.00	9.00	59.04	32.40	2.45
15.00	10.50	55.01	37.80	2.52
15.00	12.00	51.34	43.20	2.62
15.00	13.50	48.01	48.60	2.75
15.00	15.00	45.00	54.00	2.88
15.00	16.50	42.27	59.40	3.03
15.00	18.00	39.81	64.80	3.18
15.00	19.50	37.57	70.20	3.34
15.00	21.00	35.54	75.60	3.51
15.00	22.50	33.69	81.00	3.68
15.00	24.00	32.01	86.40	3.86
15.00	25.50	30.47	91.80	4.04
15.00	27.00	29.06	97.20	4.22
15.00	28.50	27.76	102.60	4.40
15.00	30.00	26.57	108.00	4.59

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
20.00	2.00	84.29	9.60	4.22
20.00	4.00	78.69	19.20	2.68
20.00	6.00	73.30	28.80	2.31
20.00	8.00	68.20	38.40	2.21
20.00	10.00	63.43	48.00	2.22
20.00	12.00	59.04	57.60	2.27
20.00	14.00	55.01	67.20	2.36
20.00	16.00	51.34	76.80	2.48
20.00	18.00	48.01	86.40	2.60
20.00	20.00	45.00	96.00	2.74
20.00	22.00	42.27	105.60	2.89
20.00	24.00	39.81	115.20	3.04
20.00	26.00	37.57	124.80	3.20
20.00	28.00	35.54	134.40	3.37
20.00	30.00	33.69	144.00	3.54
20.00	32.00	32.01	153.60	3.71
20.00	34.00	30.47	163.20	3.89
20.00	38.00	27.76	182.40	4.24
20.00	40.00	26.57	192.00	4.43

CASE 1 ANALYSIS RESULTS cont.

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
25.00	2.50	84.29	15.00	3.54
25.00	5.00	78.69	30.00	2.38
25.00	7.50	73.30	45.00	2.12
25.00	10.00	68.20	60.00	2.07
25.00	12.50	63.44	75.00	2.10
25.00	15.00	59.04	90.00	2.17
25.00	17.50	55.01	105.00	2.27
25.00	20.00	51.34	120.00	2.39
25.00	22.50	48.01	135.00	2.52
25.00	25.00	45.00	150.00	2.66
25.00	27.50	42.27	165.00	2.81
25.00	30.00	39.81	180.00	2.96
25.00	32.50	37.57	195.00	3.12
25.00	35.00	35.54	210.00	3.28
25.00	37.50	33.69	225.00	3.45
25.00	40.00	32.01	240.00	3.62
25.00	42.50	30.47	255.00	3.80
25.00	45.00	29.06	270.00	3.97
25.00	47.50	27.76	285.00	4.15
25.00	50.00	26.57	300.00	4.33

CASE 2 ANALYSIS RESULTS

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
5.00	0.50	84.29	0.90	11.00
5.00	1.00	78.69	1.80	6.17
5.00	1.50	73.30	2.70	4.68
5.00	2.00	68.20	3.60	4.03
5.00	2.50	63.44	4.50	3.72
5.00	3.00	59.04	5.40	3.59
5.00	3.50	55.01	6.30	3.55
5.00	4.00	51.34	7.20	3.58
5.00	4.50	48.01	8.10	3.65
5.00	5.00	45.00	9.00	3.76
5.00	5.50	42.27	9.90	3.88
5.00	6.00	39.81	10.80	4.03
5.00	6.50	37.57	11.70	4.19
5.00	7.00	35.54	12.60	4.36
5.00	7.50	33.69	13.50	4.54
5.00	8.00	32.01	14.40	4.72
5.00	8.50	30.47	15.30	4.91
5.00	9.00	29.05	16.20	5.11
5.00	9.50	27.76	17.10	5.31
5.00	10.00	26.57	18.00	5.52

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
10.00	1.00	84.29	3.60	5.95
10.00	2.00	78.69	7.20	3.57
10.00	3.00	73.30	10.80	2.86
10.00	4.00	68.20	14.40	2.58
10.00	5.00	63.43	18.00	2.47
10.00	6.00	59.04	21.60	2.45
10.00	7.00	55.01	25.20	2.49
10.00	8.00	51.34	28.80	2.55
10.00	9.00	48.01	32.40	2.65
10.00	10.00	45.00	36.00	2.76
10.00	11.00	42.27	39.60	2.88
10.00	12.00	39.81	43.20	3.01
10.00	13.00	37.57	46.80	3.15
10.00	14.00	35.54	50.40	3.30
10.00	15.00	33.69	54.00	3.45
10.00	16.00	32.01	57.60	3.61
10.00	17.00	30.47	61.20	3.78
10.00	18.00	29.05	64.80	3.93
10.00	19.00	27.76	68.40	4.10
10.00	20.00	26.57	72.00	4.27

CASE 2 ANALYSIS RESULTS cont.

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
15.00	1.50	84.29	8.10	4.26
15.00	3.00	78.69	16.20	2.70
15.00	4.50	73.30	24.30	2.25
15.00	6.00	68.20	32.40	2.10
15.00	7.50	63.43	40.50	2.05
15.00	9.00	59.04	48.60	2.07
15.00	10.50	55.01	56.70	2.13
15.00	12.00	51.34	64.80	2.21
15.00	13.50	48.01	72.90	2.31
15.00	15.00	45.00	81.00	2.42
15.00	16.50	42.27	89.10	2.54
15.00	18.00	39.81	97.20	2.67
15.00	19.50	37.57	105.30	2.81
15.00	21.00	35.54	113.40	2.95
15.00	22.50	33.69	121.50	3.09
15.00	24.00	32.01	129.60	3.24
15.00	25.50	30.47	137.70	3.39
15.00	27.00	29.05	145.80	3.54
15.00	28.50	27.76	153.90	3.70
15.00	30.00	26.57	162.00	3.85

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
20.00	2.00	84.29	14.40	3.42
20.00	4.00	78.69	28.80	2.27
20.00	6.00	73.30	43.20	1.95
20.00	8.00	68.20	57.60	1.85
20.00	10.00	63.43	72.00	1.84
20.00	12.00	59.04	86.40	1.88
20.00	14.00	55.01	100.80	1.95
20.00	16.00	51.34	115.20	2.04
20.00	18.00	48.01	129.60	2.14
20.00	20.00	45.00	144.00	2.26
20.00	22.00	42.27	158.40	2.38
20.00	24.00	39.81	172.80	2.50
20.00	26.00	37.57	187.20	2.63
20.00	28.00	35.54	201.60	2.77
20.00	30.00	33.69	216.00	2.91
20.00	32.00	32.01	230.40	3.05
20.00	34.00	30.47	244.80	3.20
20.00	36.00	29.05	259.20	3.34
20.00	38.00	27.76	273.60	3.49
20.00	40.00	26.57	288.00	3.64

CASE 2 ANALYSIS RESULTS cont.

H (ft)	L (ft)	alpha (degree)	Weight (kips)	F.S.
25.00	2.50	84.29	22.50	2.92
25.00	5.00	78.69	45.00	2.01
25.00	7.50	73.30	67.50	1.77
25.00	10.00	68.20	90.00	1.71
25.00	12.50	63.43	112.50	1.72
25.00	15.00	59.04	135.00	1.77
25.00	17.50	55.01	157.50	1.85
25.00	20.00	51.34	180.00	1.94
25.00	22.50	48.01	202.50	2.04
25.00	25.00	45.00	225.00	2.16
25.00	27.50	42.27	247.50	2.28
25.00	30.00	39.81	270.00	2.40
25.00	32.50	37.57	292.50	2.53
25.00	35.00	35.54	315.00	2.66
25.00	37.50	33.69	337.50	2.80
25.00	40.00	32.01	360.00	2.94
25.00	42.50	30.47	382.50	3.08
25.00	45.00	29.05	405.00	3.23
25.00	47.50	27.76	427.50	3.37
25.00	50.00	26.57	450.00	3.52

APPROACH PAVEMENT DESIGN

FROM TRAFFIC ANALYSIS:

TOTAL 18-KIP EQUIVALENT LOADS OVER 20 YEARS = 68,400 FOR THE DESIGN LANE.

ASSUMPTIONS:

SERVICIBILITY INDEX = 2.5
RELIABILITY LEVEL = 90%
STANDARD DEVIATION = 0.4
SOIL RESILIENT MODULUS = 18,000 psi
STRUCTURAL NUMBERS
ASPHALT (a_1) = 0.41
ABC (a_2) = 0.12
DRAINAGE COEFFCIENTS
m_2 = 1.25

DESIGN STRUCTURAL NUMBER

$S_N = 1.7$

ASSUME THICKNESS OF ASPHALT (D_1) = 3 INCHES

$S_N = 1.7 = a_1(D_1) + a_2(D_2)m_2$

$1.7 = 0.41(3) + 0.12(D_2)1.25$

$D_2 = 3.1''$ CITY OF PHOENIX REQUIRES 4" MINIMUM

SECTION IS 3" ASPHALT OVER 4" ABC

ASSUME $D_1 = 2''$

$S_N = 1.7 = 0.41(2) + 0.12(D_2)(1.25)$

$D_2 = 5.9''$ SAY 6"

SECTION IS 2" ASPHALT OVER 6" ABC

TRAFFIC LOADING CALCULATION SHEET

Street Name and Location

MARYLAND AVE. OVER ARIZONA CANAL

Street Type:	Current Year & A.D.T.	20th Year & A.D.T.
local commercial	1990 5,000	2010 5,000

Calculation of Mean, One-way Traffic (Thousands of Veh. per Day)

5,000/2 = 2.5

Notation for Type of Veh.	Proportion by Type of Vehicle		18^k Load by 1000 Vehicle	Equiv 18^k Load by Veh Dist
C	0.950		0.8	0.760
B	0.001		250	0.250
	Proportion Comm. Veh.	Proportion Type Com. Veh.		
2P	0.049	0.800	1.2	0.047
2S	0.049	0.150	5.8	0.043
2D	0.049	0.036	163.2	0.288
3D	0.049	0.005	598.7	0.147
2S1	0.049	0.005	408.2	0.100
2S2	0.049	0.004	956.5	0.187
3S2		neg.	514.3	
2-2		neg.	304.3	
3-2		neg.	936.8	
3-3		neg.	936.8	
2S1-2		neg.	846.7	
3S1-2		neg.	958.0	
		TOTAL OF EQUIV. 18^k loads per 1000 veh		1.822

Calculation of daily, 20 year mean equivalent 18^k single axle load applications for one-way traffic:

1.822 (2.5) = 4.56 = 4,560

Calculation of design lane loading:

20 YEARS (0.75) (4,560) = 68,400

by GNS 160+16

chk'd [illegible]

REPORT NO. 2

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS
SOIL & FOUNDATION ENGINEERING • ENGINEERING GEOLOGY • HYDROGEOLOGY
MATERIALS ENGINEERING • MATERIALS TESTING • ENVIRONMENTAL SERVICES

July 14, 1989

T. Y. Lin International
Emerson Court
Suite 175
1817 North 7th Street
Phoenix, Arizona 85006

SHB Job No. E89-68
Report No. 2

Attention: Mitchell D. Smith, P.E.
Project Manager

Re: ACDC Canal Bridge
Utility Relocations
Maryland Avenue & Arizona Canal
Phoenix, Arizona

Gentlemen:

Submitted herein are the results of our field and laboratory investigation for the referenced project. Included are boring logs, a site plan showing boring locations and a description of terminology used in describing the soils. Results of laboratory testing are attached, except for moisture contents which are included on the boring logs.

Should you have any questions, please do not hesitate in contacting us.

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

Registered Professional Engineer (Civil) Certificate No. 12176 NORMAN HAROLD WETZ Date Signed ARIZONA, U.S.A.

By ______________________
Norm H. Wetz, P.E.

Copies: Addressee (3)

mb/89-J5/7-14-89

REPLY TO: 3232 W. VIRGINIA, PHOENIX, ARIZONA 85009
PHOENIX (602) 272-6848 | TUCSON (602) 792-2779 | ALBUQUERQUE (505) 884-0950 | SANTA FE (505) 471-7836 | SALT LAKE CITY (801) 266-0720 | EL PASO (915) 564-1017 | RENO/SPARKS (702) 331-2375

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.

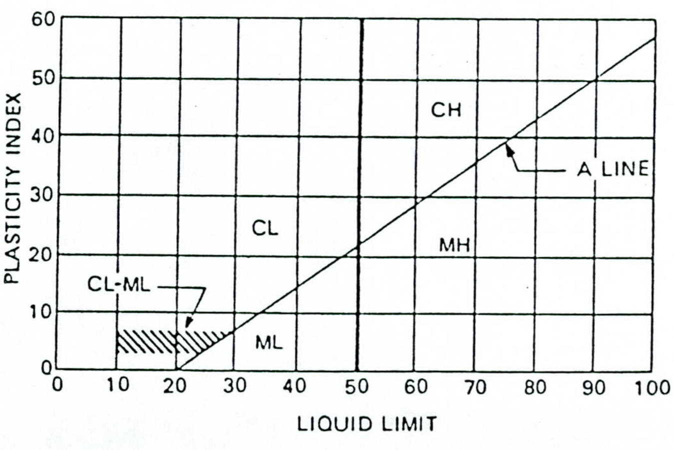
UNIFIED SOIL CLASSIFICATION SYSTEM

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.
			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

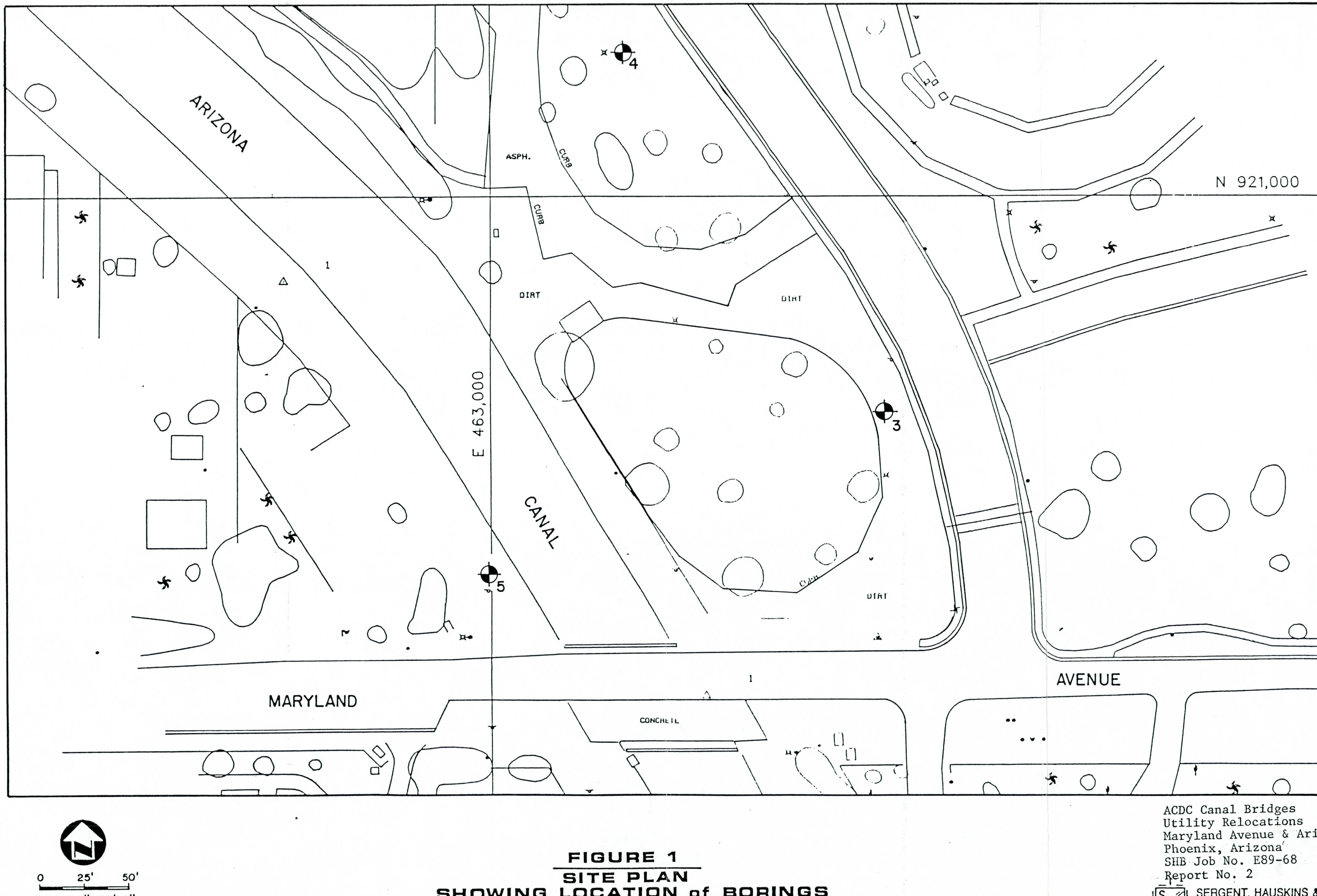


FIGURE 1

SITE PLAN
SHOWING LOCATION of BORINGS

ACDC Canal Bridges
Utility Relocations
Maryland Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E89-68
Report No. 2

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PROJECT ACDC Canal Bridge

LOG OF TEST BORING NO. 3

JOB NO. E89-68 DATE 6-23-89

RIG TYPE CME-55
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Sample Type	Blow Count	Moisture Content Percent of Dry Weight	Unified Soil Classification	Remarks	Visual Classification
0–1	S	16-22-44		SM	slightly moist to moist; firm	**SILTY SAND**, some fine grained gravel, gap graded, nonplastic to low plasticity, light brown
5	S	40-48-43	4	GM	slightly moist; hard	**SILTY SAND & GRAVEL**, some to considerable cobbles, some clay, gap graded sand, predominantly fine grained gravel, angular, weakly lime cemented, low plasticity, light brown
~6–7				ML	slightly moist to moist; very firm to hard	**SANDY SILT**, trace of clay, fine to medium grained sand, weakly lime cemented, low plasticity, light brown
10	S	50/5"		GC	slightly moist; hard	**CLAYEY SAND & GRAVEL**, some to considerable cobbles, occasional small boulders 1' to 1 1/2' in diameter, gap graded sand, predominantly fine grained gravel, subangular to angular, weakly to moderately lime cemented, low plasticity, light brown to light reddish-brown
15	S	50/4"		GM	slightly moist; hard	**SILTY SAND & GRAVEL**, some to considerable cobbles, predominantly fine to medium grained sand, predominantly fine grained gravel, subangular to angular, weakly to moderately lime cemented, low plasticity to nonplastic, light brown
20	S	50/5"				

Auger refused at 19 1/2'
Sampler refused at 19'11"

Column headings also include: Continuous Penetration Resistance; Graphical Log; Sample; Dry Density lbs. per cubic foot.

GROUNDWATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE
A - Drill 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 ACDC Canal Bridge

JOB NO. E89-68 DATE 6-23-89

LOG OF TEST BORING NO. 5

RIG TYPE CME-55
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV.
DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0								GP	slightly moist dense	MAN-MADE FILL **SILTY GRAVEL**, considerable cobbles, some fine to medium grained sand, predominantly coarse grained gravel, subrounded to angular, some lime cementation, low plasticity to nonplastic, light grayish-brown
5				S	12-6-7		4	SM-GM	slightly moist moderately firm to hard	**SILTY SAND & GRAVEL**, considerable cobbles, some clay, gap graded sand, predominantly fine grained gravel, subrounded to angular, some lime cementation, low plasticity, light reddish-brown note: Man Made Fill 3 1/2' to 6 1/2'
10				S	50/3"					
15				S	50/4"			GM	slightly moist hard	**SILTY SAND, GRAVEL & COBBLES**, trace of clay, predominantly fine to medium grained sand, predominantly coarse grained gravel, subangular to angular, moderately to strongly lime cemented, low plasticity, light brown note: some small boulders 1' to 1 1/2' in diameter below 16'
20				S	50/2"					
25										Auger refused at 24' Sampler refused at 19'8"
30										
35										
40										
45										
50										

GROUNDWATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A - Drill 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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LOG OF TEST BORING NO. 4

PROJECT: ACDC Canal Bridge

JOB NO. E89-68 DATE 6-23-89

RIG TYPE: CME-55
BORING TYPE: 6 5/8" Hollow Stem Auger
SURFACE ELEV.:
DATUM:

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blow Count	Dry Density lbs. per cubic foot	Moisture Content Percent of Dry Weight	Unified Soil Classification
0				S	4-4-10		6	GM
5				S	12-32-42			GM
10				S	50/1"			
15				S	50/4"			GM
20				S	50/1"			
25								
30								
35								
40								
45								
50								

REMARKS	VISUAL CLASSIFICATION
moist to very moist moderately firm	**SILTY SAND & GRAVEL**, predominantly fine to medium grained sand, predominantly fine grained gravel, subangular to angular, weakly lime cemented, low plasticity, brown note: 6" clayey sand topsoil at surface
slightly moist hard	**SILTY SAND & GRAVEL**, considerable cobbles, some clay, gap graded sand, predominantly fine grained gravel, low plasticity, light brown note: occasional small boulders 1' to 1 1/2' in diameter below 11 1/2'
slightly moist hard	**SILTY SAND & GRAVEL**, considerable cobbles, predominantly fine to medium grained sand, predominantly coarse grained gravel, subangular to angular, moderately lime cemented, low plasticity, light brown
	Auger refused at 19'6" Sampler refused at 19'7"

GROUNDWATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE
A - Drill 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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TABULATION OF TEST RESULTS

Job No. E89-68
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"	
1	6"-2'		NA	NA	9.9	14	17	20	23	29	34	36	42	45	49	53	
					73	100											9-68-1
1	24'6"-25'11"	SM	NV	NP	16	19	23	25	29	37	45	49	61	64	70	73	
					92	100											9-68-7
1	49'6"-54'9"	SM	NV	NP	17	21	26	29	33	44	51	55	67	70	78	81	
					90	100											9-68-12/13
2	6"-2'	GM	NV	NP	17	22	27	31	34	41	46	48	55	58	64	72	
					73	93	100										9-68-18
2	9'6"-14'10"	SM-SC	23	7	23	27	31	34	37	46	53	56	68	72	82	84	
					100												9-68-20/21
3	4'6"-6'	SM	23	3	36	42	46	48	51	57	63	65	71	74	78	82	
					85	85	100										9-68-27
4	6"-2'	GM	21	1	30	36	40	42	44	50	54	56	63	67	70	74	
					80	80	100										9-68-31
5	4'6"-6'	SM	24	3	28	33	36	38	41	47	55	58	69	75	83	92	
					100												9-68-36

Case 1

$H = 10'$, $L = 6'$, $w = 5'$

$\alpha = 59.0°$

$\overline{CF} = L = 6'$

$\overline{BC} = [(6^2) + (5/2)^2]^{1/2} = 6.5'$

$\overline{CG} = 11.66'$

$\overline{ABC} = 1/2\, H\, \overline{BC} = 1/2\,(10)(6.5) = 32.5'^{\square}$

$\overline{ACE} = 1/2\, w\, \overline{CG} = (1/2)(5)(11.66) = 29.15'^{\square}$

$\overline{W} = 1/3\, H L w \gamma = (1/3)(10)(6)(5)(120) = 12,000$ lb

$$F.S. = [((2\,\overline{ABC}) + (\overline{ACE}))C + \overline{W} \cos\alpha \tan\phi] / \overline{W} \sin\alpha$$

$$= [((2)(32.5) + (29.15))(300) + (12,000)(\cos 59°)(\tan 30°)] / (12,000)(\sin 59°)$$

(.515) (.577) (.857)

$$= (28,245 + 3,566) / 10,284$$

F.S. = 3.09

F.S. increase with $w = 5$ instead of 6

$\frac{3.09}{2.83} = 1.09$ $\frac{2.03}{1.85} = 1.09$

lowest F.S. = Case 2, $H = 25$

F.S. = 1.71 (1.09) = 1.86

Case 2

$\phi = 35^\circ$ $\gamma_a = 30\ lb/ft^3$

$C_u = 1{,}000$

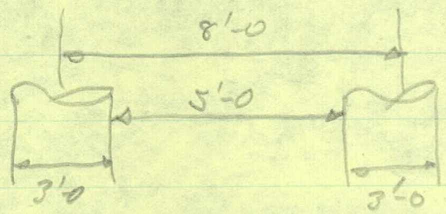
$\rho_r = 3.0$ Walls $\gamma_a = 15\ lb/ft^3$

$W_t = 120\ lb/ft^3$

$c = 300\ lb/ft^2$ soil stability Appx-C

Drilled Shaft Length

1245
1184
61 ft



Case 2 S.F. Page 8 of 16

$H = 20;\ L = 8',\ w = 5'$

$\alpha = \tan^{-1}(H/L) = 68.2^\circ$

$\overline{AC} = 21.54'$

$\overline{ABC} = 80'$

$\overline{ACFD} = w\overline{AC} = (5)(21.54) = 107.7$

$\overline{W} = \overline{ABC}\, w\, \gamma = (80)(5)(120) = 48{,}000\ lb$

$F.S. = [(2\overline{ABC} + \overline{ACFD})c + \overline{W}\cos\alpha\tan\phi] / \overline{W}\sin\alpha$

$= [((2)(80) + (107.7)](300) + (48{,}000)(\cos 68.2^\circ)(\tan 30^\circ) / (48{,}000)(\sin 68.2)$ (.371) (.577) (.928)

$= (80{,}310 + 10{,}275) / 44{,}544$

F.S. = 2.03