ERGENT, HAUSKINS & BECKWITH

GEOTECHNICAL INVESTIGATION REPORT

Pedestrian Bridge

47th Avenue Over Arizona Canal Diversion Channel

Phoenix, Arizona

Registered Professional Engineer
CERTIFICATE NO.
121..
NORMAN HAROLD
WETZ
Date Signed 10/21/86
ARIZONA, U.S.A.

SHB Job No. E86-199

Consulting Geotechnical Engineers

PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO

S H B

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY

B. DWAINE SERGENT, P.E.
LAWRENCE A. HANSEN, PH.D., P.E.
RALPH E. WEEKS, P.G.
DARRELL L. BUFFINGTON, P.E.
DONALD VAN BUSKIRK, P.G.
JOHN B. HAUSKINS, P.E.
DALE V. BEDENKOP, P.E.
DONALD L. CURRAN, P.E.
J. DAVID DEATHERAGE, P.E.
MICHAEL R. RUCKER, P.E.
GEORGE H. BECKWITH, P.E.
ROBERT W. CROSSLEY, P.E.
DONALD G. METZGER, P.G.
JONATHAN A. CRYSTAL, P.E.
PAUL V. SMITH, P.G.
ROBERT D. BOOTH, P.E.
NORMAN H. WETZ, P.E.
ROBERT L. FREW
ALLON C. OWEN, JR., P.E.

October 24, 1986

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

SHB Job No. E86-199

Attention: Lloyd W. Miller, P.E.

Re: Pedestrian Bridge
47th Avenue Over Arizona
Canal Diversion Channel
Phoenix, Arizona

Gentlemen:

Our Geotechnical Investigation Report on the referenced project is herewith submitted. The report includes 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

Registered Professional Engineer (Civil) Certificate No. 12176 NORMAN HAROLD WETZ Date Signed 10/24/86 ARIZONA, U.S.A.

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

Registered Professional Engineer (Civil) Certificate No. 13872 LAWRENCE A. HANSEN Date Signed 10/24/86 ARIZONA, U.S.A.

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

Copies: Addressee (3)

REPLY TO: 3940 W. CLARENDON, PHOENIX, ARIZONA 85019

PHOENIX ALBUQUERQUE SANTA FE SALT LAKE CITY EL PASO

TABLE OF CONTENTS Page

REPORT

APPENDIX A

APPENDIX B

APPENDIX C

SHB Job No. E86-199

Registered Professional Engineer
CERTIFICATE
12176
NORMAN HAROLD
WETZ
Date Signed 10/24/86
ARIZONA, U.S.A.

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO

1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed pedestrian bridge at 47th Avenue over the Arizona Canal Diversion Channel 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.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided by Don Davis, P.E. of Hoffman-Miller Engineers, Inc. Consulting Engineers.

It is understood that a single-span pedestrian bridge about 123 feet long will cross the Arizona Canal at 47th Avenue. The vertical loads at the abutments of the bridge will be approximately 260 kips.

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

Two exploratory borings were drilled to depths of 28 to 31 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 selected intervals in the borings. All 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 Tony J. Freiman, E.I.T., staff engineer of this firm.

3.2 Laboratory Analysis

Moisture content measurements were made on selected drive samples recovered. The results of these tests are shown on the boring logs.

Grain-size analysis and Atterberg Limits tests were also performed on selected samples. The results of these tests are presented in Appendix B.

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

At the site of the proposed pedestrian bridge, a double corrugated culvert covered with an earth embankment spans the diversion channel. At present, the embankment is used to cross the channel.

The new pedestrian bridge will span the Arizona Canal Diversion Channel at approximately Station 398+75. The area is generally void of any vegetative cover due to recent construction.

4.2 Geotechnical Profile

The soils underlying the site consist of a surface layer of silty to clayey sand fill that extends to a depth of about 5 1/2 feet at the Boring 1 location. The relative firmness of the fill varies from hard to firm. Below the fill at Boring 1 and below the ground surface at Boring 2, a layer of sandy to silty clay extends to a depth of approximately 12 1/2 feet. This medium plasticity clay ranges in relative firmness from very soft to moderately firm at the south end of the bridge site to hard at the north end of the bridge site.

Below about 12 1/2 feet, clayey sands, silty sands and sands that are firm to hard or dense were encountered. The sands extended to the final explored depths of the borings.

Sand, gravel and cobble deposits were encountered at the bottom of the borings and were the cause of auger refusal. These deposits are very dense.

4.3 Soil Moisture & Groundwater Conditions

No free groundwater was encountered in the borings and soil moisture contents were relatively low throughout their extent.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

It is recommended that the new pedestrian bridge be supported on straight or belled, drilled, cast-in-place concrete piers. We understand that vertical walls for the diversion channel will be placed aproximately 7 feet inside the bridge supports. It is further understood that the channel depth will be 20 to 25 feet below the ground surface. Due to the close proximity of the drilled piers to the channel wall, and uncertainties concerning the method and sequence of the construction, we recommend construction of drilled piers designed for end-bearing only.

In addition, the upper clayey soils are considered somewhat moisture sensitive. In the event of substantial moisture increases in the supporting clayey soils, excessive settlements of the foundations could result. The potential for moisture increases is quite high due to the adjacent Arizona Canal.

Therefore, we recommend that drilled piers extend at least to the proposed channel bottom. It should be noted that extending the piers to the depth of the sand, gravel and cobbles (approximately 28 to 31 feet below grade) will provide much greater support than the overlying soils, and will allow the use of smaller diameter piers.

5.2 Straight, Manually-Cleaned, Cast-in-Place Concrete Piers

5.2.1 Downward Loads

Straight, manually-cleaned, drilled, cast-in-place concrete piers are recommended for the support of the foundation loads involved. Safe downward capacities of drilled piers extending a minimum of 25 feet below the finished grade elevation are presented in Tables 1 and 2. Capacities are based on end-bearing only. Methodology and input data parameters used in the analysis of drilled pier capacities are outlined in Appendix C.

Capacities apply to full dead plus live loads. A one-third increase is recommended when considering wind or seismic forces.

5.2.2 Estimated Settlements

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 the end-bearing case 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 piers, incorporating the pier diameter and the pier tip depth.

5.2.3 Resistance to Lateral Loads

It is understood that design for lateral loads will be in accordance with procedures detailed by Broms (1965, 1964a, 1964b)*. Further, the soils are to be modeled

*References are listed at the end of this report.

TABLE 1

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

DEPTH BELOW GRADE, FEET	CAPACITY, KIPS			
	d = 2.0 Ft.	d = 3.0 Ft.	d = 4.0 Ft.	d = 5.0 Ft.
25.0	38	85	151	235
27.5	44	99	176	275
30.0	50	113	201	314
32.5	100	226	402	628
35.0	100	226	402	628

TABLE 2

SAFE DOWNWARD CAPACITIES FOR BELLED, DRILLED, CAST-IN-PLACE CONCRETE PIERS

DEPTH BELOW GRADE, FEET	CAPACITY, KIPS			
	d = 4.0 Ft.	d = 4.5 Ft.	d = 5.0 Ft.	d = 5.5 Ft.
25.0	151	191	236	285
27.5	176	223	275	333
30.0	201	254	314	380
32.5	402	509	628	760

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 θ = 25° and c_u = 500 pounds per square foot. The passive earth pressure coefficient for the cohesionless case is 3.0. The in situ unit weight of the soil is estimated to be 120 pounds per cubic foot.

Implementation of Broms' procedures also requires a coefficient of horizontal subgrade reaction, k_h. For the cohesive case, a value of k_hD = 300 pounds per square inch, independent of depth, is recommended. Thus, for a 24-inch diameter pier, k_h = 13 pounds per cubic inch. For 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. A factor of safety of 3 should be used for lateral analysis of forces longitudinal to the bridge structure alignment.

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 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.

All loose material should be manually-cleaned from the base of the piers so that undisturbed native soil is exposed throughout. This will likely impose requirements of a minimum shaft diameter of 30 inches to allow access and casing of the shaft.

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 Inspection & Construction

Continuous inspection of the construction of drilled piers should be carried out by the geotechnical engineer.

The inspector should verify proper diameter, depth and cleaning, and should also verify the nature of materials encountered in the pier excavations. Concrete placement should be continuously observed by the inspector to ensure that it meets requirements. An inspection report should be submitted on each pier stating, in writing, that all details have been inspected and meet requirements.

Some caving is expected near the top of each excavation, particularly in the silty sand fill at Boring 1 and the low consistency sandy clay at Boring 2. We expect that concrete quantities will be somewhat above the neat volume because of the anticipated sloughing. We recommend that a short section of casing be readily available for support of the upper part of the excavation.

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, May, pp. 79-99.

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.

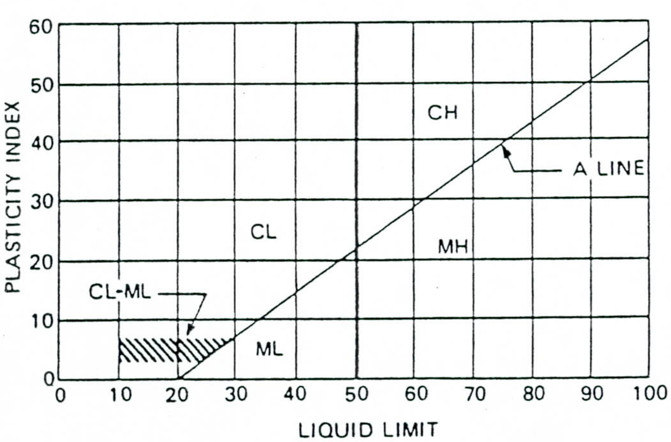
UNIFIED SOIL CLASSIFICATION SYSTEM

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.

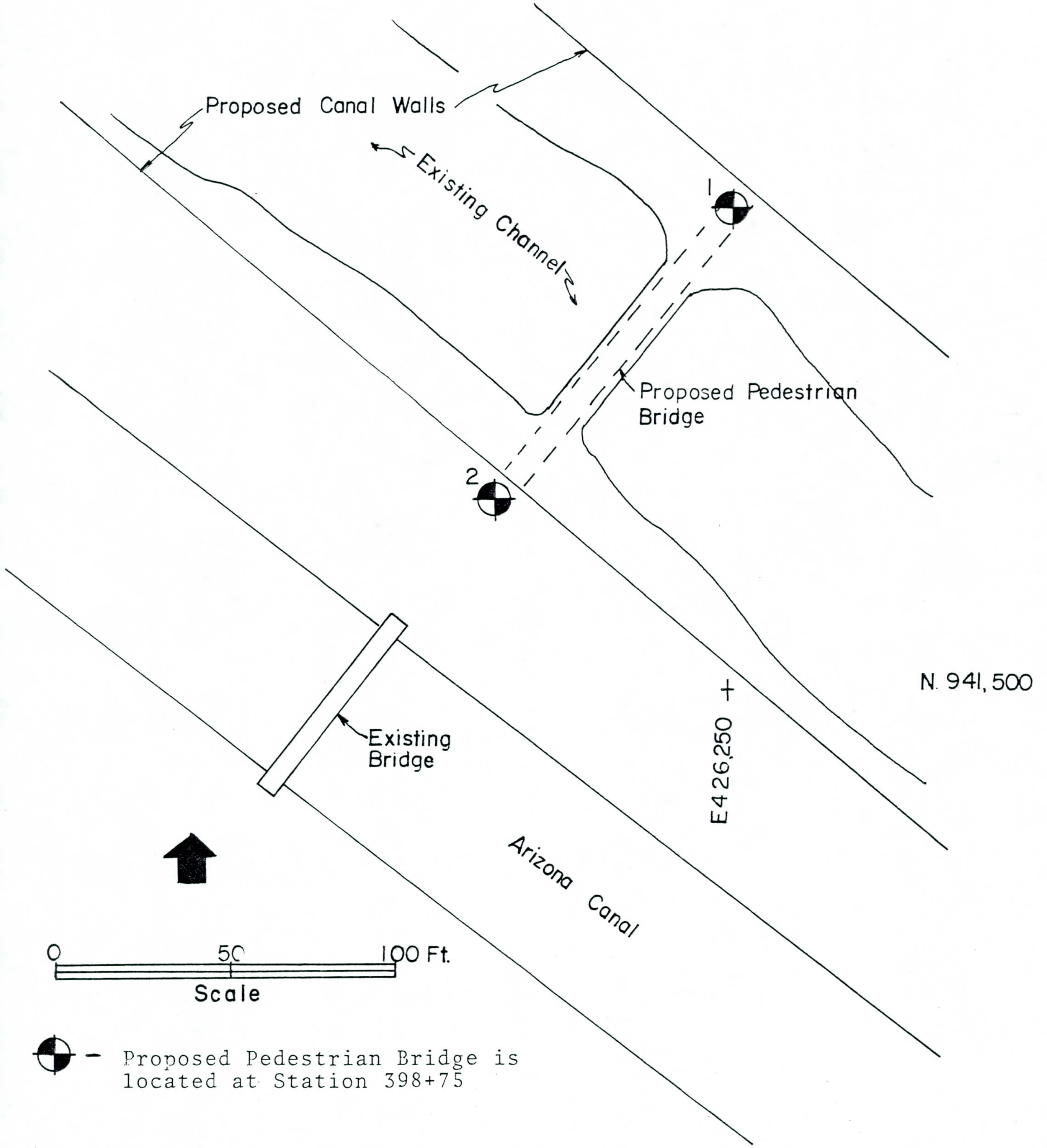
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.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

SITE PLAN

SHOWING LOCATIONS OF TEST BORINGS



- Proposed Pedestrian Bridge is located at Station 398+75

Reference Drawing: "Hydraulic Plan & Profile, Arizona Canal Diversion Channel" by U.S. Army Corps of Engineers, Los Angeles District, Plate 4, undated

Pedestrian Bridge
47th Avenue Over Arizona
Canal Diversion Channel
Phoenix, Arizona
SHB Job No. E86-199

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO

PROJECT Pedestrian Bridge

JOB NO. E86-199 DATE 9-22-86

LOG OF TEST BORING NO. 1

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1224'±

DATUM Topographic Map

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								
			☒	S	50/5"		6	SM
								SC
5			☒	S	19		29	
								CL
10			☒	S	67		14	
15			☒	S	79		3	
20			☒	S	98		3	SP
25			—	S	50/1½"		12	
30								

REMARKS	VISUAL CLASSIFICATION
slightly moist hard	FILL SILTY SAND, predominantly fine grained, low plasticity, brown
slightly moist firm	FILL CLAYEY SAND, considerable silt, predominantly fine grained, low plasticity, brown
slightly moist hard	SILTY CLAY, some sand, medium plasticity, brown note: moderately lime cemented to strongly lime cemented below 8'
dry to slightly moist very dense	SILTY SAND, some fine grained gravel, predominantly fine to medium grained, nonplastic, gray note: some coarse grained gravel & cobbles below 22'
	Auger refused at 28' on cobbles note: water was added to borehole before sampling, except at 6" to 1'5"

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

CONSULTING GEOTECHNICAL ENGINEERS

PHOENIX · ALBUQUERQUE · SANTA FE · SALT LAKE CITY · EL PASO

PROJECT Pedestrian Bridge

JOB NO. E86-199 DATE 9-22-86

LOG OF TEST BORING NO. 2

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1223'±

DATUM Topographic Map

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									moist to very moist very soft to moderately firm	SANDY CLAY, considerable silt, medium plasticity, brown
			X	S	15		15			
5			X	S	2		17	CL		
10			X	U	27	107	18			
15			X	S	76		5	SC	moist hard to firm below 19'	SAND, some fine grained gravel, well graded, angular, low to medium plasticity, brown note: coarse grained gravel & occasional cobble from 17' to 19' note: small amount of fine grained gravel below 19', predominantly fine grained, low plasticity
20			X	S	24		8			
25			X	S	44		6	SM	moist very firm to hard	SILTY SAND, some fine to coarse grained gravel, trace of clay & cobbles, angular, nonplastic, brown note: considerable cobbles below 28'
30				S	50/3"		8			
35										Auger refused at 31' on cobbles note: added water to borehole before sampling

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

CONSULTING GEOTECHNICAL ENGINEERS

PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY • EL PASO

SERGENT, HAUSKINS & BECKWITH

TABULATION OF TEST RESULTS

Job No. E86-199
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	9.5'-11'	CL	42	21	80	85	91	93	95	97	98	98	99	100			6-199-3
1	19.5'-20'11"	SM	NV	NP	12	16	23	29	37	53	63	67	77	81	86	91	
					100												6-199-5
2	14.5'-16'	SC	31	12	13	16	20	25	31	45	58	63	77	83	89	91	
					100												6-199-10

DESIGN OF DRILLED, CAST-IN-PLACE CONCRETE PIERS

I. SAFE DOWNWARD CAPACITY:

* Consider end bearing only

$P_D = Q_E A_E / F.S.$ = DESIGN CAPACITY, KIPS

Q_E = ULTIMATE END BEARING
- SHOWN IN FIG. C-1, p. 2

A_E = END AREA $= \pi d^2/4$
where d = pier diameter, feet

F.S. = FACTOR OF SAFETY = 2.5

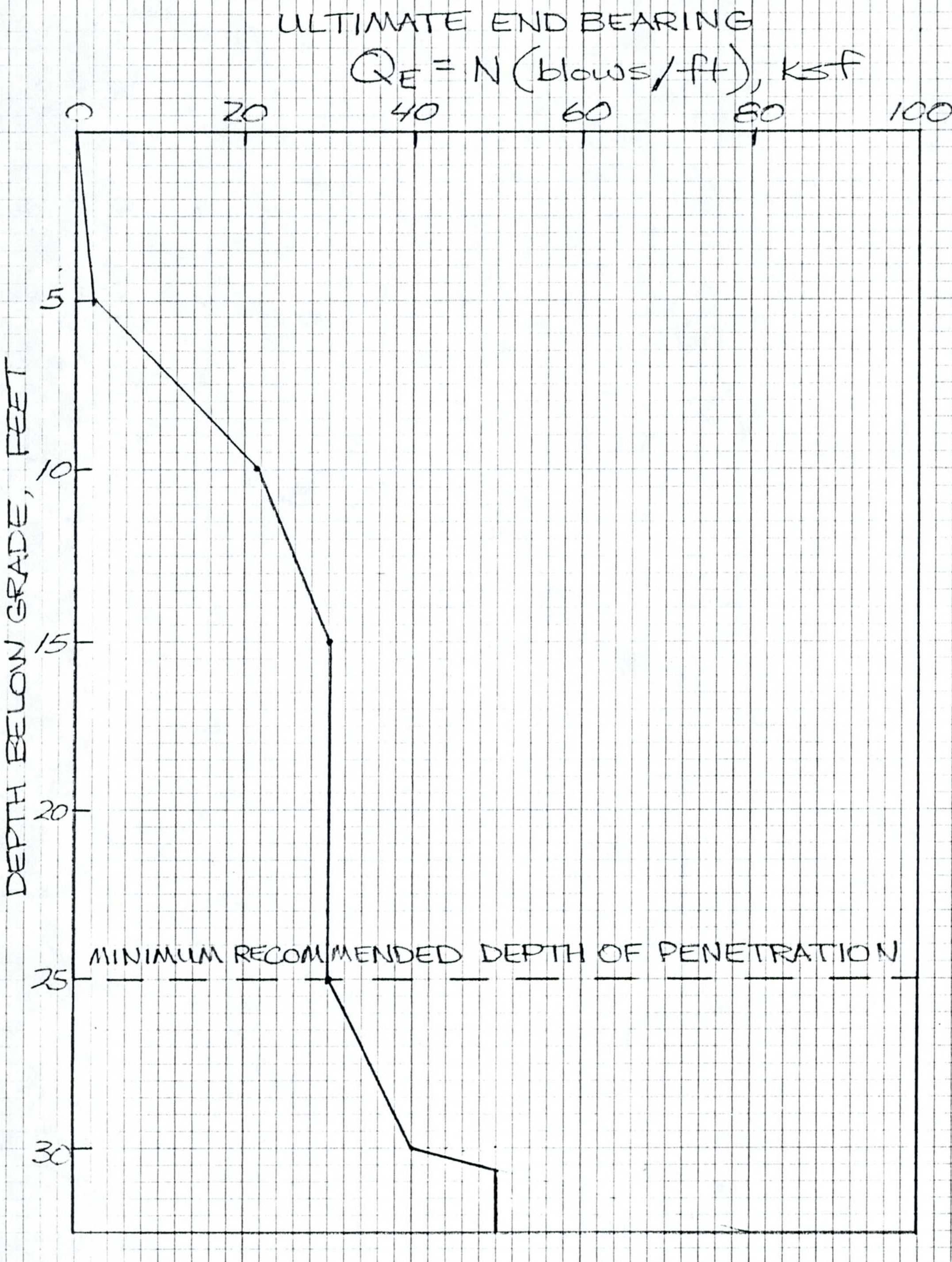
$\therefore P_D = 0.314\, Q_E d^2$, SEE TABLE C-2 FOR DESIGN CAPACITIES

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX · ALBUQUERQUE · SANTA FE · SALT LAKE CITY · EL PASO

Project: Pedestrian Bridge
Job No: E86-199
Computed by: NAW. Ckd. by: SSC
Date 10/16/86 Page 1 of

FIGURE C-1

DESIGN ULTIMATE END BEARING PROFILE FOR COMPUTING VERTICAL CAPACITY



SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

Project: Pedestrian Bridge
Job No: E86-199
Computed by: NHW Ckd. by: SSC
Date: 10/16/86 Page 2 of

TABLE C-1

DESIGN VERTICAL DOWNWARD CAPACITY OF DRILLED PIERS BASED ON END BEARING

PIER TIP DEPTH BELOW GROUND SURFACE, FEET	Q_E KSF	CAPACITY, P_D, KIPS d = 2.0 Ft.	d = 3.0 Ft.	d = 4.0 Ft.	d = 5.0 Ft.
25.0	30	38	85	151	235
27.5	35	44	99	176	275
30.0 AND BELOW	40	50	113	201	314

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX · ALBUQUERQUE · SANTA FE · SALT LAKE CITY · EL PASO

Project Pedestrian Bridge
Job No: E86-199
Computed by: N.HW. Ckd. by: SSC
Date 10/16/86 Page 3 of

II DESIGN OF BELLED, DRILLED, CAST-IN-PLACE CONCRETE PIERS

* SAFE DOWNWARD CAPACITY

CONSIDERING THE FUNCTION OF A BELL - USE END BEARING ONLY

RECALL FROM STRAIGHT PIER ANALYSIS

$$P_D = 0.314\, Q_E d^2$$

WHERE d = DIAMETER OF THE BELL, FEET
- USE TABLE C-1, AND EXTEND BY INCLUDING LARGER VALUES OF d.

TABLE C-2

DESIGN VERTICAL DOWNWARD CAPACITY OF BELLED PIERS BASED ON END BEARING

PIER TIP DEPTH FEET	Q_E KSF	CAPACITY, P_D, KIPS d = 4.0 FEET	d = 4.5 FEET	d = 5.0 FEET	d = 5.5 FEET
25	30	151	191	236	285
27.5	35	176	223	275	333
30.0 AND BELOW	40	201	254	314	380

III SETTLEMENT ANALYSIS OF STRAIGHT CAST-IN-PLACE CONCRETE PIERS

* CONSIDER END BEARING ONLY

$$\delta = \mu_0 \mu_1 (qd/E_s)$$

WHERE δ = SETTLEMENT

q = AVG. BEARING PRESSURE

$= Q/(\pi d^2/4)$

d = PIER DIAMETER

E_s = SOIL MODULUS

$\mu_0 \mu_1$ = DIMENSIONLESS PARAMETERS DEPENDENT ON DEPTH.

$$\therefore \delta = \mu_0 \mu_1 (4Q/\pi d E_s)$$

WHERE Q IS THE APPLIED AXIAL LOAD FOR A CIRCULAR FOUNDATION, $\mu_1 = 0.60$, μ_0 IS DEPENDENT ON THE RATIO D/d, WHERE D = DEPTH TO FOOTING BELOW FINISHED GRADE.

FIGURE C-2 SHOWS VARIATION OF μ_0 WITH DEPTH.

MODULUS VALUES ARE TAKEN FROM A RELATIONSHIP BETWEEN SPT BLOWCOUNT (N-VALUE) AND E_s BASED ON CEMENTED PHOENIX SOILS, PRESENTED IN BECKWITH, G.H. & HANSEN, L.A. "CALCAREOUS SOILS OF THE SOUTHWESTERN UNITED STATES", ASTM STP 777, 1982, PP 16-35.

THE FINAL WORKING RELATIONSHIP FOR SETTLEMENT CALCULATION

$$\delta = 0.6\, \mu_o \left(4/\pi\right)\left(Q/dE_s\right)$$

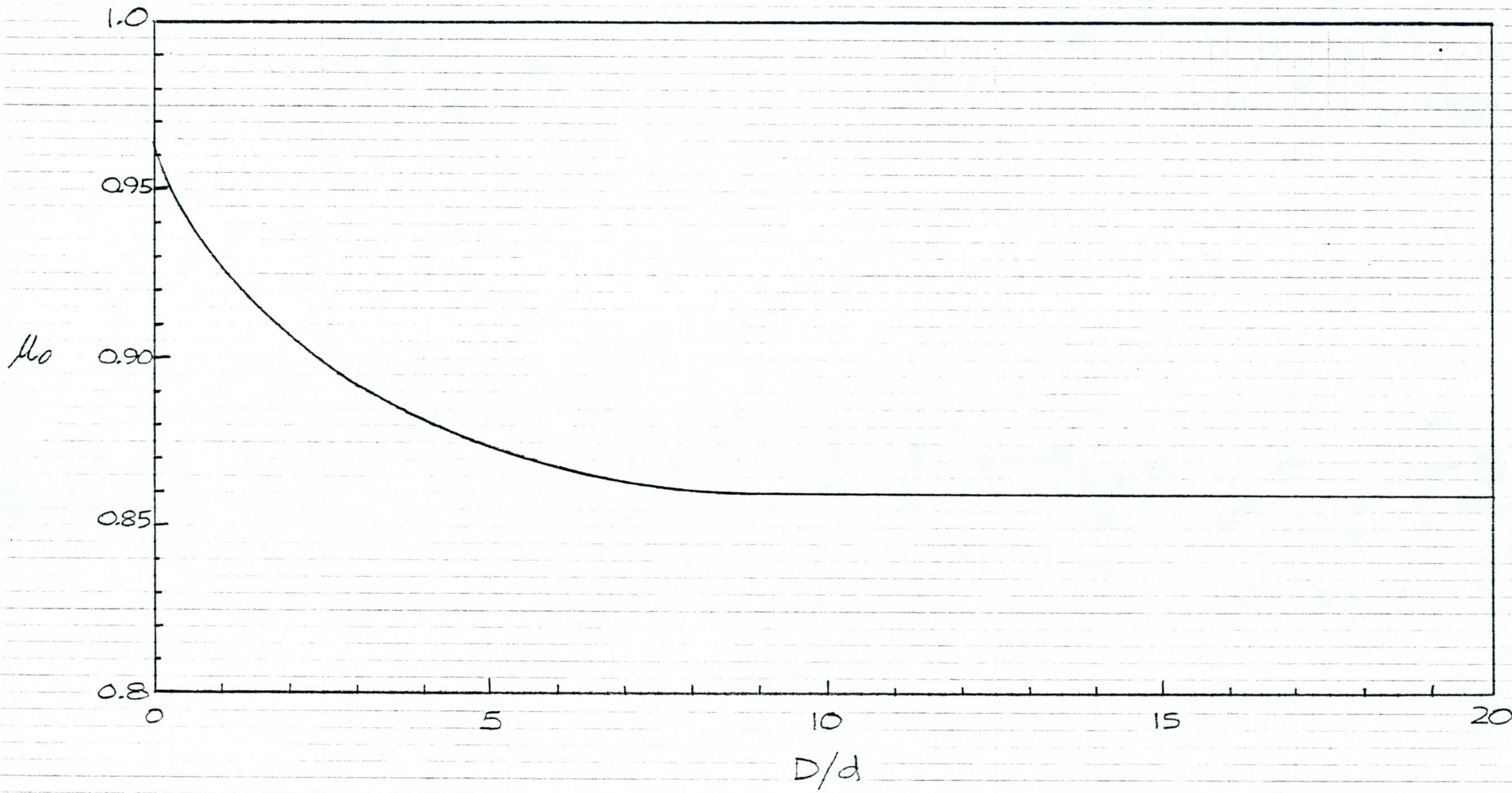
$$= 0.764\, \mu_o Q/dE_s$$

IF Q IS IN KIPS, d IN FEET AND E_s IN KSF, THE RELATIONSHIP FOR δ IN INCHES IS -

$$\delta = 9.17\, \mu_o Q/dE_s$$

FIGURE C-2

VARIATION OF μ_o WITH D/d



D= DEPTH TO FOOTING BELOW FINISHED GRADE

d= PIER DIAMETER

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PHOENIX · ALBUQUERQUE · SANTA FE · SALT LAKE CITY · EL PASO

Project Pedestrian Bridge

Job No: E86-199

Computed by: NHW Ckd. by: SSC

Date 10/16/86 Page 7 of

TABLE C-2 PRESENTS ESTIMATED SETTLEMENTS FOR ABUTMENT PIERS. SETTLEMENTS ARE PRESENTED AS INCHES/KIP VERTICAL LOAD. TO CALCULATE THE SETTLEMENT FOR A PARTICULAR VERTICAL LOAD FOLLOW THE PROCEDURE BELOW.

EXAMPLE:

PIER TIP ELEVATION = 1202
PIER DIAMETER = 2.5'
VERTICAL LOAD = 200 KIPS

$$\delta = (0.002)(200) = .4 \text{ INCHES}$$

FIGURES C-1 & C-2 CAN BE USED FOR EITHER STAIGHT SHAFTED OR BELLED PIERS SINCE ONLY END BEARING IS CONSIDERED.

SERGENT, HAUSKINS & BECKWITH
CONSULTING GEOTECHNICAL ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE • SALT LAKE CITY

Project Pedestrian Bridge
Job No: E86-199
Computed by: NHW Ckd. by: SSC
Date 10/16/86 Page 9 of

TABLE C-2

NORMALIZED SETTLEMENT IN INCHES/KIP VERTICAL LOAD
FOR ABUTMENT PIERS ASSUMING END BEARING ONLY

PIER TIP DEPTH FEET	Es KSF	SETTLEMENT, IN/KIP dia = 2.0 FEET	3.0 FEET	4.0 FEET	5.0 FEET
25	2000	.0016	.0013	.0010	.0008
27.5	3000	.0013	.0009	.0007	.0005
30.0	3500	.0011	.0008	.0006	.0005