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GEOTECHNICAL INVESTIGATION REPORT

Broadway Road Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona

SHB Job No. E83-103
Addendum No. 2

Consulting Geotechnical Engineers

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February 20, 1984

RGA Consulting Engineers, Inc.
1102 West Indian School Road
Phoenix, Arizona 85013

SHB Job No. E83-103
Addendum No. 4

Attention: Harold E. Ditzler, P.E.

Re: Broadway Road Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona

Gentlemen,

In accordance with your additional request, we are herein presenting the results of a lateral load analysis conducted for the referenced project.

1. Introduction

Lateral load analyses were conducted on single-pier models for three cases; abutment piers, interior piers with no scour, and interior piers with 9 feet of scour. The boundary and loading conditions modeled in each case were provided by Keith D. Zwickl, P.E., of RGA Consulting Engineers, Inc. The object of the lateral analyses was to determine pier top deflection and estimate depth

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below the ground surface to the point of fixity of the pier structure.

2. Methodology of Lateral Analysis

Each pier model was analyzed for both fully cohesive and fully cohesionless soil conditions. Input strength parameters were C = 1.0 ksf, Ø = 0 and C = 0, Ø = 30° for the cohesive and cohesionless conditions, respectively.

The procedure developed by Matlock (1970)* was utilized in p-y curve construction. The ultimate lateral soil bearing pressure for use in p-y curve construction was calculated using Broms' (1965) procedure. Pier top deflection and point of pier fixity (assumed to be the point of maximum moment below the pier top) were determined utilizing the computer program COM 622 (Reese, 1977).

A brief summary of our analysis and illustrations of the modeled pier configurations are attached.

This addendum should be attached to the original report and made a part thereof.

*References are listed at end of addendum.

Should any questions arise concerning this addendum, please do not hesitate to contact us.

Respectfully submitted,

Sergent, Hauskins & Beckwith Engineers

By James R. Fahy
James R. Fahy, E.I.T.

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

Registered Professional Engineer (Civil)
CERTIFICATE NO. 13872
LAWRENCE A. HANSEN
Date Signed 2-20-84
ARIZONA, U.S.A.

Copies: Addressee (4)

REFERENCES

Broms, B.B., 1965, "Design of Laterally Loaded Piles", ASCE, JSMFD, Volume 91, No. SM3, May, pp. 79-99.

Matlock, H., 1970, "Correlations for Design of Laterally Loaded Piles in Soft Clay", Offshore Technology Conference, April.

Reese, L.C., 1977, "Laterally Loaded Piles: Program Documentation", ASCE, JGED, Volume 103, No. GT4, April.

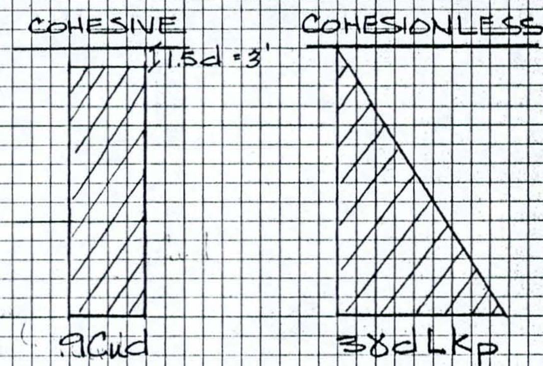
LATERAL ANALYSIS TO DETERMINE PIER TOP DEFLECTION AND ESTIMATE POINT OF FIXITY.

CASE I: ABUTMENTS

MODEL:

NOTE: AXIAL LOAD ASSUMES 4 PIERS SUPPORTING THE ABUTMENT.

ULTIMATE LATERAL RESISTANCE



A. COHESIVE CASE $\phi = 0$

$C_u = 1.0$ KSF

$P_{ULT} = 9C_u d$ WHERE: d = PIER DIAMETER = 2.0'

$P_u = 9 \cdot 1 \cdot 2 = 18$ K/FT. = 1500 lb/IN.

B. COHESIONLESS CASE: $\phi = 30°$, $C = 0$; $\gamma = 120$ PCF

$P_u = 3\gamma d L K_p$

$K_p = \tan^2(45 + \phi/2) = 3.00$

d = 2'

DEPTH	P_u (lb/IN.)
2'	360
4'	720
8'	1440
15'	2700
20'	3600
35'	6300

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Date 2/14/84 Page 1 of

P-Y CURVE DEVELOPMENT:

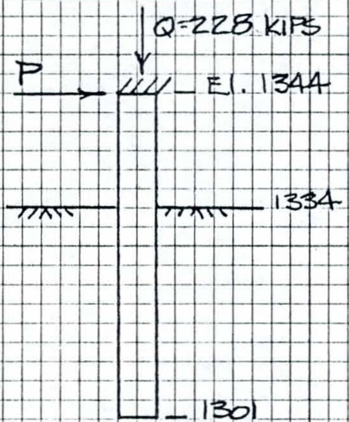
ASSUME: $\epsilon_{50} = 1.0\%$; $Y_c = 2.5\,\epsilon_{50}\,d$

$= 2.5(0.01)(2')\cdot 12\frac{IN}{FT} = 0.6''$

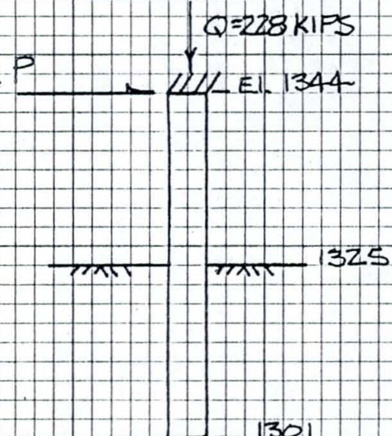
Y/Y_c	P/P_{ULT}	Y IN.	COHESIVE CASE, ALL DEPTHS (P)	COHESIONLESS CASE DEPTH BELOW SURFACE & P (lb/in) 2' (P)	4' (P)	8' (P)	15' (P)	20' (P)	35' (P)
0.1	0.23	0.06	345	83	166	331	621	828	1449
0.25	0.32	0.15	480	115	230	461	864	1152	2016
0.5	0.40	0.3	600	144	288	576	1080	1440	2520
1.0	0.50	0.6	750	180	360	720	1350	1800	3150
2.0	0.63	1.2	945	227	454	907	1701	2268	3969
4.0	0.79	2.4	1185	284	569	1138	2133	2844	4977
8.0	1.0	4.8	1500	360	720	1440	2700	3600	6300

FLEXURAL RIGIDITY : $EI = 3156000\,psi \cdot \frac{\pi(24)^4}{64} = 5.14\times10^{10}\ lbs\text{-}in^2$

CASE 2



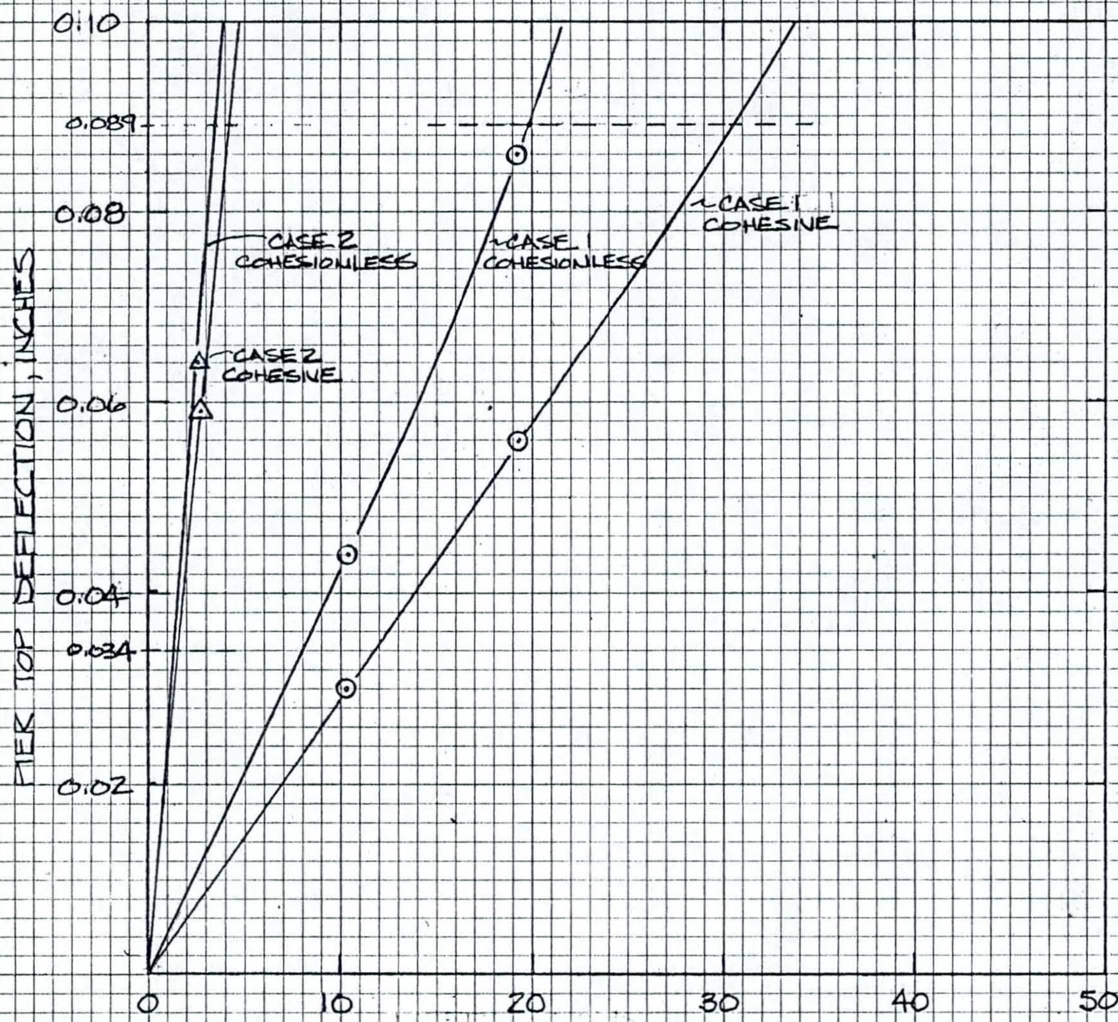
CASE 3



NOTE: AXIAL LOAD ASSUMES 6 PIERS
AT EACH PIER BENT.

FIGURE 1

LATERAL LOAD VERSUS DEFLECTION
FOR CASE 1 & 2 CONDITIONS



FIGURES 1 & 2 CAN BE USED TO DETERMINE LATERAL FORCE NEEDED TO CAUSE DEFLECTIONS OF 0.034" & 0.089" FOR TEMPERATURE FORCE LOADING CONDITIONS INDICATED ON LETTER OF TRANSMITTAL, RGA CONSULTANTS, 2-14-84.

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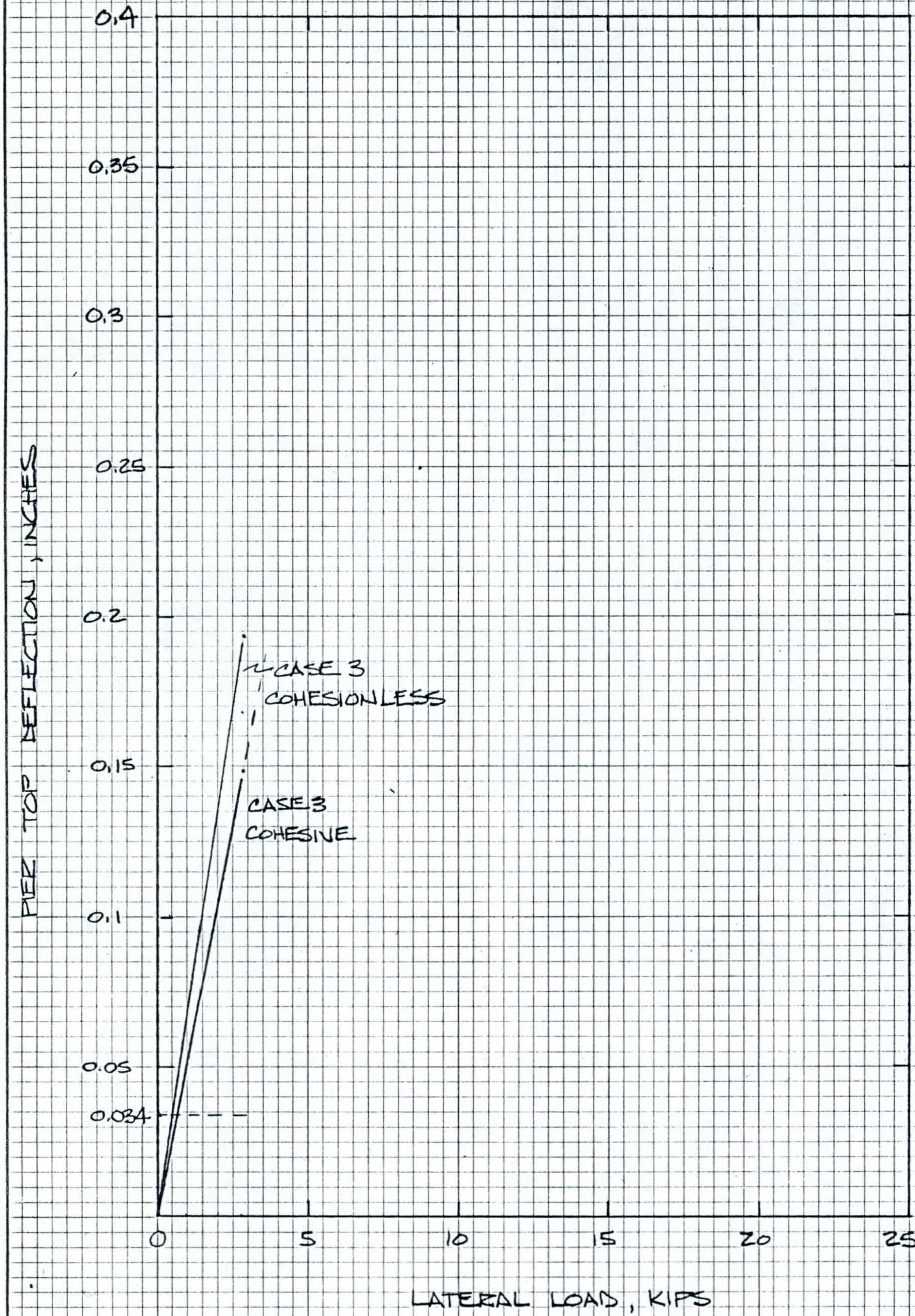
FIGURE 2
LATERAL LOAD VERSUS DEFLECTION
FOR CASE 3



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SUMMARY OF LATERAL LOAD ANALYSES FOR BROADWAY ROAD BRIDGE PIERS

CASE	LATERAL LOAD KIPS	DEFLECTION, Δ, INCHES: ABUTMENT PIERS	DEFLECTION, Δ, INCHES: INTERIOR PIERS NO SCOUR	DEFLECTION, Δ, INCHES: INTERIOR PIERS 9' SCOUR	DEPTH BELOW GROUND LINE TO POINT OF FIXITY, FEET
1-COHESIVE	30.5	0.09			10
	10.3	0.030			10
	19.3	0.056			10
1-COHESIONLESS	19.9	0.089			11
	10.3	0.044			11
	19.3	0.086			11
2-COHESIVE	4.2		0.089		6
	8.6		0.182		6
	2.8		0.059		6
2-COHESIONLESS	3.6		0.089		7
	8.6		0.202		7
	2.8		0.064		7
3-COHESIVE	0.65			0.034	3
	8.6			0.462	3
	2.8			0.148	3
3-COHESIONLESS	0.5			0.034	6
	8.6			0.632	6
	2.8			0.193	6

rga consulting engineers

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LETTER OF TRANSMITTAL

DATE 2-10-84	JOB NO. 83029
ATTENTION Nick Karan	
RE:	

TO Maricopa County Flood Control District

GENTLEMEN:

WE ARE SENDING YOU ☒ XX Attached ☐ Under separate cover via ______ the following items:

☐ Shop drawings ☐ Prints ☐ Plans ☐ Samples ☐ Specifications

☐ Copy of letter ☐ Change order ☐ ______

COPIES	DATE	NO.	DESCRIPTION
2	1-26-84		Revised Geotechnical Investigation Report

THESE ARE TRANSMITTED as checked below:

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☒ x For your use ☐ Correct as noted ☐ Submit ____ copies for distribution

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☐ For review and comment ☐ ______

☐ FOR BIDS DUE ______ 19 ____ ☐ PRINTS RETURNED AFTER LOAN TO US

REMARKS ______

FLOOD CONTROL DISTRICT
RECEIVED

FEB 10 '84

CH ENG	HYDRO
ASST	LMgt
ADMIN	SB
C & O	FI
1 ENGR APM	DESIGN
FINANCE	2 ER
REMARKS	3 CGF

COPY TO ______

SIGNED: Jay E. Mihalek, P.E.
Project Manager

If enclosures are not as noted, kindly notify us at once.

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING

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ALLON C. OWEN, JR., P.E.
ROBERT D. BOOTH, P.E.
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RALPH E. WEEKS, P.G.

January 26, 1984

RGA Consulting Engineers, Inc.
1102 West Indian School Road
Phoenix, Arizona 85013

SHB Job No. E83-103
Addendum No. 2

Attention: Harold E. Ditzler, P.E.

Re: Broadway Road Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona

Gentlemen,

Our revised Geotechnical Investigation Report on the referenced project is herewith submitted. The report includes the results of test drilling, laboratory analysis and recommended criteria for foundation design. This report replaces our initial submittal dated August 18, 1983 and Addendum No. 1 dated December 7, 1983.

The recommendations presented herein are based on methodology and interpretations specified in part by the Flood Control District of Maricopa County, as outlined in our letter to you dated January 23, 1984. We do not agree with all interpretations, but we do believe they will result in a safe design.

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SALT LAKE CITY (801) 566-5411

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

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By Lawrence A. Hansen
Lawrence A. Hansen, Ph.D., P.E.

Professional Engineer (Civil)
Certificate No.
13872
LAWRENCE A.
HANSEN
Date Signed 1-26-84
ARIZONA, U.S.A.

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

Professional Engineer (Civil)
Certificate No.
15633
GEORGE H.
BECKWITH
Date Signed 1-26-84
ARIZONA, U.S.A.

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REPORT

SHB Job No. E83-103

Broadway Road Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona
SHB Job No. E83-103
Addendum No. 2

1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed Broadway Road Bridge over the Roosevelt Water Conservation District (RWCD) Floodway located in Mesa, 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 Jay E. Mihalek, P.E., and Mihai Harabor, P.E., of RGA Consulting Engineers, Inc.

It is understood that a three-span, two-lane highway bridge will cross the proposed Roosevelt Water Conservation District Floodway. The bridge will be approximately 68 feet wide and 95 feet in length, as shown in the site plan, Appendix A.

The piers will be parallel with the flow and deep enough to permit overexcavation of up to 5 feet to permit lowering of the invert and installation of rock riprap if necessary. The bridge will be centered on the centerline of the proposed floodway and on the centerline of the existing roadway or the monument line as determined by the City of Mesa.

The total vertical load on each abutment will be approximately 865 kips (620 kips DL and 245 kips LL). The total vertical load on each pier bent will be approximately 1,370 kips (860 kips DL and 510 kips LL). It is estimated that each pier will carry a total vertical load of 100 to 160 kips.

The channel is to be earthlined and trapezoidal in section with two to one side slopes and 30-foot bottom width, as shown in Figure 1. The design flow rate is 2,300 cubic feet per second at a depth of approximately 7.2 feet and a velocity of 6.3 feet per second. The channel invert elevation will be 1334.0 feet.

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

Three exploratory borings were drilled to depths of 28 to 50 feet below existing grade. The borings were performed using 6 5/8-inch O.D. hollow stem auger. Standard penetration testing was performed at 5-foot intervals in the borings. In two of the borings, the hole was maintained full of water during standard penetration testing.

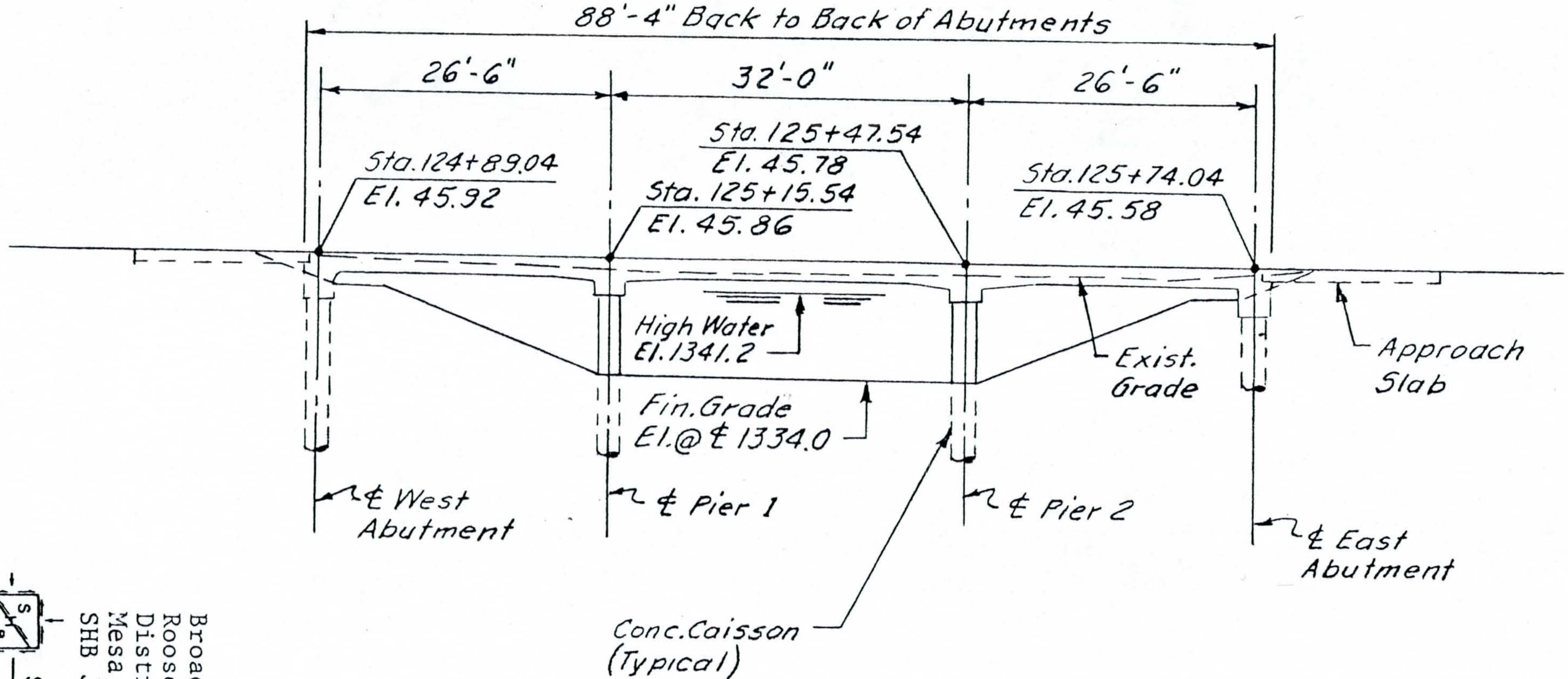
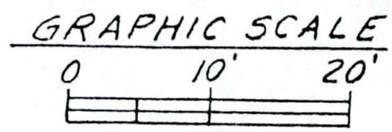


FIGURE 1

SECTION ON CENTERLINE OF ROADWAY



Broadway Road Bridge Over
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District (RWCD) Floodway
Mesa, Arizona
SHB Job No. E83-103

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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 Norman H. Wetz, P.E., of this firm.

3.2 Laboratory Analysis

Moisture content determinations were made on all standard penetration test samples recovered from borings 1 and 2. The results of these tests are shown on the boring logs, and in a graphical summary in Appendix B.

Grain-size analysis and Atterberg Limits tests were performed on two selected samples to aid in soil classification. The results of these tests are presented in Appendix B.

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

An existing irrigation canal is located immediately to the west of this bridge site. Also, an existing drainage canal with a small box culvert is located on the western portion of the bridge site. Broadway Road is existing and is a paved two-lane highway.

4.2 Geotechnical Profile

The subsurface soils consist of a surface layer of man-made fill that extends approximately 2½ feet below existing grade at the location of boring 1. This material is a clayey sand of low plasticity and was found to be relatively firm. Sandy clay underlies the man-made fill and is exposed at the surface at the location of boring 2. This material extends to about 18 feet below existing grade. The sandy clay is weakly to moderately cemented and firm to hard. Clayey sand extends from about 18 feet to the full depth of the borings. This material is moderately lime cemented and is hard.

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.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

Some of the near surface soils are somewhat moisture sensitive and would be weakened with substantial moisture increases. Thus, for the loads involved in this type of structure, excessive settlements could be experienced for shallow spread-type footings bearing upon the near surface soils at the site. Therefore, it is recommended that the main structure elements be supported on straight, machine-cleaned, cast-in-place concrete piers bearing a minimum of 15 feet below finished grade. Alternatively, belled, hand-cleaned, cast-in-place concrete piers bearing at or below elevation 1327 could be used.

1334
15
1319

Drilled foundations would be less affected by moisture increases, and the possibility of moisture increase at bearing depths would be more remote. Design criteria are presented in Section 5.2 for drilled piers.

5.2 Cast-in-Place Concrete Piers

5.2.1 Downward Loads

Straight, machine-cleaned, drilled, cast-in-place concrete piers are recommended for the support of the foundation loads involved. Safe downward capacities of piers extending a minimum of 15.0 feet below the finished grade elevation are presented in Figure 2 for abutment piers and in Figure 3 for interior piers. Methodology and input design parameters used in analysis of drilled pier capacities are outlined in Appendix C. Capacities shown in Figures 2 and 3 are based on side shear resistance only.

Safe downward capacities of belled piers are shown in Figure 4. The capacities were calculated assuming end-bearing only. Methodology and input design parameters are outlined in Appendix C.

FIGURE 2

SAFE DOWNWARD CAPACITY OF STRAIGHT, CAST-IN-PLACE DRILLED PIERS FOR ABUTMENTS

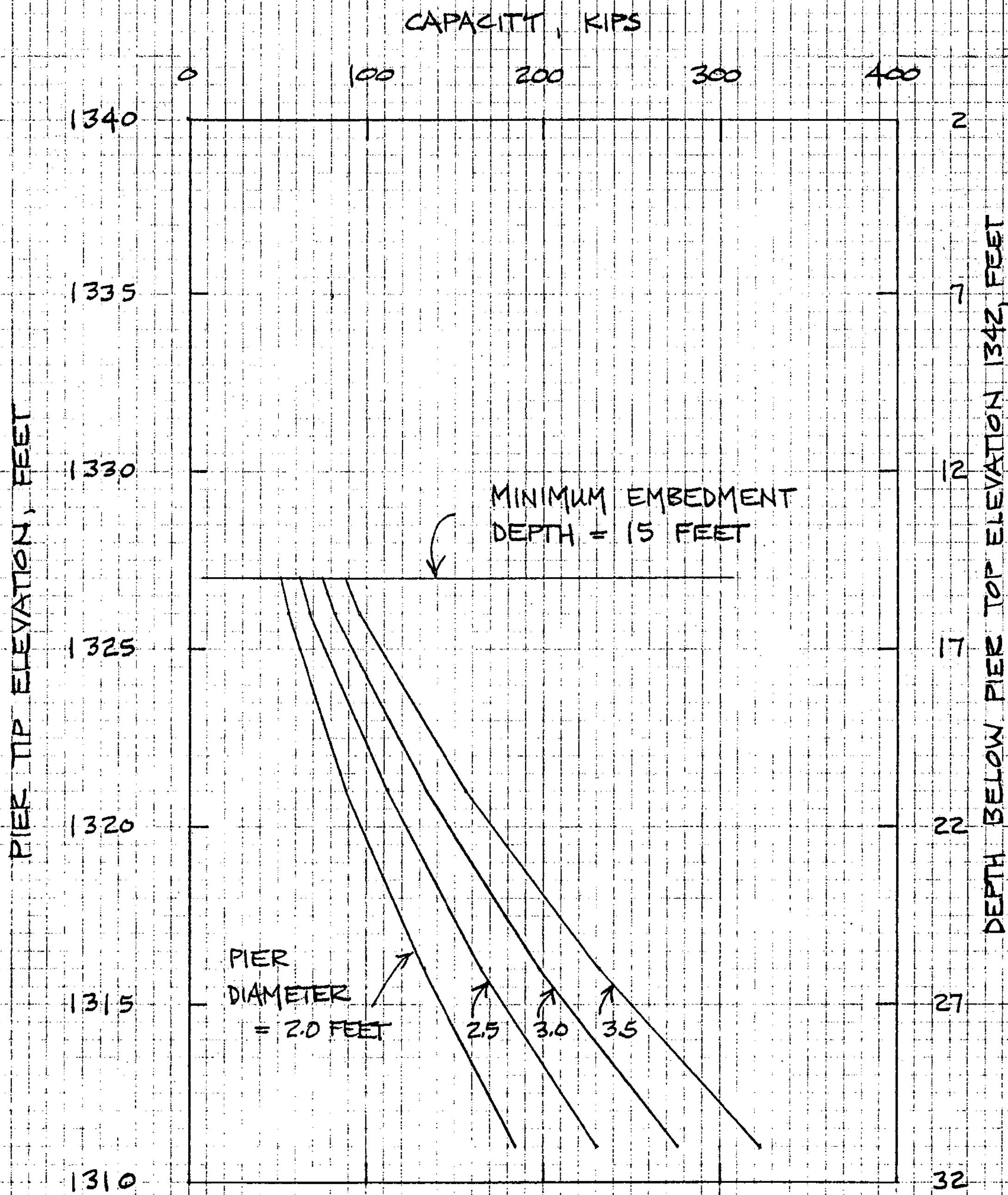
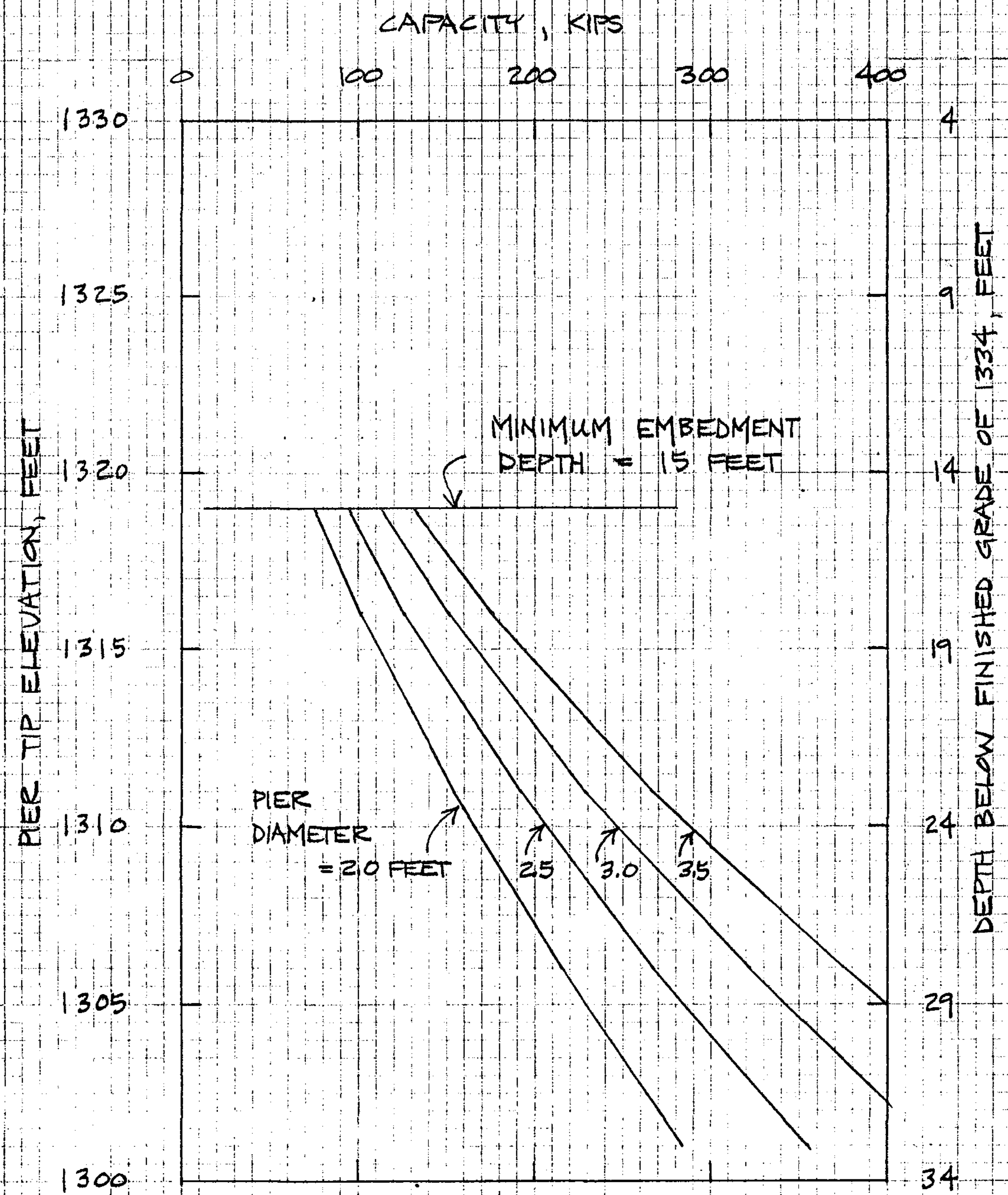


FIGURE 3

SAFE DOWNWARD CAPACITY OF STRAIGHT, CAST-IN-PLACE DRILLED PIERS FOR PIER LOCATIONS 1 & 2



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Project BROADWAY ROAD BRIDGE / RWCD
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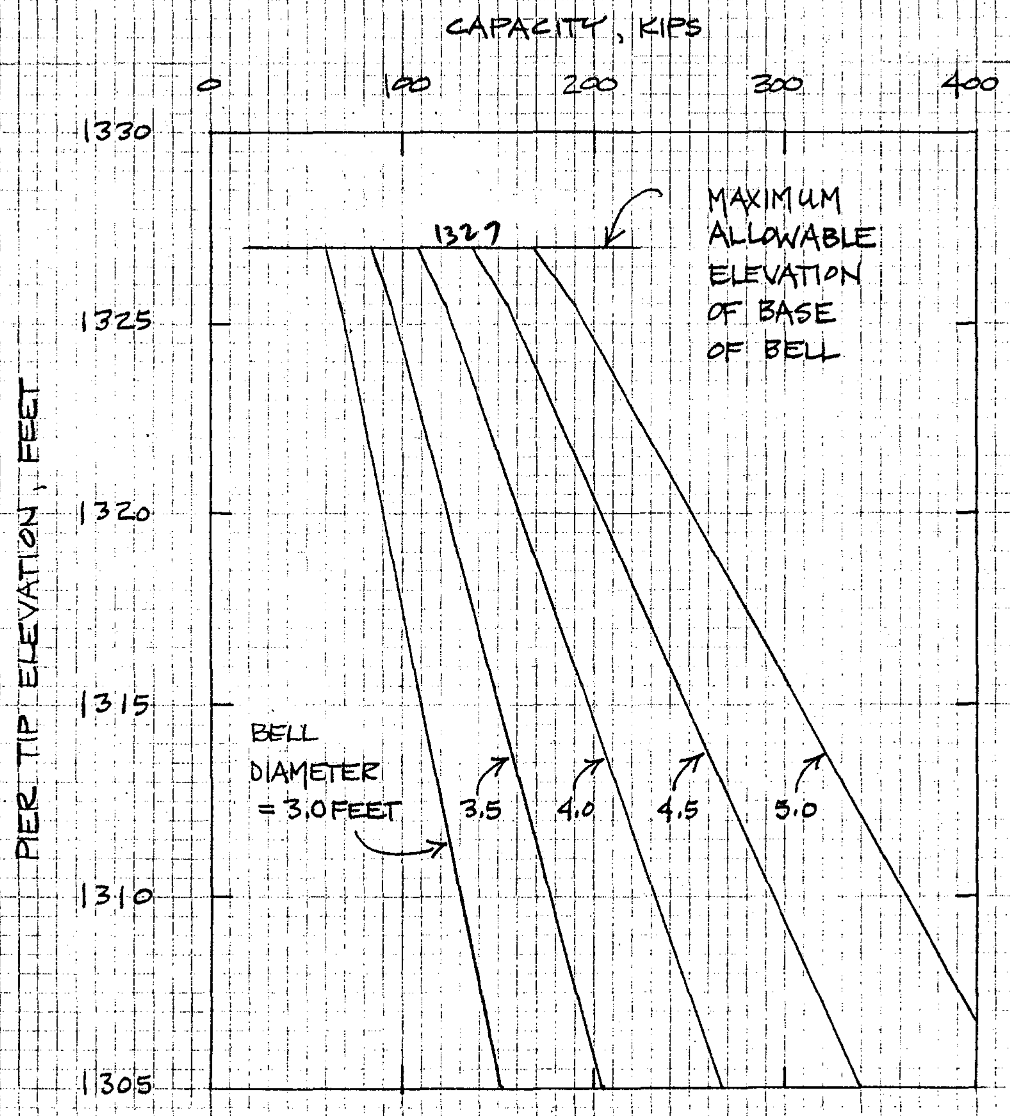
Date 1/22/84 Page ____ of ____

FIGURE 4

SAFE DOWNWARD CAPACITY OF BELLED, CAST-IN-PLACE DRILLED PIERS



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 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 both straight shafted and belled piers, incorporating the pier diameter and the pier tip elevation.

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 soil is to 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

*References are listed at end of report.

resistance are $\emptyset = 30^{o}$ 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 Broms' procedures requires, also, 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. Thus, for a 24-inch diameter pier, $k_h = 19$ 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.

The above criteria apply to both straight shafted and belled piers. For belled piers, the lateral resistance can be conservatively estimated by using the diameter of the shaft for the total length of the pier, thus ignoring the larger diameter of the bell.

A more rigorous analysis would include the variation in diameter.

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. The auger should be placed back in the holes and two additional passes made to clean loose material present in the bottom of the holes.

All loose material should be cleaned from the base of drilled-and-belled piers so that undisturbed native soil is exposed throughout. Manual cleaning of belled piers will be necessary for adequate removal of loose disturbed material. 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 the materials encountered in the pier excavations. Concrete placement should be continuously observed by the inspector to insure 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. All belled piers should be entered and observed by the geotechnical engineer's representative for verification of cleaning and contact with proper bearing material.

It appears that straight shafted, drilled pier excavations can be advanced to the depths recommended with little or no caving. Since caving is expected to be very minimal, concrete quantities may be very near the neat volume indicated by the plans. As noted above, inspection of belled pier excavations will likely require casing to preclude any possibility of caving.

5.3 Abutment Wall Design Criteria

Free draining granular backfill should be utilized behind the abutments and in roadway approach fills. This material should consist of sand and gravel, and have no more than 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 on the degree of restraint. Rigid, absolutely restrained abutments would be subjected to earth pressures represented by a hydrostatic load diagram of about 50 pounds per square foot per foot of depth. Rotation or lateral translation of the walls equal to about 0.001 times the height would reduce earth pressures to the active state of about 30 pounds per square foot per foot of depth. Slight lateral translation equal to about 0.0005 times the height would result in an intermediate pressure diagram on the order of 40 pounds per square foot per foot of depth.

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.

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.

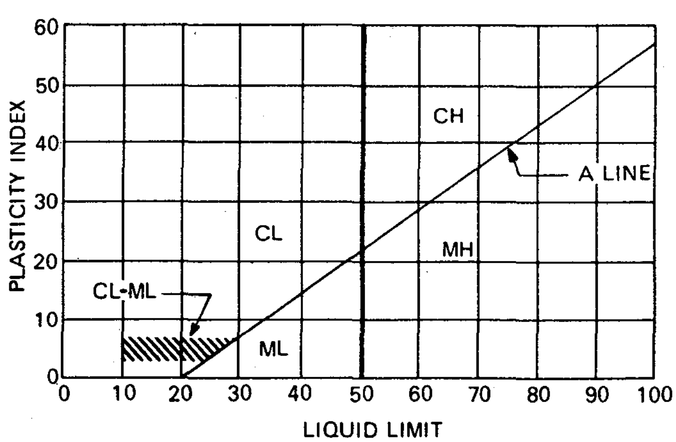
UNIFIED SOIL CLASSIFICATION SYSTEM

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.

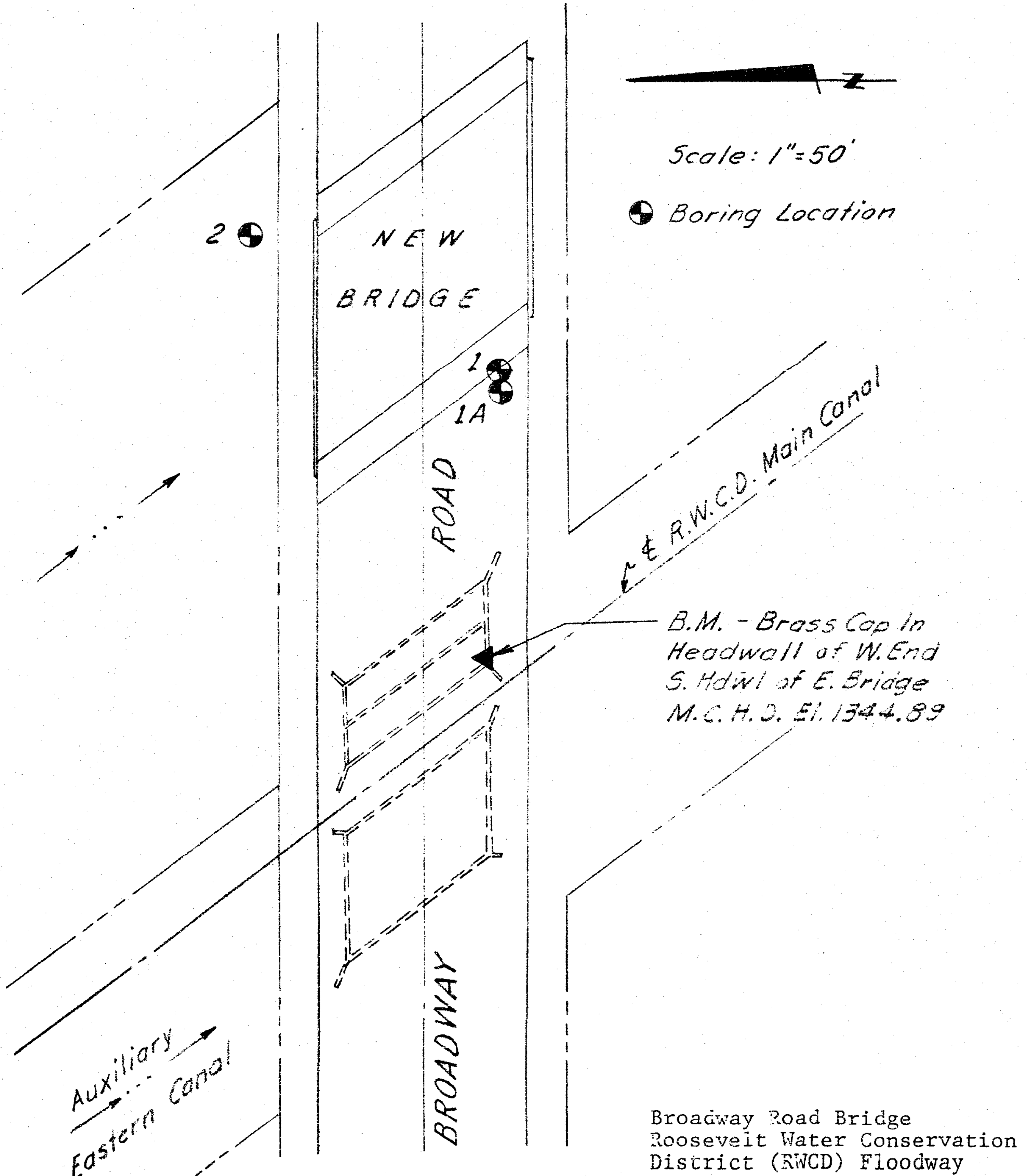
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.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



Reference Drawing: "Plan & Profile Sheet C-1, Broadway Road Bridge Over RWCD Floodway", Flood Control District of Maricopa County, Dated 12-83

Broadway Road Bridge
Roosevelt Water Conservation District (RWCD) Floodway
Mesa, Arizona
SHB Job No. E83-103

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CONSULTING SOIL AND FOUNDATION ENGINEERS
PHOENIX • TUCSON • ALBUQUERQUE

PROJECT Broadway Road Bridge Over RWCD Floodway

LOG OF TEST BORING NO. 1

JOB NO. E83-103 DATE 8-5-83

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1343.6'

DATUM MCHD

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								
1			X	S	19		5	SC
5			X	S	27		12	
10			X	S	79		8	CL
15			X	S	69		9	
20			X	S	51		7	
23								SC
25			X	S	50/5½"		10	
28			X	S	50/4½"		12	
30								

REMARKS	VISUAL CLASSIFICATION
dry firm	Man-made FILL CLAYEY SAND, some gravel, low plasticity, brown
slightly moist to dry firm to hard	SANDY CLAY, weakly to moderately lime cemented, medium plasticity, brown to light brown
slightly moist hard	CLAYEY SAND, well graded, angular, moderately lime cemented, low plasticity, light brown
	Auger refused at 28' Sampler refused at 28'10½"

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.

A-5

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PROJECT Broadway Road Bridge Over RWCD Floodway

LOG OF TEST BORING NO. 1A

JOB NO. E83-103 DATE 11-22-83

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1343.6'

DATUM MCHD

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								SC	dry	Man-made FILL CLAYEY SAND, some gravel, low plasticity, brown
			⊠	S	15					
5			⊠	S	14				moist to slightly moist moderately firm to hard	SANDY CLAY, weakly to moderately lime cemented, medium plasticity, brown to light brown
10			⊠	S	65			CL		
15			⊠	S	50/4½"					
20			⊠	S	30				slightly moist firm to hard	CLAYEY SAND, well graded, angular, moderately lime cemented, low plasticity, light brown
25			⊠	S	47			SC		
30			⊠	S	165					
35										Stopped auger at 30' Sampler refused at 31'2½"
										note: borehole maintained full of water during SPT test

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.

A-6

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PROJECT Broadway Road Bridge Over RWCD Floodway

LOG OF TEST BORING NO. 2

JOB NO. E83-103 DATE 1-18-84

RIG TYPE CME-55
BORING TYPE 6½" Hollow Stem Auger
SURFACE ELEV. 1342.5'
DATUM MCHD

Depth in Feet	Continuous Penetration Resistance	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
1		X	S	16		6	CL
5		X	S	44		4	
10		X	S	76		8	
15		X	S	107		15	
17							CL
20		X	S	50/5"		14	
25		X	S	50/5"		23	
30		X	S	145		9	
35		X	S	61		4	SC
40			S	50/5 3/4"		10	
43							CL
45			S	50/4"		12	
50			S	50/2"		9	

REMARKS	VISUAL CLASSIFICATION
slightly moist firm	SANDY CLAY, low plasticity, reddish brown
slightly moist to dry very firm to hard	SANDY CLAY, moderately to strongly lime cemented, low to medium plasticity, light brown
slightly moist hard	CLAYEY SAND, some gravel, well graded, moderately lime cemented, low plasticity, light brown
slightly moist hard	SANDY CLAY, moderately to strongly lime cemented, low plasticity, light brown
	Stopped auger at 49'6" Sampler refused at 49'8" note: borehole maintained full of water during SPT test

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

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

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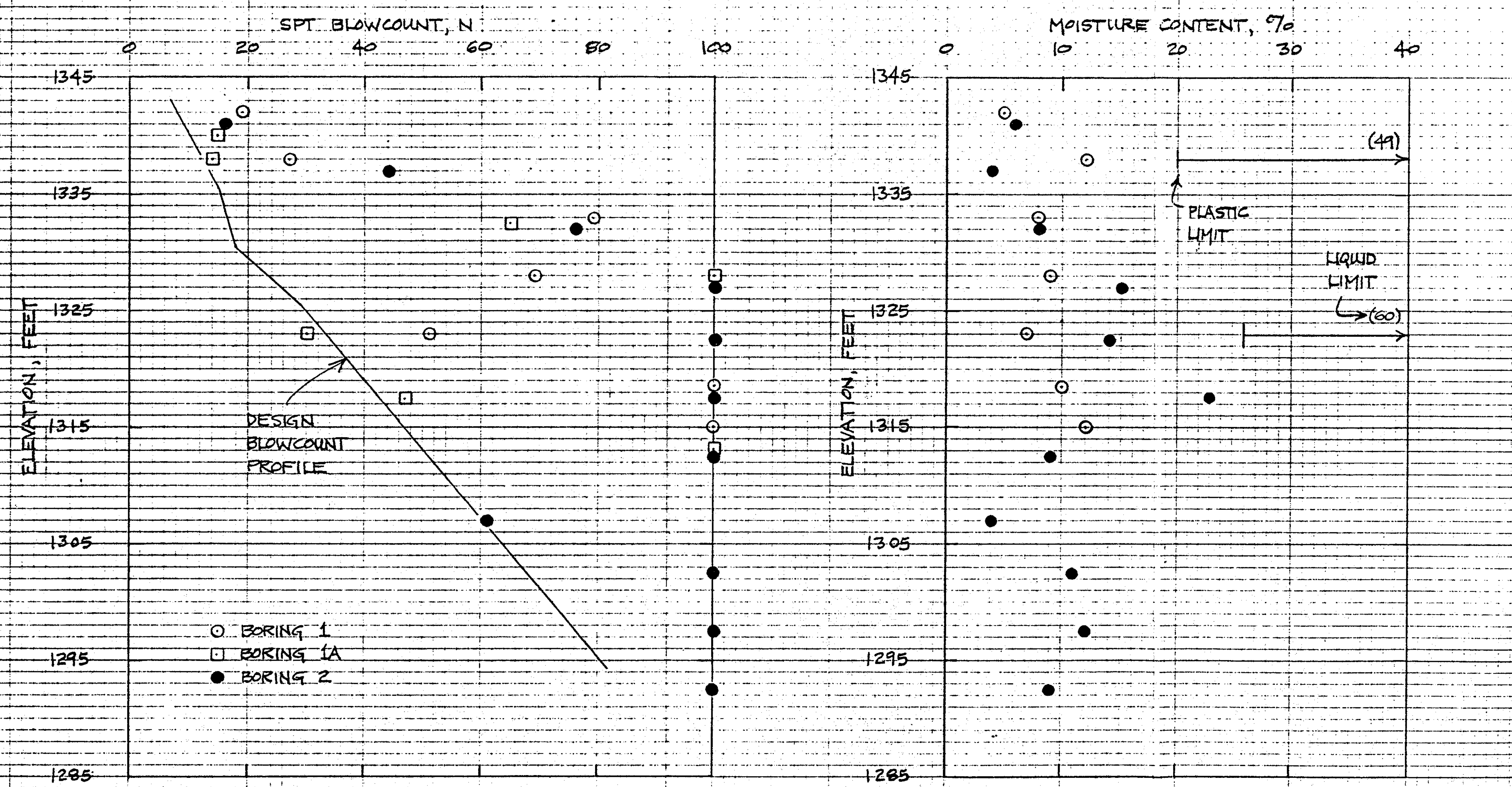
TABULATION OF TEST RESULTS

Job No. E83-103

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#200 .75"	#100 1"	#50 1.5"	#40 2"	#30 2.5"	#16 3"	#10 3.5"	#8 4"	#4 6"	.25" 8"	.375" 10"	.5" 12"	LAB NO
					SIEVE ANALYSIS-ACCUM % PASSING												
1	AT-5'	CL	49	29	65.8	77	84	85	87	93	96	97	100				3-103-2
1	AT-20'	SC	60	34	43.4	49	54	57	60	69	78	80	93	97	100		3-103-5

FIGURE B-1

SUMMARY OF GEOTECHNICAL DATA



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$$\begin{array}{r} 34 \\ 27 \\ \hline 7 \end{array}$$

$q_u = 3.2$

$c = \frac{3.2}{2} = 1.6$

$\pi \times \frac{5^2}{4} \times 3.2 = 62.8^T$

$125^T < 170$

$\pi \times \frac{5^2}{4} \times \frac{24}{6} = 78.5$

$\times 2 = 157$

$$\begin{array}{r} 45 \\ 27 \\ \hline 18 \end{array}$$

$N = 24$

$N_1 = 42$

$\Delta = 1 \times \frac{4}{5.5} = .73$

$q_{allow} = 5.\cancel{8}^T$

$.003 \times 157 = .48$

$q_{allow} = \frac{9 \times 1.6}{3} = \quad = 4.8$

I. SAFE DOWNWARD CAPACITY OF STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS

A. CONSIDER END-BEARING ONLY

$P_D = Q_E A_E / FS$ = DESIGN CAPACITY IN KIPS

Q_E = ULTIMATE END-BEARING FROM FIG. C-1, p.2, KIPS

A_E = END AREA = $\pi d^2/4$

d = PIER DIAMETER IN FEET

FS = SAFETY FACTOR = 3.0

$P_D = 0.262\, Q_E\, d^2$

CONSIDER PIER DIAMETERS OF 2, 2.5, 3.0 & 3.5 FEET BASED ON RGA CONSULTING ENGINEERS, INC. PRELIMINARY LOCATION PLAN & ELEVATION (SHEET S-1, DATED 1/17/84), EAST & WEST ABUTMENTS HAVE PIER TOP AT ELEV. 1342, APPROXIMATELY. PIERS 1 & 2 WILL HAVE PIER TOP AT ELEV 1346±. FINISHED GRADE AT THE ABUTMENTS WILL BE ABOVE THE PIER TOP, BUT AT PIERS 1 & 2 WILL BE AT ELEV 1334. CONSIDER EXCAVATION AND OTHER EARTHWORK WILL NOT LOWER Q_E & Q_S VALUES SHOWN IN FIGURE C-1, p. 2, THUS, USE Q_E & Q_S VALUES FOR ELEVATION SHOWN.

DESIGN CAPACITIES USING THE ABOVE EXPRESSION ARE LISTED IN TABLE C-1, p. 3.

B. CONSIDER SIDE SHEAR ONLY

$P_D = \sum_{i=1}^{n} Q_{Si} A_{Si} L_i / FS$ = DESIGN CAPACITY IN KIPS

Q_{Si} = ULTIMATE SIDE SHEAR FROM FIG. C-1, p. 2, KIPS

A_{Si} = UNIT SIDE AREA = πd

d = PIER DIAMETER IN FEET

L_i = SEGMENT LENGTH IN FEET

$P_D = 1.05\, d \sum_{i=1}^{n} Q_{Si} L_i$

FIGURE C-1

DESIGN ULTIMATE END-BEARING AND SIDE SHEAR PROFILE FOR COMPUTING VERTICAL CAPACITY

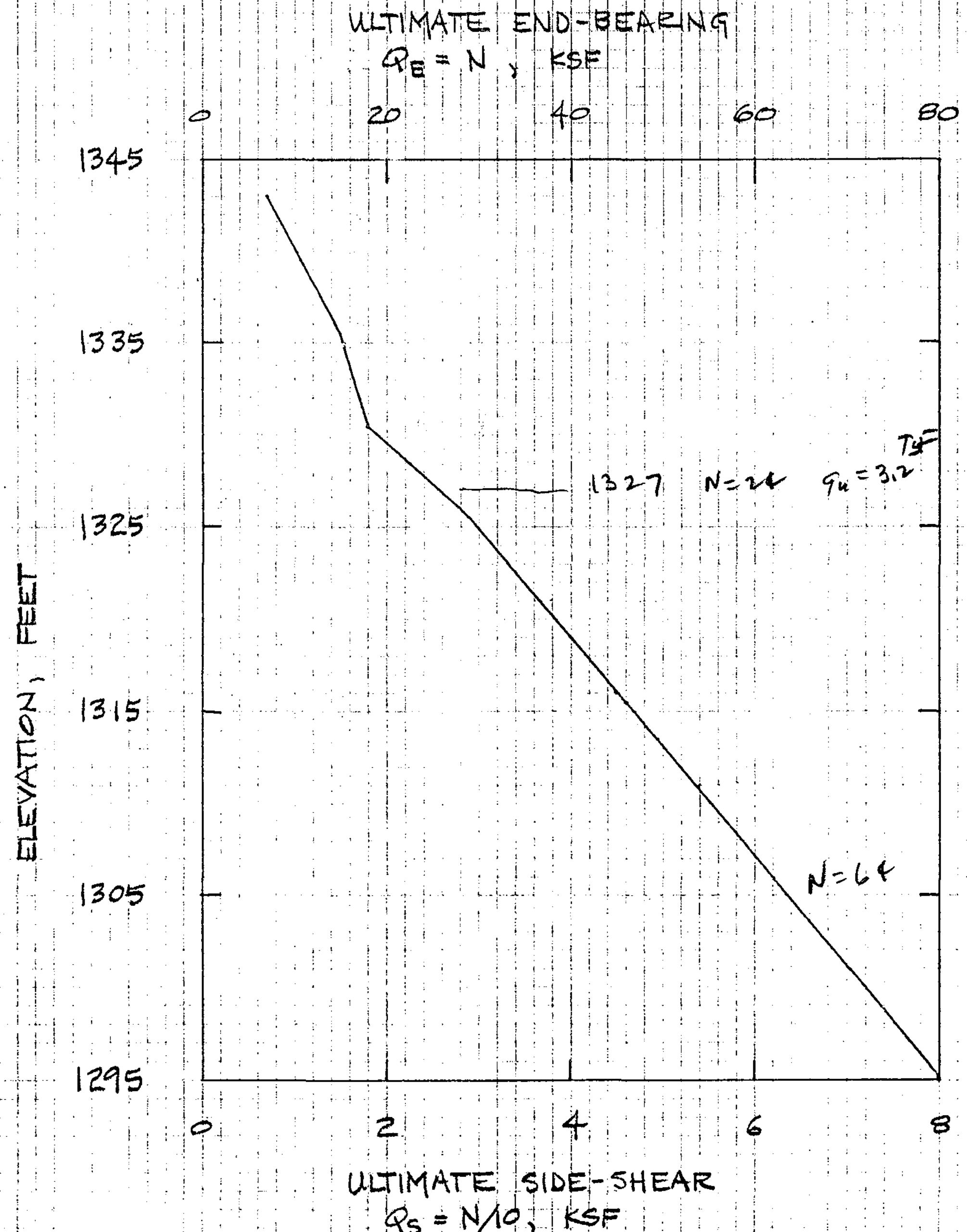

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TABLE C-1

DESIGN VERTICAL DOWNWARD CAPACITY OF DRILLED PIERS BASED ON END-BEARING

PIER TIP ELEVATION FEET	q_E KSF	CAPACITY, P_D, KIPS d=2.0 FEET	d=2.5 FEET	d=3.0 FEET	d=3.5 FEET
1340	10	10	16	24	32
1335.5	15	16	25	35	48
1330.5	18	19	29	42	58
1325.5	29	30	47	68	93
1315.5	46	48	75	108	148
1305	64	67	105	151	205
1295	80	84	131	189	257

TABLE C-2

INCREMENTAL VERTICAL DOWNWARD CAPACITY OF DRILLED PIERS BASED ON SIDE SHEAR

ELEVATION INCREMENT FEET	Q_{si} KSF	L_i FT	INCREMENTAL CAPACITY, P_{Di}, KIPS d=2.0 FEET	d=2.5 FEET	d=3.0 FEET	d=3.5 FEET
1342-37	1.05	5	11.0	13.8	16.5	19.3
1337-31	1.60	6	20.2	25.2	30.2	35.3
1331-26	2.25	5	23.6	29.5	35.4	41.3
1326-21	3.25	5	34.1	42.7	51.2	59.7
1321-16	4.10	5	43.0	53.8	64.6	75.3
1316-11	4.95	5	52.0	65.0	78.0	91.0
1311-06	5.80	5	60.9	76.1	91.4	106.6
1306-01	6.60	5	69.3	86.6	104.0	121.3
1301-95	7.50	6	94.5	118.1	141.8	165.4

CONSIDER ABUTMENTS AND PIERS 1 & 2 SEPARATELY, SINCE FINISHED GRADE IS DIFFERENT AT THEIR RESPECTIVE LOCATIONS. CONSIDER PIER DIAMETERS OF 2.0, 2.5, 3.0 & 3.5 FEET. DISALLOW SIDE SHEAR IN THE FIRST 3 FEET BELOW FINISHED GRADE FOR PIERS 1 & 2. FOR THE ABUTMENTS, FINISHED GRADE IS ABOVE THE TOP OF THE PIER, AND IT IS LIKELY THE ABUTMENT WILL BE CAST ON GRADE, OR BACKFILLED WITH ON-SITE SOIL. FOR THE ABUTMENTS, THEN, USE SIDE SHEAR FROM THE TOP OF THE PIER, INCLUDING THE FIRST 3 FEET.

DESIGN PIER CAPACITIES BASED ON SIDE SHEAR ARE LISTED AS INCREMENTAL VALUES IN TABLE C-2, p. 3. TOTAL CAPACITIES FOR ABUTMENTS ARE LISTED IN TABLE C-3, p. 5. THESE WERE CALCULATED BY SUMMING THE INCREMENTAL VALUES BELOW THE PIER TOP ELEVATION OF 1342. TOTAL CAPACITIES FOR PIER LOCATIONS 1 & 2 ARE LISTED IN TABLE C-4, p. 5. THESE WERE CALCULATED BY SUMMING THE INCREMENTAL VALUES BELOW ELEVATION 1331 (3 FEET BELOW FINISHED GRADE OF 1334).

C. CONCLUSION

COMPARISON OF TABLES C-1, C-2 & C-3 INDICATES GREATER CAPACITY IN SIDE SHEAR. THIS WILL DEVELOP WITH APPROXIMATELY 0.1" SHEAR DISPLACEMENT, IF CONSIDERING ULTIMATE, AND LESS FOR WORKING LOAD LEVELS DEFINED BY A SAFETY FACTOR OF 3.0. FOR DETERMINING DESIGN CAPACITIES USE TABLES C-3 AND C-4.

TABLE C-3

DESIGN VERTICAL DOWNWARD CAPACITY
OF DRILLED PIERS FOR ABUTMENTS
BASED ON SIDE SHEAR

PIER TIP ELEVATION FEET	EMBEDED LENGTH FEET	CAPACITY, P_D, KIPS d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET
1331	11	31	39	47	55
1326	16	55	68	82	96
1321	21	89	111	133	156
1316	26	132	165	198	231
1311	31	184	230	276	322
1306	36	245	306	367	428

TABLE C-4

DESIGN VERTICAL DOWNWARD CAPACITY
OF DRILLED PIERS FOR PIER LOCATIONS
1 & 2 BASED ON SIDE SHEAR

PIER TIP ELEVATION FEET	EMBEDED LENGTH FEET	CAPACITY, P_D, KIPS d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET
1321	13	58	72	87	101
1316	18	101	126	151	176
1311	23	153	191	229	267
1306	28	214	267	321	374
1301	33	283	354	425	495

II. SAFE DOWNWARD CAPACITY OF BELLED, DRILLED, CAST-IN-PLACE CONCRETE PIERS

FROM ANALYSIS FOR STRAIGHT PIERS, AND CONSIDERING THE FUNCTION OF A BELL, CONSIDER END-BEARING ONLY, WITH

$$P_D = 0.262\, q_E\, d^2$$

WHERE d IS NOW THE DIAMETER OF THE BELL IN FEET. THUS, NEED ONLY EXTEND TABLE C-1, P. 3, BY INCLUDING LARGER VALUES OF d:

TABLE C-5

DESIGN VERTICAL DOWNWARD CAPACITY OF BELLED PIERS BASED ON END-BEARING

PIER TIP ELEVATION FEET	q_E KSF	CAPACITY, P_D, KIPS $d = 4.0$ FEET	$d = 4.5$ FEET	$d = 5.0$ FEET	$d = 5.5$ FEET
1340	10	42	53	65	79
1335.5	15	63	80	98	119
1330.5	18	75	95	118	143
1325.5	29	122	154	190	230
1315.5	46	193	244	301	365
1305	64	268	340	419	507
1295	80	335	424	524	634

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

A. CONSIDER END-BEARING ONLY

BASIC RELATIONSHIP IS PRESENTED IN

CHRISTIAN, J.T. AND CARRIER, W.D. "JANBU, BJERRUM AND KJAERNLSI'S CHART REINTERPRETED", CANADIAN GEOTECHNICAL JOURNAL, VOL. 15, 1978, pp. 123-128.

C-7

THE RELATIONSHIP IS

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

WHERE

δ = SETTLEMENT

q = AVERAGE BEARING PRESSURE

d = PIER DIAMETER

E_s = SOIL MODULUS

μ_0, μ_1 ARE DIMENSIONLESS PARAMETERS DEPENDENT ON DEPTH

SUBSTITUTING $q = Q/(\pi d^2/4)$, WE HAVE

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

WHERE Q IS THE APPLIED AXIAL LOAD. FOR A CIRCULAR FOUNDATION, $\mu_1 = 0.62$. μ_0 IS DEPENDENT ON THE RATIO D/d, WHERE D = DEPTH TO FOOTING BELOW FINISHED GRADE. THIS VARIATION IS SHOWN IN FIGURE C-2, p. 8. MODULUS VALUES ARE TAKEN FROM A RELATIONSHIP BETWEEN SPT BLOWCOUNT (N-VALUE) AND E_s BASED ON CEMENTED PHOENIX SOILS. THIS IS PRESENTED IN

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Job No.: E 83-103
Computed by: SSC Ckd. by: LAH
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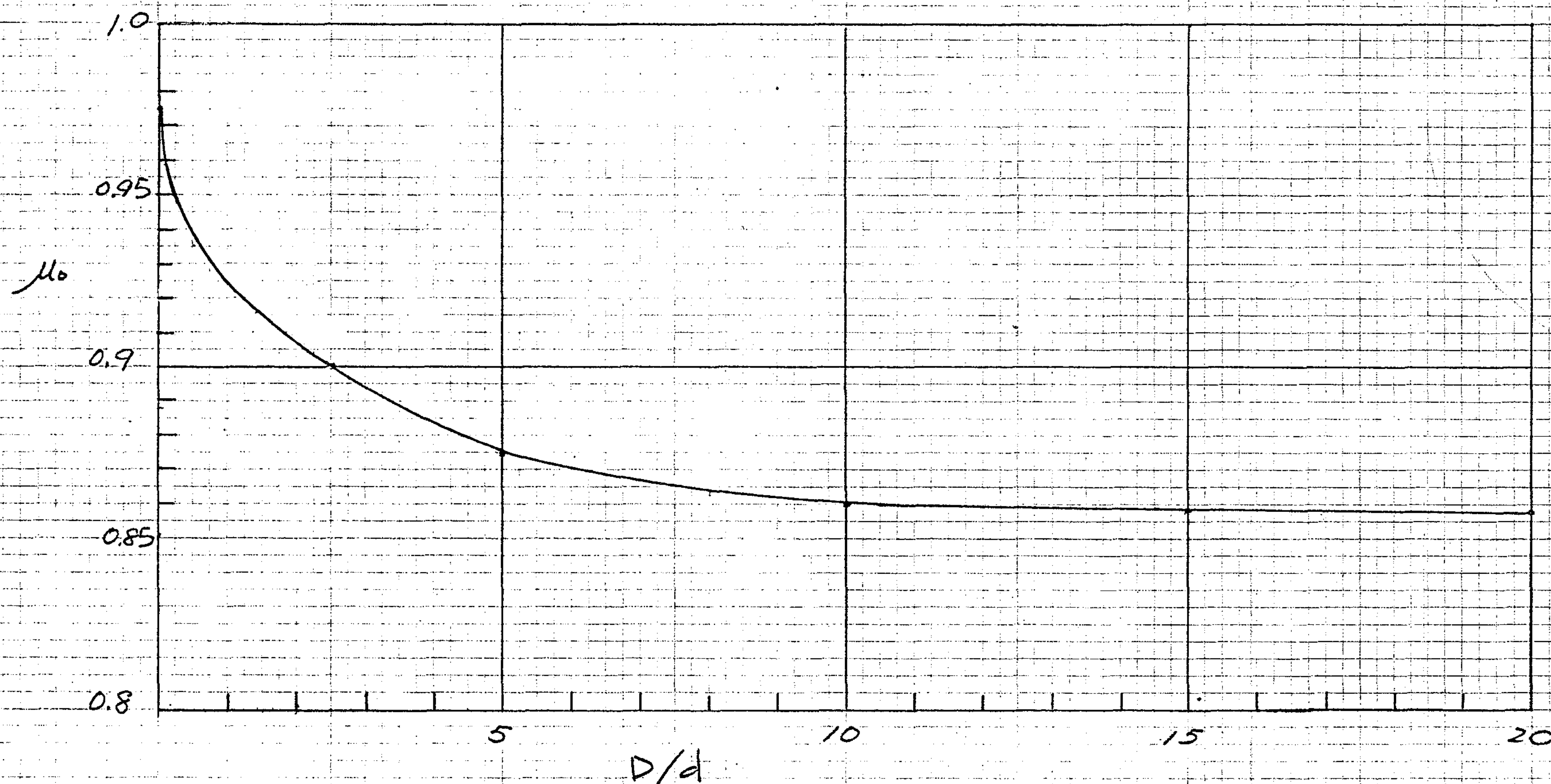
FIGURE C-2

VARIATION OF μ_o WITH D/d



(D = DEPTH TO FOOTING BELOW FINISHED GRADE)
(d = PIER DIAMETER)

BECKWITH, G.H. AND HANSEN, L.A. "CALCAREOUS SOILS OF THE SOUTHWESTERN UNITED STATES", ASTM STP 777, 1982, pp 16-35.

THE WORKING RELATIONSHIP FOR CALCULATING SETTLEMENT IS, THEN

$$\delta = 0.62\, \mu_1 (4/\pi)(Q/d\,E_s)$$
$$= 0.789\, \mu_1 Q/d\,E_s$$

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

$$\delta = 9.47\, \mu_1 Q/d\,E_s$$

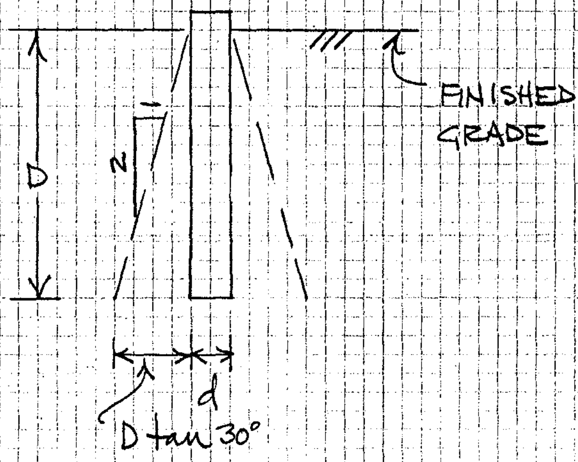
ESTIMATED SETTLEMENTS ARE PRESENTED IN TABLE C-6 FOR ABUTMENT PIERS AND IN TABLE C-7 FOR INTERIOR PIERS (PIER LOCATIONS 1 + 2). AS EXAMPLE OF THE USE OF THESE TABLES, ASSUME A DESIGN VERTICAL LOAD OF 160 KIPS APPLIED TO AN INTERIOR PIER WITH TIP ELEVATION 1310.5 AND DIAMETER OF 2.0 FEET. THE ESTIMATED SETTLEMENT IS

$$\delta = 0.00205(160) = 0.32 \text{ INCHES}$$

FIGURES C-6 AND C-7 CAN BE USED FOR EITHER STRAIGHT SHAFTED OR BELLED PIERS SINCE ONLY END-BEARING IS CONSIDERED.

B. CONSIDER SIDE-SHEAR (FRICTION) ONLY

FOR THIS CASE THE RELATIONSHIP DESCRIBED ABOVE WAS MODIFIED TO ALLOW FOR A LARGER EFFECTIVE DIAMETER, AND THUS A LOWER BEARING PRESSURE. AS SHOWN BELOW, AN EQUIVILANT FOOTING DIAMETER IS USED, LOCATED AT THE PIER TIP ELEVATION



$$d^* = 2D\tan 30° + d$$

$$q^* = Q/(\pi d^{*2}/4)$$

CALCULATION OF SETTLEMENT PROCEEDS AS BEFORE. CHARTS WERE PREPARED FOR ABUTMENT PIERS AND INTERIOR PIERS, SHOWN IN TABLES C-8 AND C-9, RESPECTIVELY. THESE CAN BE USED IN THE SAME MANNER AS TABLES C-6 AND C-7.

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TABLE C-6

NORMALIZED SETTLEMENT IN INCHS/KIP VERTICAL LOAD
FOR ABUTMENT PIERS (FINISHED GRADE = 1342)
ASSUMING END-BEARING ONLY

PIER TIP ELVATION FEET	E_s KSF	SETTLEMENT, IN/KIP d=2.0	d=2.5	d=3.0	d=3.5	d=4.0	d=4.5	d=5.0
1337	275	0.0155	0.0125	0.0106	0.0093	0.0080	0.0071	0.0064
1331	375	0.0110	0.0088	0.0075	0.0064	0.0057	0.0051	0.0046
1326	790	0.0052	0.0041	0.0035	0.0030	0.0027	0.0024	0.0021
1321	995	0.0041	0.0033	0.0028	0.0024	0.0021	0.0019	0.0018
1316	1500	0.0027	0.0022	0.0018	0.0016	0.0013	0.0012	0.0011
1311	1980	0.0021	0.0016	0.0014	0.0012	0.0010	0.0009	0.0008
1306	2500	0.0016	0.0013	0.0011	0.0009	0.0008	0.0007	0.0007
1301	2700	0.0015	0.0012	0.0010	0.0008	0.0007	0.0006	0.0006
1295	3150	0.0013	0.0010	0.0009	0.0007	0.0006	0.0006	0.0005

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TABLE C-7

NORMALIZED SETTLEMENT IN INCHS/KIP VERTICAL LOAD FOR INTERIOR PIERS (FINISHED GRADE = 1334) ASSUMING END BEARING ONLY.

PIER TIP ELEVATION FEET	E_S KSF	SETTLEMENT, IN/KIP d=2.0	d=2.5	d=3.0	d=3.5	d=4.0	d=4.5	d=5.0
1331	375	0.0115	0.0093	0.0078	0.0067	0.0059	0.0053	0.0047
1326	790	0.0053	0.0043	0.0036	0.0031	0.0028	0.0025	0.0021
1321	995	0.0041	0.0033	0.0028	0.0024	0.0021	0.0019	0.0018
1316	1500	0.0027	0.0022	0.0018	0.0016	0.0013	0.0012	0.0011
1311	1980	0.0021	0.0016	0.0014	0.0012	0.0010	0.0009	0.0008
1306	2500	0.0016	0.0013	0.0011	0.0009	0.0008	0.0007	0.0007
1301	2700	0.0015	0.0012	0.0010	0.0008	0.0007	0.0006	0.0006
1295	3150	0.0013	0.0010	0.0009	0.0007	0.0006	0.0006	0.0005

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TABLE C-8

NORMALIZED SETTLEMENT IN
INCHES PER KIP VERTICAL LOAD
FOR ABUTMENT PIERS
(FINISHED GRADE = 1342)
BASED ON SIDE SHEAR ONLY

PIER TIP ELEVATION FEET	SETTLEMENT, IN/KIP			
	d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET
1337	0.0044	0.0042	0.0040	0.0038
1331	0.0017	0.0016	0.0016	0.0015
1326	0.0006	0.0006	0.0006	0.0006
1321	0.0004	0.0004	0.0004	0.0004
1316	0.0002	0.0002	0.0002	0.0002
1311	0.0001	0.0001	0.0001	0.0001
1306	0.0001	0.0001	0.0001	0.0001
1301	0.0001	0.0001	~0	~0
1295	~0	~0	~0	~0

TABLE C-9

NORMALIZED SETTLEMENT IN INCHES PER KIP VERTICAL LOAD FOR INTERIOR PIERS (FINISHED GRADE = 1334) BASED ON SIDE SHEAR ONLY

PIER TIP ELEVATION FEET	SETTLEMENT, IN/KIP			
	d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET
1331	0.0046	0.0042	0.0039	0.0036
1326	0.0011	0.0010	0.0010	0.0009
1321	0.0005	0.0005	0.0005	0.0005
1316	0.0003	0.0003	0.0003	0.0003
1311	0.0002	0.0002	0.0002	0.0002
1306	0.0001	0.0001	0.0001	0.0001
1301	0.0001	0.0001	0.0001	0.0001
1295	0.0001	0.0001	~0	~0