SERGENT, HAUSKINS & BECKWITH

GEOTECHNICAL INVESTIGATION REPORT

University Drive Bridge Over Roosevelt
Water Conservation District (RWCD) Floodway
Mesa, Arizona

SHB Job No. E83-99

Consulting Geotechnical Engineers
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SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING

B. DWAINE SERGENT, P.E. JOHN B. HAUSKINS, P.E. GEORGE H. BECKWITH, P.E. ROBERT D. BOOTH, P.E.
NORMAN H. WETZ, P.E. DALE V. BEDENKOP, P.E. ROBERT R. KOONS, P.E. ROBERT W. CROSSLEY, P.E.
WAYNE A. ERICSON, P.E. ROBERT L. FREW DONALD G. METZGER, P.G. RALPH E. WEEKS, P.G.
DONALD L. CURRAN, P.E. ALLON C. OWEN, JR., P.E.

August 24, 1983

Dashney & Associates, Inc.
1107 North Beeline Highway
Suite D
Payson, Arizona 85541

SHB Job No. E83-99

Attention: Gary A. Dashney, P.E.

Re: University Drive Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona

Gentlemen,

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

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

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By Robert W. Crossley
Robert W. Crossley, P.E.

Registered Professional Engineer (Civil) Certificate No. 11049 ROBERT W. CROSSLEY

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

Registered Professional Engineer (Civil) Certificate No. 5633 GEORGE HENRY BECKWITH Date Signed 8-24-83 ARIZONA, U.S.A.

Copies: Addressee (3)
E. M. Plummer Consulting Engineers
Attn: E. M. Plummer, P.E. (2)

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

PHOENIX (602) 272-6848 ALBUQUERQUE (505) 884-0950 SANTA FE (505) 471-7836 SALT LAKE CITY (801) 566-5411

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SHB Job No. E83-99

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1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed University Drive Bridge Over 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 slab support.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided by Mr. Shafi Hasan of E.M. Plummer Consulting Engineers.

It is understood that a four-span, four-lane roadway bridge will cross the proposed Roosevelt Water Conservation District Floodway. The bridge will be approximately 82½ feet wide and 187 feet in length. The bridge will be built utilizing continuous frame concrete construction. The floodway channel is to be unlined with channel invert at elevation 1336.89 at the centerline of the bridge. No abutments are involved.

Each of the outer pier bents will carry about 882 kips vertical load (dead plus live load), while each of the three interior bents will carry 1,839 kips. It is anticipated that the spans will be carried on rows of circular concrete piers which are to be drilled and cast-in-place below grade and formed above grade, joined by a 3½-foot high cap beam below the bridge deck. Drilled piers will be terminated about 4 to 5 feet below the channel invert and will be formed up to the cap beam.

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

Five exploratory borings were drilled to depths of about 51 feet below existing grade. Boring 1 experienced auger refusal at 32 feet due to the very strongly cemented condition of the soils at that depth. 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. 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 a logs of the test borings. The field investigation was supervised by Norman H. Wetz, P.E., and Robert W. Crossley, P.E., staff engineers of this firm.

3.2 Laboratory Analysis

Moisture content and dry density determinations were made on selected tube samples recovered. The results of these tests are shown on the boring logs.

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

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

The existing Roosevelt Conservation District Canal is located immediately to the west of this bridge site. University Drive is a major, paved, two-lane roadway. Curb, gutter and sidewalk are in place south of the side of the roadway, while unimproved shoulders are present on the north side. Vacant fields presently exist beyond the roadway in the area designated for the new floodway, both on the north and south sides of University Drive.

4.2 Geotechnical Profile

The subsoils consist of stratified, predominantly fine grained desert alluvium. Sandy clay of medium to high plasticity is the dominant soil type, with lesser thicknesses of silty clay and clayey sand of low to medium plasticity also present throughout the depths examined.

All of the soils are cemented to some degree with calcium carbonate and possibly other minerals. The degree of cementation may be described as generally weak in the upper 15 feet or so, but moderate to strong cementation occurs below this depth. In most of the borings, especially strong cementation was encountered from about 30 to 45 feet below grade (elevation 1315 to 1300).

4.3 Soil Moisture & Groundwater Conditions

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

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

The near surface soils are somewhat moisture sensitive in that they would be weakened with substantial moisture increases. Additionally, most of the clayey soils in the profile are moderately expansive. This condition is not considered a problem for the proposed structure; however, inasmuch as deep drilled pier foundations are applicable.

It is recommended that the main structure elements be supported on straight, machine-cleaned, cast-in-place concrete piers. Tip elevation of piers should be at least 10.0 feet below the cap beam connection for outer bents and at least 10.0 feet below channel invert for the interior bents. Design criteria for straight drilled piers are given in Section 5.2.

As an alternative, manually cleaned, drilled-and-belled piers which derive their support mainly from end-bearing may be utilized. Design criteria for drilled-and-belled piers are given in Section 5.3.

5.2 Straight, Machine-Cleaned, 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 for outer and interior bents are given in Design Charts 1 and 2 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

It is estimated that settlements of pier foundations designed and constructed in accordance with criteria presented herein will not exceed ½ inch. Settlements will occur rapidly and will be essentially complete after the application of the first few cycles of live loading.

5.2.3 Resistance to Lateral Loads

Design Charts 3 and 4, shown in Appendix C, present the estimated ultimate lateral soil bearing pressures versus depth. It is recommended that a factor of safety of 2.0 be used in determining safe lateral capacities. These values may be used to compute the ultimate lateral capacities of drilled piers by the Hansen method or other "stability" methods.

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.

5.2.5 Placement of Concrete

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

5.2.6 Quality Assurance & Construction

Continuous observations of the construction of drilled piers should be carried out by a representative of the geotechnical engineer.

He should confirm proper diameter, depth and cleaning, and should also confirm the nature of the materials encountered in the pier excavations. Concrete placement should be continuously observed to confirm that it meets requirements. A quality assurance report should be submitted on each pier stating, in writing, that all details have been observed and meet requirements.

It appears that drilled pier excavations can be advanced to the depths recommended with little or no caving. Caving is expected to be very minimal, so concrete quantities will be very near the neat volume indicated by the plans.

5.3 Manually Cleaned, Drilled-and-Belled, Cast-in-Place Concrete Piers

5.3.1 Downward Loads

A safe soil bearing pressure of 16.0 ksf is recommended for belled piers at elevation 1325.0 or deeper. This value is based on manual cleaning of all loose, disturbed materials from the entire end-bearing areas. The bearing pressure applies to full dead plus live loads and may be safely increased by one-third for total loads including wind or seismic forces.

5.3.2 Estimated Settlements

It is estimated that the foundation settlements of foundations designed and constructed in accordance with criteria presented in this section will be less than ½ inch. Settlements are expected to occur rapidly and be essentially complete at the end of construction.

5.3.3 Resistance to Lateral Loads

Criteria in Section 5.2.3 should be used in determining the resistance to lateral loads.

5.3.4 Geotechnical Conditions for Construction

The discussion in Sections 5.2.4 through 5.2.6 is applicable for drilled-and-belled piers.

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.

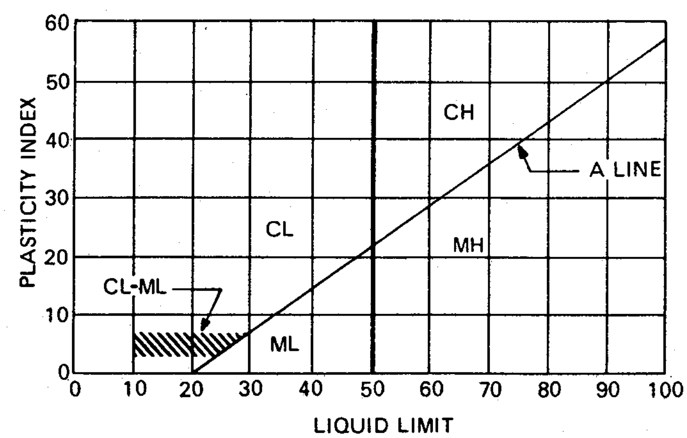
UNIFIED SOIL CLASSIFICATION SYSTEM

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.

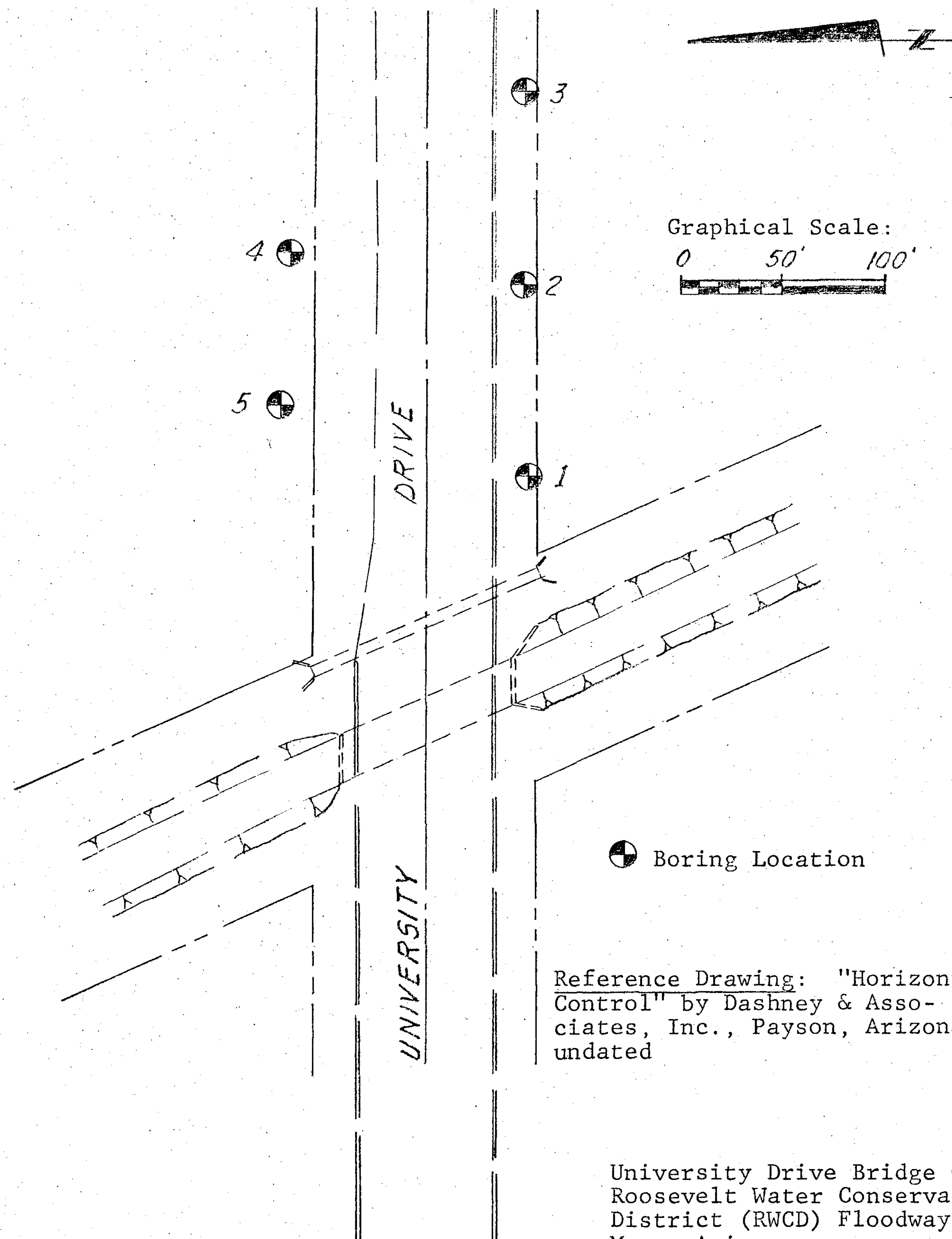
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
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: "Horizontal Control" by Dashney & Associates, Inc., Payson, Arizona, undated

University Drive Bridge Over
Roosevelt Water Conservation
District (RWCD) Floodway
Mesa, Arizona
SHB Job No. E83-99

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CONSULTING SOIL AND FOUNDATION ENGINEERS
PHOENIX • TUCSON • ALBUQUERQUE

PROJECT University Drive Bridge Over RWCD Floodway

JOB NO. E83-99 DATE 8-5-83

LOG OF TEST BORING NO. 1

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1346.0'+0.3'

DATUM Dashney & Associates, Inc.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	46			SC	very firm	Probable Man-Made FILL CLAYEY SAND, some gravel, well graded, low plasticity, brown
5			X	S	23				firm to very hard	SANDY CLAY, weakly to moderately lime cemented, medium plasticity, reddish-brown to light brown
10			X	S	52					note: occasional thin lenses of clayey sand, well graded, angular, low plasticity, brown
15			X	S	47			CL & SC		
20			X	S	50/5¾"					
25			X	S	50/5"					
30			X	S	36					
35										Auger refused at 32' on strongly cemented strata

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT University Drive Bridge Over RWCD Floodway

JOB NO. E83-99 DATE 8-8-83

LOG OF TEST BORING NO. 2

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1346.0'+0.3'

DATUM Dashney & Associates, Inc.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	51			CL	hard	SILTY CLAY, weakly lime cemented, medium plasticity, light brown
5			X	S	37				moderately firm to hard	SANDY CLAY, stratified, weakly to moderately lime cemented, medium plasticity, light brown to brownish-white, interbedded with CLAYEY SAND, predominantly fine, angular, low to medium plasticity, brownish-white
10			X	S	24					
15			X	S	85					
20			X	S	67					
25			X	S	30			CL & SC		
30			X	S	25					
35			X	S	93					
40			X	S	90					
45			X	S	50/3"					
50			X	S	50/3"					Stopped auger at 49'6" Sampler refused at 50'3"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

U – 3" O.D. 2.42" I.D. tube sample.

T – 3" O.D. thin-walled Shelby tube.

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PROJECT: University Drive Bridge Over RWCD Floodway

LOG OF TEST BORING NO. 3

JOB NO. E83-99 DATE 8-8-83

RIG TYPE: CME-75
BORING TYPE: 6½" Hollow Stem Auger
SURFACE ELEV.: 1346.2'+0.3'
DATUM: Dashney & Associates, Inc.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0			X	S	43			CL
5			X	S	35			SC-SM
10			X	U	37	103	12	
15			X	U	74	99	14	
20			X	U	100/6"	98	14	
25			X	S	116		13	
30			X	S	23		10	CL & SC
35			X	S	50/5"		17	
40			X	S	50/4"		12	
45			X	S	50/6"			
50			X	S	50/5"			

REMARKS	VISUAL CLASSIFICATION
very firm	SILTY CLAY, weakly lime cemented, medium plasticity, light brown
firm	CLAYEY SAND, considerable silt, medium to fine, angular, weakly lime cemented, low plasticity, brown
firm to hard	SANDY CLAY, stratified, weakly to moderately lime cemented, medium plasticity, light brown to brownish-white, interbedded with CLAYEY SAND, predominantly fine, angular, low to medium plasticity, brownish-white note: strongly cemented below 30'
	Stopped auger at 49'6" Sampler refused at 50'5"

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.

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PROJECT University Drive Bridge Over RWCD Floodway

JOB NO. E83-99 DATE 8-9-83

LOG OF TEST BORING NO. 4

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1344.8'+0.2'

DATUM Dashney & Associates, Inc.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			X	S	45			CL	very firm	SANDY CLAY, weakly lime cemented, medium plasticity, reddish-brown
5			X	S	48				firm to hard	SANDY CLAY, stratified, medium to high plasticity, brown to brownish-white & CLAYEY SILT (ML-CL), weakly to moderately lime cemented, low plasticity, light brown, interbedded with CLAYEY SAND, predominantly medium to fine, angular, medium plasticity, light brown note: strongly cemented below 30'
10			X	S	21					
15			X	S	105		15			
20			X	S	104		15	CH, CL-ML & SC		
25			X	S	54		12			
30			X	S	46		11			
35			X	S	50/3"		12			
40			X	S	50/6"				hard	SANDY CLAY, moderately lime cemented, medium to high plasticity, light brown
45			X	S	50/6"			CH		
50			X	S	50/5"					Stopped auger at 49'6" Sampler refused at 50'5"

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.

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LOG OF TEST BORING NO. 5

PROJECT University Drive Bridge Over RWCD Floodway

JOB NO. E83-99 DATE 8-9-83

RIG TYPE CME-75

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1344.8'+0.2'

DATUM Dashney & Associates, Inc.

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
0–7			X	S	41			SC	very firm	CLAYEY SAND, predominantly medium to fine, angular, stratified, weakly lime cemented, low to medium plasticity, brown to brownish-white
5			X	S	37					
7–29								CH	moderately firm to hard	SANDY CLAY, stratified, weakly to moderately lime cemented, medium to high plasticity, brownish-white
10			X	S	18					
15			X	S	53					
20			X	S	64					
25			X	S	99					
29–50								CL-CH	very firm to hard	SANDY CLAY, moderately to strongly lime cemented, medium to high plasticity, brown
30			X	S	38					
35			X	S	50/3"					
40			X	S	50/3"					
45			X	S	125					
50			X	S	117					Stopped auger at 49'6" Sampler refused at 50'10"

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GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

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

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LABORATORY TESTING PROCEDURES

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

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

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

TABULATION OF TEST RESULTS

Job No. E83-99

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#200	#100	#50	#40	#30	#16	#10	#8	#4	.25"	.375"	.5"	LAB NO
					.75"	1"	1.5"	2"	2.5"	3"	3.5"	4"	6"	8"	10"	12"	
3	29.5′-31′	CL	41	25	63.1	70	75	77	80	87	92	94	97	98	100		3-99-18
4	24.5′-26′	CH	61	31	63.6	70	72	75	77	84	90	92	98	99	100		3-99-28
5	14.5′-16′	CH	72	40	67.6	74	78	80	81	86	92	93	99	100			3-99-37
5	44.5′-46′	CH	52	28	67.9	77	82	83	86	90	93	94	97	100			3-99-43

The table sits under the group heading SIEVE ANALYSIS-ACCUM % PASSING, which spans the sieve columns.

REPORT ON LABORATORY TESTS

DATE ______

PROJECT University Drive Bridge Over Roosevelt Water Conservation District (RWCD) Floodway JOB NO. E83-99

LOCATION Mesa, Arizona LAB NO. 3-99-15

SAMPLE Boring #3 @ 15'-16'

IN SITU DIRECT SHEAR TESTS

In Situ - Point No. 1 (= ± 0.50 KSF)

Initial Moisture Content	13.7 %
Dry Density (PCF)	91.8
Submerged	
Final Moisture Content	- %
Maximum Vertical Deformation @ T Max.	(+) 0.023 Inches
Shearing Stress, T Max.	1.54 KSF

In Situ - Point No. 2 (= ± 1.00 KSF)

Initial Moisture Content	14.4 %
Dry Density (PCF)	98.7
Submerged	
Final Moisture Content	- %
Maximum Vertical Deformation @ T Max.	(+) 0.018 Inches
Shearing Stress, T Max.	2.80 KSF

In Situ - Point No. 3 (= ± 1.47 KSF)

Initial Moisture Content	14.7 %
Dry Density (PCF)	102.8
Submerged	
Final Moisture Content	- %
Maximum Vertical Deformation @ T Max.	(+) 0.021 Inches
Shearing Stress, T Max.	4.83 KSF

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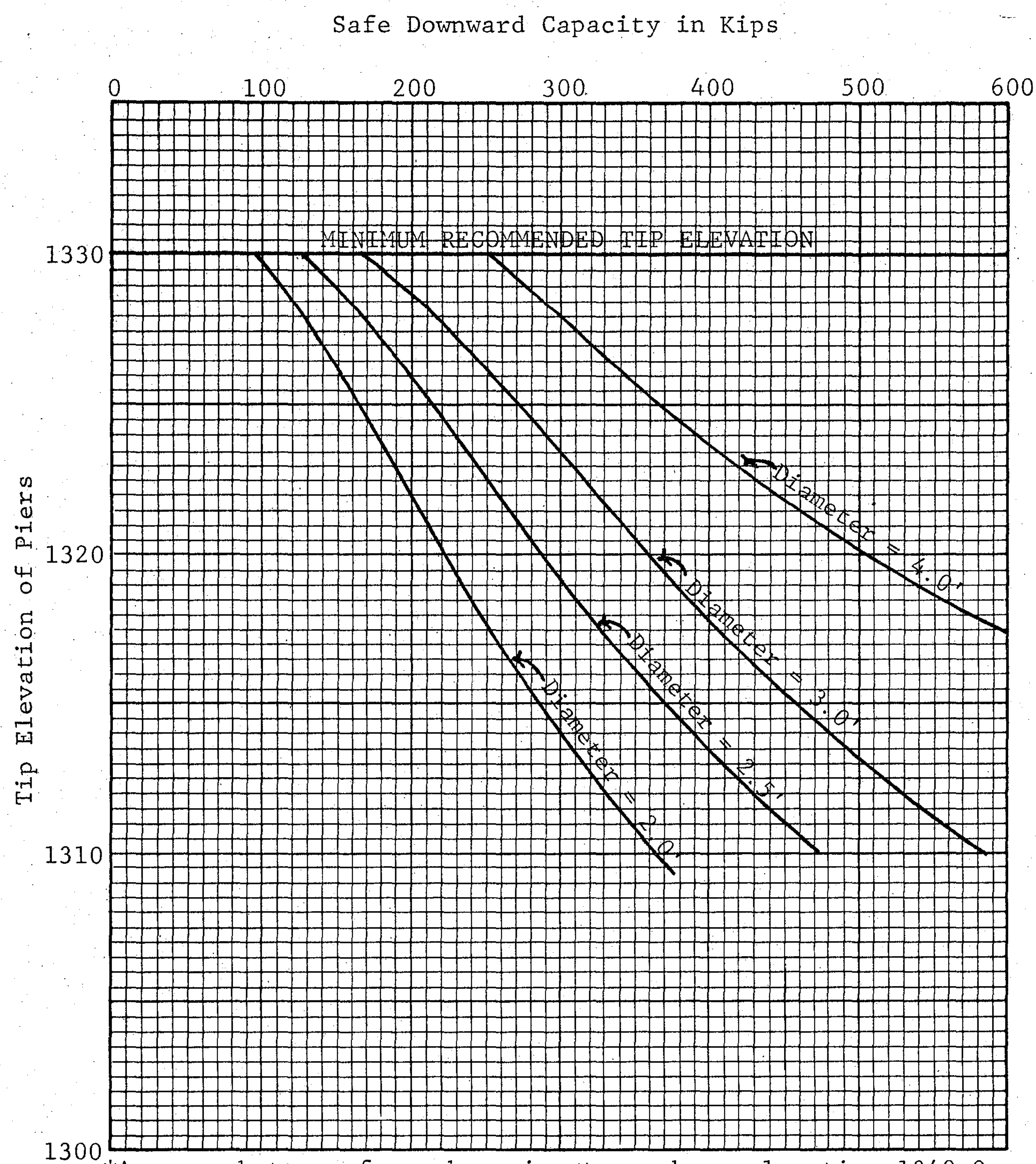
PROJECT University Drive Bridge Over RWCD Floodway JOB NO. E83-99

DESIGN CHART 1 - APPLIES TO OUTER PIER BENTS

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



*Assumes bottom of cap beam is at or above elevation 1340.0 feet and piers are completely imbedded in native soils.

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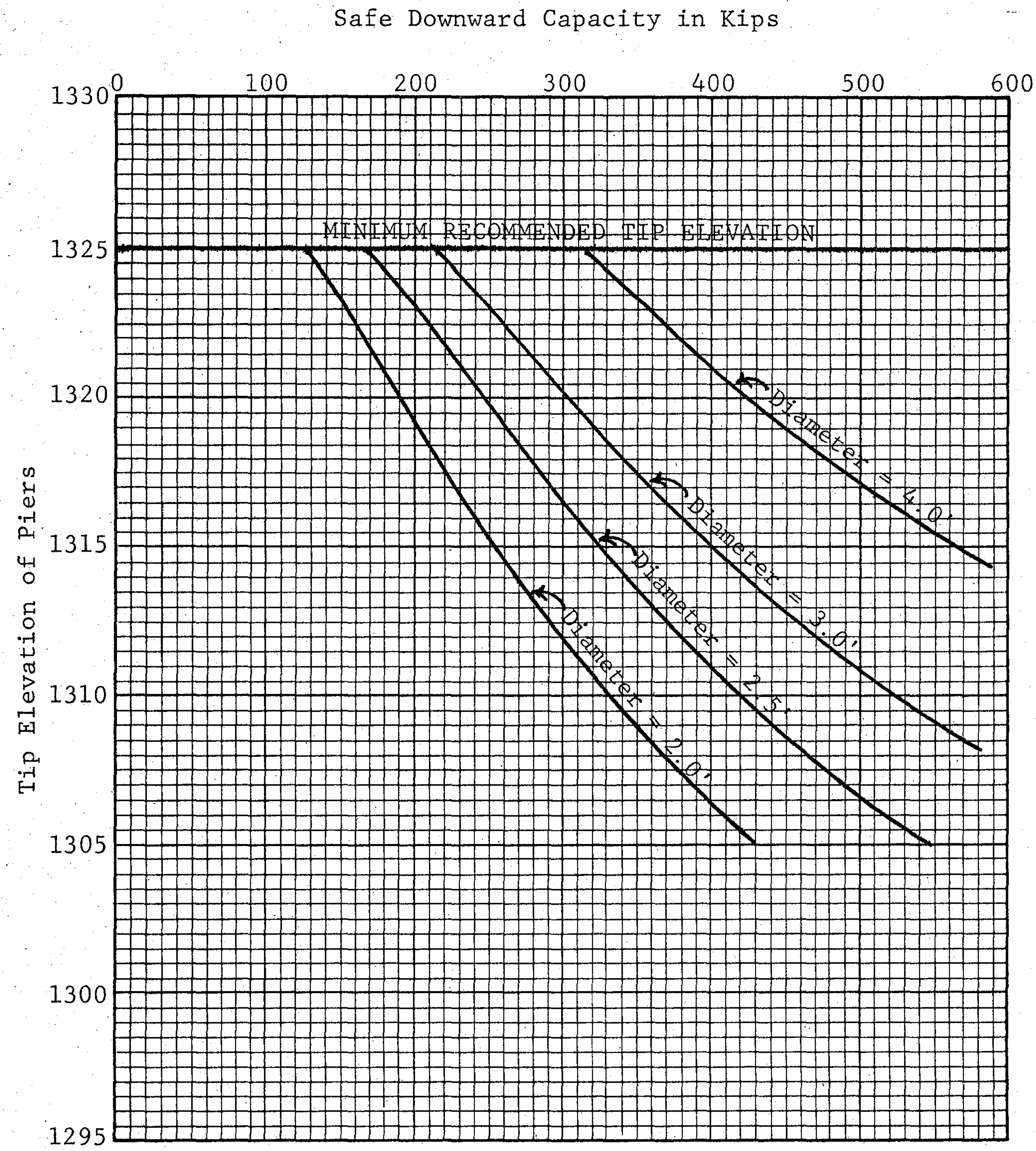
PROJECT University Drive Bridge Over RWCD Floodway JOB NO. E83-99

DESIGN CHART 2 - APPLIES TO INTERIOR PIER BENTS

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



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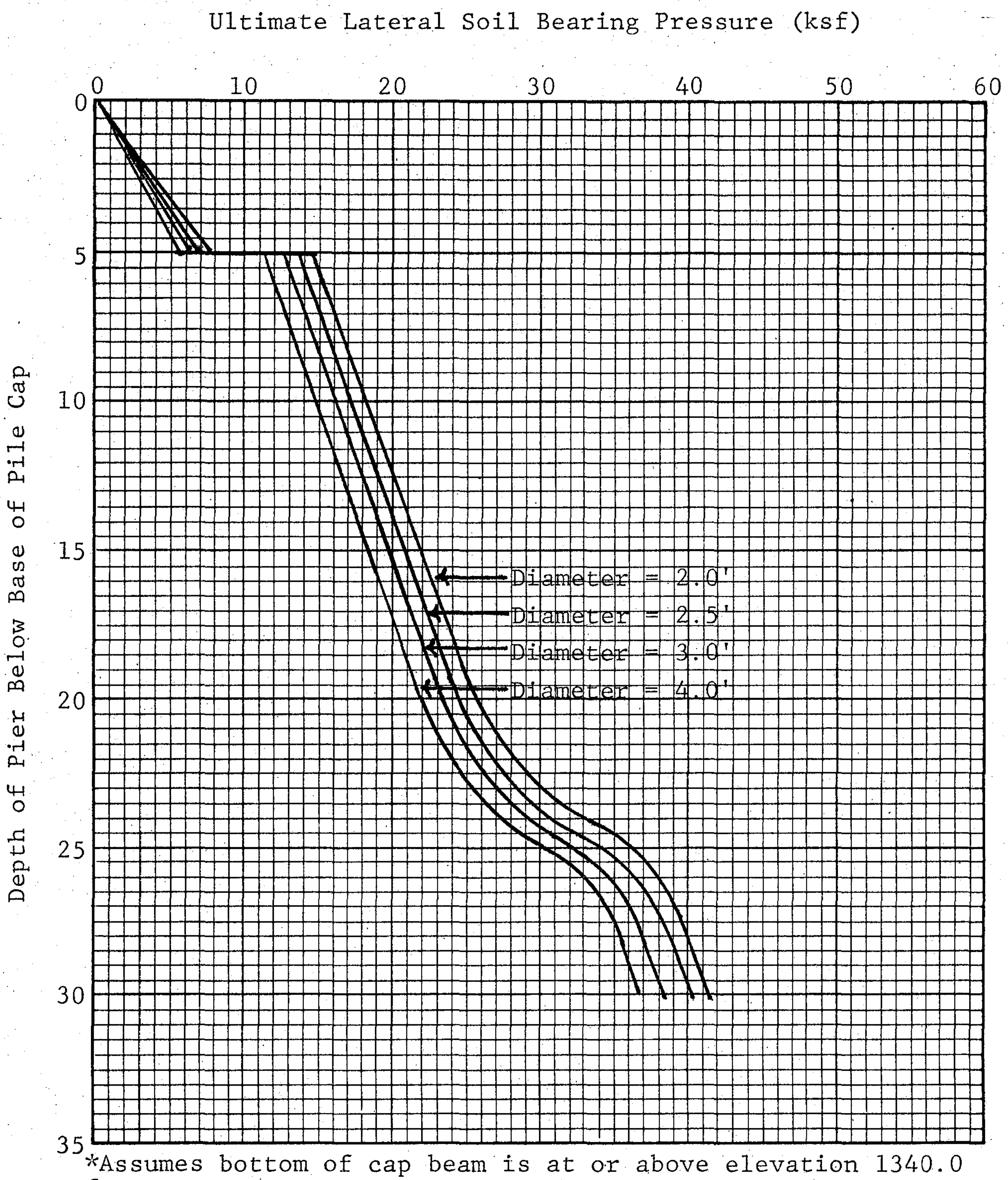
PROJECT University Drive Bridge Over RWCD Floodway JOB NO. E83-99

DESIGN CHART 3 - APPLIES TO OUTER PIER BENTS

ULTIMATE LATERAL SOIL BEARING PRESSURE FOR STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS VERSUS DEPTH*



*Assumes bottom of cap beam is at or above elevation 1340.0 feet.

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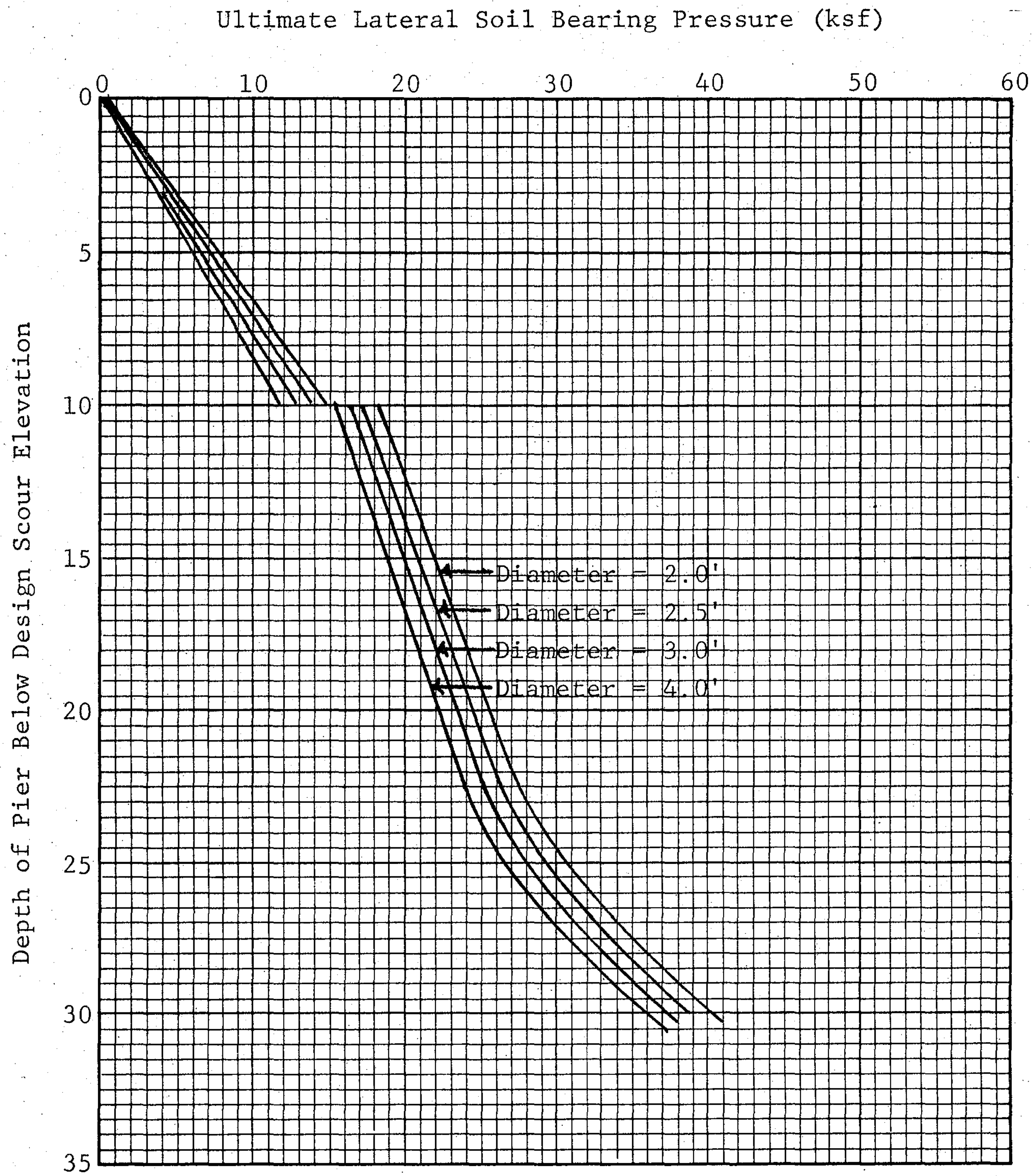
PROJECT University Drive Bridge Over RWCD Floodway JOB NO. E83-99

DESIGN CHART 4 - APPLIES TO INTERIOR PIER BENTS

ULTIMATE LATERAL SOIL BEARING PRESSURE FOR STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE PIERS VERSUS DEPTH



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