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GEOTECHNICAL INVESTIGATION REPORT

Proposed Bridge

Van Buren Over Agua Fria River

Maricopa County, Arizona

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March 14, 1985

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SHB Job No. E84-121

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Attention: E. M. Plummer, P.E.

Re: Proposed Bridge
Van Buren Over Agua Fria River
Maricopa County, Arizona

Gentlemen:

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

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

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

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

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

Registered Professional Engineer (Civil) CERTIFICATE NO. 5533 GEORGE HENRY BECKWITH Date Signed 3/14/85 ARIZONA, U.S.A.

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1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm at the site of the proposed Van Buren Road Bridge over the Agua Fria River in Maricopa County, Arizona. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site in order to provide recommendations for foundation and abutment design.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided by E. M. Plummer, P.E. of E. M. Plummer Consulting Engineers, Inc.

It is understood that a 1,160-foot long bridge with ten 116-foot spans will be constructed. Pier bents and abutments are planned to be supported by three piers. Maximum vertical loads of 1,605 kips and 2,924 kips are anticipated for the abutments and pier bents, respectively. Thus, single pier loads will be about 535 kips for the abutments and about 976 kips for the pier bent locations.

3. INVESTIGATION

3.1 Subsurface Exploration

Nine exploratory borings were drilled to auger refusal to depths ranging from 12 to 37 feet below existing grade. The borings were advanced using 6 5/8-inch O.D.

hollow stem auger and CME truck-mounted drill rig. Two of the borings were extended with a 6-inch diameter ODEX system to a depth of about 80 feet below existing grade.

Two additional borings were advanced utilizing a Becker Hammer drill to depths of about 89 feet below existing grade. The Becker Hammer drill advances a double-walled drive pipe with a link belt pile driving hammer. The drive pipe used was 6-inch O.D. by 3 3/4-inch I.D.

The results of the field investigation are presented in Appendix A, which include 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 Anthony J. Freiman and Suang S. Cheng, staff engineers of this firm.

3.2 Laboratory Analysis

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

Grain-size analysis and Atterberg Limits tests were performed on a selected sample. 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

An existing two-lane roadway, constructed on-grade, crosses the Agua Fria River along the proposed bridge alignment. The vegetation is sparse, consisting of small desert brush and weeds.

4.2 Geotechnical Profile

The subsurface profile at the site is highly stratified. The upper-lying soils, which extend to depths of approximately 50 to 60 feet below existing grade, consist of stratified deposits of poorly graded, clean sands, silty and clayey sands, silty and clayey sands and gravels and relatively clean sands and gravels. These soils are loose to medium dense near the surface and become dense to very dense with depth.

Finer-grained soil deposits underlying the surface soils are predominantly clayey sands and sandy clays of low to medium plasticity. These soils are very firm to hard.

The upper 10 to 15 feet of the granular soils are generally clean and uncemented. The granular soils below this surface layer contain varying amounts of clay and probably possess at least a weak degree of cementation. The contact of these two soil layers probably represents the maximum depth of scour during Holocene time (about the past 10,000 years).

4.3 Soil Moisture & Groundwater Conditions

Groundwater was encountered in four of the borings at depths ranging from 32 to 85 feet below existing grade. Groundwater encountered in Borings 3, 7 and 9 was at depths of 35, 32 and 33.5 feet, respectively. These shallow perched water zones are present primarily after major flows in the Agua Fria. Groundwater was encountered in Boring 11 at a depth of 85 feet.

Regional groundwater levels for the general site area as recorded in 1977 vary from 100 to 200 feet below the ground surface (Ross, 1978)*. The groundwater in Boring 11 is possibly part of the regional groundwater regime and not a seasonal perched zone.

Soil moisture contents were relatively low in zones in which perched groundwater was not encountered.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

In order to provide a high capacity foundation system which can be installed to the depths necessary to ensure foundation stability for conditions of extreme scour,

*References are listed at end of report.

straight, drilled, cast-in-place concrete piers are recommended for the support of the bridge. Due to the caving tendencies of the upper silty sand and gravel deposits and the varying degrees of cementation in the deeper silty and clayey sand and gravel deposits, efficient construction of the drilled piers may require the use of casing or slurry assisted drilling techniques.

The exact construction techniques necessary for efficient excavation of piers are uncertain. It may be most practicable to extend casing by vibratory methods to the contact of the clayey granular soils and complete the holes as open excavations. Although some degree of caving would occur in the deeper soils, the concrete overruns might be less costly than the use of slurry assisted construction. The method of excavation should be left up to the contractor with either slurry assisted construction or the open excavation method with surface casing being allowed.

Driven H-piles with tip reinforcement are an alternative foundation system which is technically feasible for the project. However, we doubt that this system is economically competitive due to the deep cap excavation to design scour elevation, which would be required along with other factors as discussed below.

Factors that tend to make drilled piers more economical than driven H-piles include the very high load capacities for a single foundation element and the

ability to extend the piers to the bridge girders without requiring a cap. In contrast, substantial excavation, backfill, and cap construction is necessary for H-piles. Also, there would be a question whether the driven H-piles could be advanced to the required depths below extreme scour elevation. A program of driving of test piles would be necessary to confirm that they could be extended to the required depths. Recommendations for driven piles can be provided by this office if desired.

The use of deep spread-type footings is also technically feasible. However, in view of the depths that would be required to provide scour protection, and the consequent volume of excavation and backfill, this approach does not appear to be practical.

Potential erosion of the channel around foundations due to the combination of general scour, local scour, and long-term degradation is a critical factor in establishing design criteria for foundation support. Foundation design recommendations given herein are in terms of the design scour depth.

In order to allow consideration of various levels of risk of channel erosion in design, two criteria are presented in Section 5.2.1 for determination of required pier depths. Criterion 1 applies to the extreme case of channel scour selected for design. Pier capacities for this criterion, based on the conservative assumption of full saturation of the supporting soils, are computed on

the basis of the submerged unit weights of the soil. A factor of safety of 1.5, applicable to extreme loading conditions, is recommended for this criterion. Criterion 2 applies to a combination of local and general scour (and perhaps some increment of long-term degradation) which could reasonably be expected to take place several times during the life of the bridge. Capacities for this criterion are calculated on the basis of moist unit weights (no development of a water table). Recommended factors of safety recommended for this criterion are 2.5 for long-term loads and 1.75 for long-term plus short-term live loads and seismic loads. Pier depths should be established by the greatest of the two tip elevations determined by the two criteria.

Because of the uncertainty of long-term degradation and the complex variables involved in local and general scour estimates, some calculated risk of channel erosion exceeding the design scour depth is involved. A margin of safety is afforded for this case by the factors of safety incorporated into the two design criteria.

Several methods for reducing erosion potential, including cutoff walls, trash racks, protective piles or piers, and riprap blankets, could be employed as a part of the design. The use of one of these methods, or some combination thereof, might allow a reduction of the design scour depths for the two design criteria presented. A discussion of these methods is given in Section 5.4.

5.2 Drilled Piers

5.2.1 Vertical Capacities

Ultimate downward capacity versus depth for various diameters of drilled piers for Criteria 1 and 2 are presented in Appendix C. Ultimate downward capacities for Criterion 1, the extreme scour condition, are presented in Figure 1 in Appendix C. A factor of safety of 1.5 is recommended for the determination of safe downward capacities for this condition. Ultimate downward capacities for Criterion 2, which models the condition of local pier scour plus general channel scour, are presented in Figure 2 in Appendix C. Factors of safety of 2.5 and 1.75 should be used in determining safe downward capacities for the cases of long-term loading and long-term loading plus short-term live and seismic loading, respectively.

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 $\frac{1}{4}$ inch.

5.2.3 Resistance to Lateral Loads

Recommended ultimate lateral bearing pressures versus depth for various pier diameters were established by

the procedure developed by Hansen (1961). The ultimate lateral capacities for Criteria 1 and 2 are shown in Figure 3, which is included in Appendix C. Factors of safety of 2.5 for long-term loads and 1.75 for extreme loads such as seismic forces and hydraulic forces during scour are recommended for Criterion 2. A factor of safety of 1.5 for total loads, including extreme seismic and hydraulic forces, are recommended for Criterion 1.

The drilled piers will be of a sufficient length to be defined as long piers. Thus, commonly applied rigid pier analyses do not accurately model the load deflection characteristics of these piers. A nondimensional procedure developed by Reese and Matlock (1956) is recommended for calculating moments and deflections in the initial part of the analysis.

The elastic analysis of Reese and Matlock is summarized by Tomlinson (1977). A value of n_h of 34 pci for Criterion 1 and 50 pci for Criterion 2 are recommended for the application of this method.

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.

Soil-pier interaction analysis can be performed by this firm by the computer program COM-622 developed by Reese (1977), when exact lateral loads become

available. In this method a finite difference technique is used which is based on elastic beam column theory. Soil behavior is described by p-y (soil reaction- displacement) curves. This approach more accurately models soil pier interaction than the simplified techniques.

5.2.4 Construction & Quality Control

Continuous observation of the construction of drilled piers should be carried out by a representative of the geotechnical engineer. The representative 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 representative to ensure that it meets requirements. A quality control report should be submitted on each pier stating, in writing, that all details have been observed and meet the requirements. Stabilization of the soils may be necessary to construct drilled piers to the desired depths. Slurry assisted construction or open hole excavation using surface casing through the surface soil are believed to be the most efficient means of advancing drilled pier excavations to the depth required. Guide specifications for construction of piers by the slurry assisted method are given in Appendix D.

In accordance with Section 9 of the guide specifications, four 2-inch inside diameter steel pipes should

be cast in each pier to allow nondestructive testing with a gamma ray scattering probe if slurry assisted construction techniques are employed. Recommendations for the design and installation of this system can be provided by this firm if needed.

5.3 Abutment Walls

5.3.1 Backfill

Because of the potential for flow of water adjacent to the approach fill and relatively rapid drawdown, a clean, granular, free-draining backfill is recommended for use behind the abutment wall wing and retaining walls in conjunction with a weephole system. It should meet the following grading requirements as determined by ASTM D422.

Sieve Size (Square Openings)	Percent Passing by Dry Weight
3 inch	100
no. 4	30-70
no. 200	0-5

The material should be nonplastic when tested by ASTM D4318. Backfill should be compacted to at least 95 percent of maximum dry density in accordance with ASTM D1557.

5.3.2 Lateral Earth Pressures

The earth pressure against abutment walls depends upon

the degree of restraint. With the recommended backfill and drainage conditions given in Section 5.3.1, rigid, absolutely restrained abutments will be subjected to earth pressures represented by a hydrostatic load diagram of about 55 pounds per square foot per foot of depth. Lateral translation of the wall 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 45 pounds per square foot per foot of depth. These values are recommended for use in establishing the design earth pressures considering the anticipated magnitude of wall movement.

5.4 Methods for the Reduction of Scour Potential

Several possible procedures are available which, depending upon the exact details, would minimize or essentially eliminate the potential for scour damage. Several measures which may be feasible for the project are as follows:

A. Cutoff Walls. Placement of a cutoff wall downstream of the structure to control the elevation of flow. This could be accomplished by installation of a continuous wall by slurry assisted construction or by drilling a row of cast-in-place concrete piers by the slurry method. The methodology used in this type of construction and case histories involving similar ground conditions are outlined by Boyes (1975) and Xanthakos (1979).

A variation of this procedure where precast concrete wall sections are placed in a hole held open by the slurry with the annulus being backfilled is discussed by Boyes (1975). This approach has the advantage of enabling the use of thinner sections of concrete in that the width of the slurry wall is controlled by the geometry of excavation equipment and generally must be at least 2.5 feet for the types of soil conditions involved.

Another variation of a cutoff wall would be to dig an open trench and fill it with soil-cement or lean concrete mixed with local materials. Deep lift compaction of this type of a system could be accomplished with vibratory rollers.

B. Trash Racks. Case histories (Brice, et al, 1978) indicate that scour is aggravated by debris hanging up on the piers and changing their effective width and hydraulic characteristics. Placement of metal trash racks 15.0 and 20.0 feet upstream from the piers is, thus, a method that can minimize the possibility of deep local scour.

C. Protective Piers. Placement of a single pier or group of piers upstream of the bridge pier appears to be an effective way of reducing local scour. Based on model studies, Chang and Korim (1971) have reported that scour depths have been effectively reduced in the range of 25 to 40 percent with this approach.

D. Riprap. Placing riprap around the portion of the piers subject to the most intense local scour is a technique that is effective in that it increases the velocity of insipient scour and reduces the overall rate of scour during flooding of a major storm where the peaks are of finite duration.

A method which has been suggested involves placing a 5.0-foot thick zone of riprap in the interval of 5.0 to 10.0 feet below the finished channel grade for a distance of 15.0 feet outside

the edge of the pier group in all directions. Riprap could be obtained by screening the coarser particles in the channel deposits near the bridge. Criteria for the design of this type of system and for evaluating the channel velocities required to initiate scour are given by Stephenson (1979).

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Xanthakos, P.P., 1979, "Slurry Walls", McGraw-Hill Book Company, New York, New York.

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.

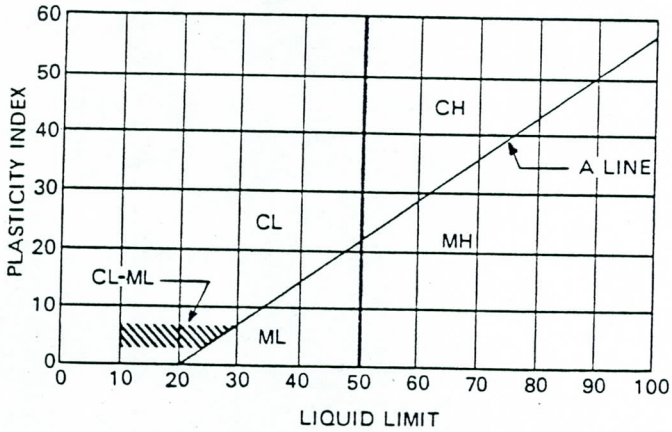
UNIFIED SOIL CLASSIFICATION SYSTEM

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

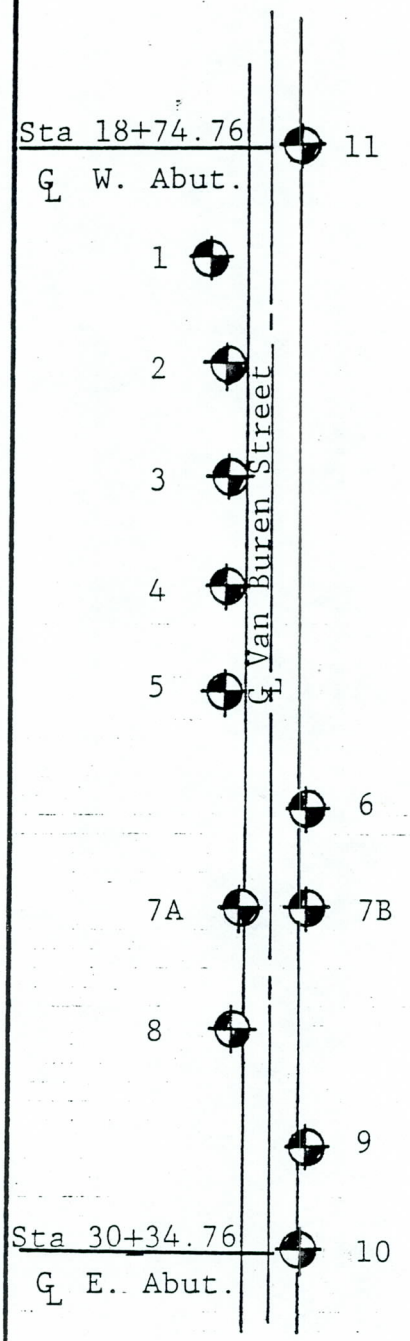
2. Relative Consistency. Terms for description of clays which are saturated or near saturation.

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



Graphic Scale

0 100 200 300

N

Boring Locations

11	Sta	18+74.76
1	Sta	19+90.76
2	Sta	21+06.76
3	Sta	22+22.76
4	Sta	23+38.76
5	Sta	24+54.76
6	Sta	25+70.76
7	Sta	26+86.76
8	Sta	28+02.76
9	Sta	29+18.76
10	Sta	30+34.76

Reference Drawing: "Van Buren Crossing of the Aqua Fria River", by Dashney & Associates, Inc., Consulting Engineers, dated 9-84

SHB Job No. E84-121

Proposed Bridge
Van Buren Over Agua Fria River
Maricopa County, Arizona
SHB Job No. E84-121

SERGENT, HAUSKINS & BECKWITH
CONSULTING SOIL AND FOUNDATION ENGINEERS
PHOENIX • ALBUQUERQUE • SANTA FE

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-4-85

LOG OF TEST BORING NO. 1

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 971.0'±

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			S		11			ML	moist moderately firm	CLAYEY SILT, medium plasticity, brown note: lens of sandy silt at 4'6"
5			S		50/5"			SP	moist very dense to medium dense	SAND, poorly graded, fine grained, subangular to subrounded, nonplastic, light brown note: some cobbles at 5'6"
10			S		21					
15			S		25			SC	moist medium dense	CLAYEY SAND, some gravel, medium to coarse grained sand, rounded to subrounded, nonplastic, brown
20			S		50/6"			GC	moist very dense	CLAYEY SAND, considerable gravel, poorly graded, rounded sand, nonplastic, brown note: cobbles & large gravel at 17'6"
25										Auger refused at 23' Hole caved to 7'6"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-4-85

LOG OF TEST BORING NO. 2

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 970.0'±

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0				S	18			SM	moist medium dense	SILTY SAND, fine grained, nonplastic, brown
5				S	17			SP	slightly moist medium dense	SAND, some gravel, poorly graded, coarse grained sand, rounded to subrounded, nonplastic, gray
10				S	49			GP	moist dense	SAND, GRAVEL & COBBLES, poorly graded, rounded to subangular, nonplastic, gray
15				S	47			SP & GP	moist dense	SILTY SAND, some gravel, poorly graded, fine to medium grained sand, nonplastic, gray to brown
20				S	50/5"			GP	moist very dense	CLAYEY SAND, GRAVEL & COBBLES, coarse grained sand, rounded, low to medium plasticity, brown
25				S	79			GP	very moist very dense	SAND, GRAVEL & COBBLES, poorly graded, subrounded, nonplastic, brown
30										Auger refused at 27'6" Hole caved to 8'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-31, 2-1, 2-12-85

LOG OF TEST BORING NO. 3

RIG TYPE CME-55 & Mobile B-80

BORING TYPE 6½" HSA & 4½" ODEX

SURFACE ELEV. 968.0'±

DATUM

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
50									very wet saturated below 43'	SILTY SAND, considerable gravel, gravel ¼" to ½", quartz, chert & volcanics, subangular to angular, nonplastic to low plasticity, various colors note: increased coarse grained sand, gravel ½" to 1", subrounded
55										
60			S	50/5½" (no recovery)						
				A						CLAYEY SAND, some gravel, poorly graded, fine grained, subangular, medium plasticity, brown note: more gravel below 72'
65								SC		
70				A						
75				A						
80										Auger refused at 19' Began ODEX at 19' Stopped ODEX at 80'

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-31, 2-1, 2-12-85

LOG OF TEST BORING NO. 3

RIG TYPE CME-55 & Mobile B-80

BORING TYPE 6½" HSA & 4½" ODEX

SURFACE ELEV. 968.0'±

DATUM

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 loose to medium dense	SILTY SAND, some gravel, poorly graded, rounded, nonplastic to low plasticity, dark brown note: some rubble & miscellaneous fill to approximately 4'
				S	23					
5				S	5			SM		
10				S	21 (no recovery)					
15				S	38			SC	moist dense	CLAYEY SAND, some gravel, poorly graded, subangular to subrounded sand, slightly lime cemented, low plasticity, brown
20				A					very moist to moist	SAND, considerable 3/8" to ½" gravel, coarse grained, subrounded to subangular, nonplastic, brown note: less gravel with depth
25				A				SP		
30				A				GP	moist to very moist	GRAVEL, considerable coarse to very coarse grained sand, gravel 3/8" to 3/4", subrounded, nonplastic, brown
35				A				GP	wet	GRAVEL, some very coarse grained sand, gravel 3/8" to ½", subrounded, nonplastic, brown note: some clay from 38' to 39'
40				A						
45				A				SW-SM		
50				A						

GROUND WATER

DEPTH	HOUR	DATE
35'	1:00p	2-1

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-1-85

LOG OF TEST BORING NO. 4

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 966.0'±

DATUM

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification	REMARKS	VISUAL CLASSIFICATION
0			☒	S	50/2"			SM	moist very dense	SILTY SAND, poorly graded, rounded, nonplastic, brown note: concrete fragments & other fill
5			☒	S	13				moist medium dense to very dense	GRAVELLY SAND, considerable silt, well graded, subangular, nonplastic, gray to brown note: asphalt fragments from 13' to 15'
10			☒	U	12	112	7	GW		
15			☒	S	50/4"					
20										Auger refused at 16' Hole caved to 13'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-1-85

LOG OF TEST BORING NO. 5

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 964.0'±

DATUM

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								SM	moist medium dense	SILTY SAND, some clay, low plasticity, brown
				S	33				moist medium dense to dense	SAND, some gravel, poorly graded, rounded to subangular, nonplastic, brown
5				S	22			SP		note: man-made fill with concrete fragments
10				S	32			SP-SC	moist dense	CLAYEY SAND, some gravel, poorly graded, rounded to subangular, low plasticity, brown
15				S	50/5"			GP		note: small cobbles to large gravel at 17'
20									moist very dense	CLAYEY SAND, GRAVEL & COBBLES, poorly graded, subrounded to subangular, low plasticity, brown
										Auger refused at 19' Hole caved to 17'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-1-85

LOG OF TEST BORING NO. 6

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 963.0'±

DATUM

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 medium dense	SAND, poorly graded, rounded to subrounded, nonplastic, brown note: trace of concrete fragments
			☒	S	15			SP		
5			☒	S	26					
									moist very dense	SAND, GRAVEL & COBBLES, poorly graded, subrounded, nonplastic, brown note: increasing size of cobbles below 10'
10			☒	S	65			GP		
15										Auger refused at 12' Hole caved to 10'

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-13-85

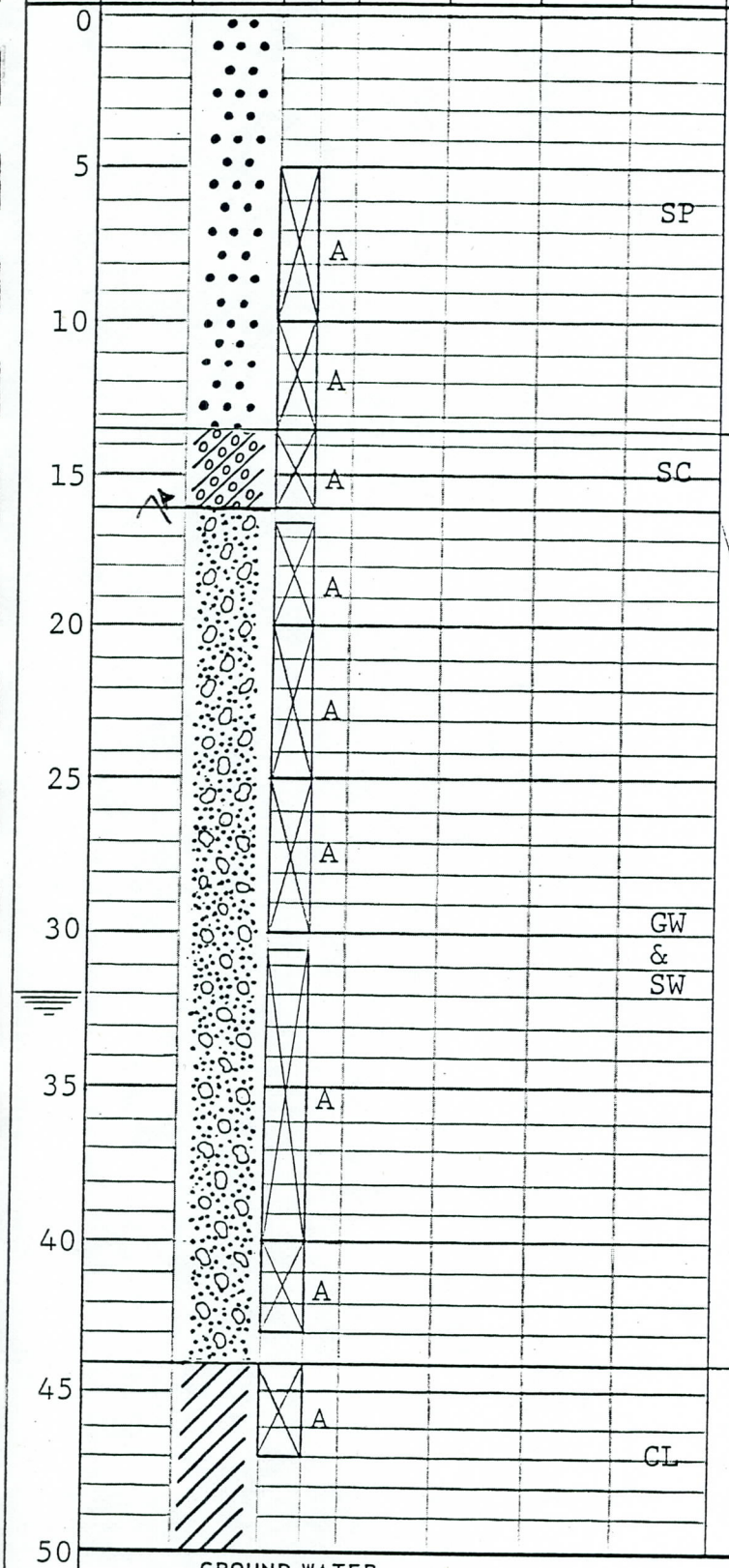
LOG OF TEST BORING NO. 7A&B

RIG TYPE Mobile B-80

BORING TYPE 6½" HSA & 4½" ODEX

SURFACE ELEV. 963.0'±

DATUM



REMARKS	VISUAL CLASSIFICATION
moist to very moist	SAND & GRAVEL, trace of clay, poorly graded, coarse grained, subangular, nonplastic, dark gray
very moist	CLAYEY SAND & GRAVEL, well graded, coarse grained, subangular, low plasticity, brown
moist	GRAVEL & SAND, well graded, coarse grained, subangular, nonplastic, dark gray note: thin layer of fine grained sand between 20' to 21' in depth note: small amount of clay below 38'
very moist to moist at 32'	
slightly moist	SILTY CLAY, considerable sand, weakly lime cemented, low plasticity, brown

GROUND WATER

DEPTH	HOUR	DATE
35'	11:00	2-15

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-13-85

LOG OF TEST BORING NO. 7A&B

RIG TYPE Mobile B-80

BORING TYPE 6½" HSA & 4½" ODEX

SURFACE ELEV. 963.0'±

DATUM

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
50				S	13				very moist moderately firm to very firm	CLAYEY SAND, well graded, fine grained, low plasticity, brown note: very clean sand at 50' to 52', increase in clay content with depth between 53' to 56' note: considerable clay between 75' to 78'
55				A						
60										
				A						
65								SC		
70										
75				A						
80				S	33					
85										Auger refused at 16' Began ODEX at 16' Stopped ODEX at 81'

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 2-4-85

LOG OF TEST BORING NO. 8

RIG TYPE CME-55

BORING TYPE 6⅜" Hollow Stem Auger

SURFACE ELEV. 964.0'±

DATUM

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 medium dense to very dense	SAND & GRAVEL, poorly graded, angular to sub-angular, nonplastic, brown note: coarse grained gravel & small cobbles below 7' increasing to large cobbles with depth
5								SP		
10			S		50/4"					
15			S		42			SC	moist dense	CLAYEY SAND, considerable gravel & cobbles, poorly graded, subangular, low plasticity, brown
20										Auger refused at 16' Hole caved to 13'9"

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-31-85

LOG OF TEST BORING NO. 9

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 966.0'±

DATUM

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 medium dense	SAND, some silt & gravel, poorly graded, rounded, nonplastic, brown
				S	18		4			
5				S	15		3			note: considerable gravel below approximately 10'
								SP-SM		
10				S	24		5			
15				S	53		22	SM	moist very dense	SILTY SAND, fine grained, low plasticity, brown
20				S	66		5	GP	moist very dense	SAND, GRAVEL & COBBLES, poorly graded, rounded, nonplastic, brown
25				S	50		4	SP-SM	moist dense to very dense	SAND, trace of silt & gravel, poorly graded, rounded, nonplastic, light brown
30				S	43			GP	moist dense to very dense	GRAVEL, some sand, poorly graded, subangular to rounded, nonplastic, light brown
35				S	50/4"					
40										Auger refused at 37' Hole caved to 19'

GROUND WATER

DEPTH	HOUR	DATE
33.5'	10:30	1-31

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-23-85

LOG OF TEST BORING NO. 10

RIG TYPE Becker Hammer Drill

BORING TYPE 6 5/8" O.D. Pipe

SURFACE ELEV. 970.0'±

DATUM

Depth in Feet	Blows Per Foot	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	0								moist dense	SILTY, GRAVELLY SAND, well graded, angular, rounded to subangular gravel, slightly lime cemented, nonplastic, brown note: cobbles below 10'
	0									
	6									
	6									
5	2			A						
	12									
	21									
	6									
	10							SW		
10	2									
	20									
	32									
	26									
	19									
15	23			A						
	31									
	72									
	51								moist dense	CLAYEY, GRAVELLY SAND, some cobbles, well graded, subrounded to angular gravel, slightly lime cemented, low plasticity, dark brown
	45									
20	20							SC		
	11									
	27									
	25									
	4			A						
25	15								moist very dense	GRAVELLY SAND, some cobbles, well graded, angular, subangular to rounded gravel, slightly lime cemented, low plasticity, brown
	18									
	32									
	26							SW		
	58									
30	50									
	55									
	54									
	49									
	28									
35	28			A					moist very dense	SANDY, CLAYEY GRAVEL, some cobbles, well graded, subangular to rounded, slightly lime cemented, medium plasticity, brown
	45									
	45									
	65									
	40									
40	37									
	65									
	71							GW-GC		
	38									
	79			A			5			
45	144									
	42									
	50									
	76									
	104									
	99									
50										

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-23-85

LOG OF TEST BORING NO. 10

RIG TYPE Becker Hammer Drill

BORING TYPE 6 5/8" O.D. Pipe

SURFACE ELEV. 970.0'±

DATUM

Depth in Feet	Blows Per Foot	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
50	164									
	93									
	63							GW-		
	46							GC		
55	30			A						
	43									
	21								very moist	CLAYEY, GRAVELLY SAND,
	23								very dense	some cobbles, well
	13							SW		graded, angular, sub-
	11									angular gravel, slightly
60	22									lime cemented, medium to
	159									high plasticity, brown
	72									
	20			A					very moist	CLAYEY SAND, some grav-
65	14								firm	el, angular well graded
	3									sand, subrounded poorly
	8									graded gravel, slightly
	24									lime cemented, medium
	42									plasticity, dark brown
70	22									
	32							SC		
	62									
	36									
	37			A			23			
75	39									
	26									
	38									
	66									
	28									
80	2									
	37									
	71									
	63								very moist	SILTY SAND, some clay
	37			A			20		firm	& gravel, poorly graded,
85	33									fine grained, subrounded
	21							SM		gravel, slightly lime
	42									cemented, low plasticity,
	61									dark brown
	126									
90										Stopped hammer at 89'

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-24-85

LOG OF TEST BORING NO. 11

RIG TYPE Becker Hammer Drill

BORING TYPE 6 5/8" O.D. Pipe

SURFACE ELEV. 972.0'±

DATUM

Depth in Feet	Blows Per Foot	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	28								slightly moist to moist	GRAVELLY, SILTY SAND, poorly graded, fine to medium grained, subangular to subrounded gravel, slightly lime cemented, nonplastic, brown
	16								very dense	
	11									
	11									
5	12			A						
	3									
	1									
	1									
	1									
10	4									
	1									
	1									
	1									
	4							SP		note: cobbles below 20'
15	8			A						
	10									
	16									
	21									
	16									
20	28									
	28									
	26									
	25									
	17			A						
25	23									
	46									
	27									
	40								moist	SANDY, SILTY GRAVEL, some cobbles, well graded, subrounded to angular, angular sand, moderately lime cemented, nonplastic, brown
	64								very dense	
30	49									
	50									
	33									
	46									
	51			A			5			
35	43									
	27									
	33									
	14									
	37									
40	41							GP		
	47									
	56									
	92									
	36			A						
45	45									
	150									
	100									
	118									
	106									
50	80									

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT Proposed Bridge

JOB NO. E84-121 DATE 1-24-85

LOG OF TEST BORING NO. 11

RIG TYPE Becker Hammer Drill

BORING TYPE 6 5/8" O.D. Pipe

SURFACE ELEV. 972.0'±

DATUM

Depth in Feet	Blows Per Foot	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
50	77							GP		
	61									
	68								moist	SANDY, CLAYEY GRAVEL, some silt & cobbles, well graded, subrounded to angular, slightly lime cemented, low plasticity, brown
	55			A					very dense	
	82									
55	47							GC		
	49									
	47									
	23									
60	14								very moist to saturated	SANDY CLAY, some gravel, poorly graded, medium grained sand, slightly lime cemented, medium plasticity, brown
	20									
	23									
	26								very firm	
	21			A			8			
65	42									
	38									
	24									
	45									
	58									
70	62									
	40									
	57									
	31									
	48			A						
75	70							CL		
	100									
	47									
	36									
	66									
80	23									
	25									
	60									
	85									
	35			A						
85	31									
	13									
	48									
	28									
	20									
90										Stopped hammer at 89'

GROUND WATER

DEPTH	HOUR	DATE
85'	12:00	1-24

SAMPLE TYPE

A – Auger cuttings. B – Block sample

S – 2" O.D. 1.38" I.D. tube sample.

SERGENT, HAUSKINS & BECKWITH

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.

SERGENT, HAUSKINS & BECKWITH

TABULATION OF TEST RESULTS

Job No. E84-121

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	SIEVE ANALYSIS-ACCUM % PASSING #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
1	39'-49'	GW-GC	36	15	10	12	15	18	22	31	37	39	46	53	64	73	
					81	85	92	96	100								4-121-6
1	69'-79'	SC	42	18	49	55	65	68	72	76	79	80	85	87	89	92	
					94	100											4-121-9
1	79'-89'	SM	33	9	36	47	67	72	75	78	79	79	81	82	85	86	
					95	100											4-121-10
11	29'-39'	GP	20	NP	4.3	6	8	10	14	26	33	34	39	42	45	51	
					56	63	77	89	93	100							4-121-14
11	59'-69'	SC	41	20	31	34	42	46	51	60	64	64	67	69	72	76	
					80	83	89	94	100								4-121-17
3	40'-45'	SW-SM	24	2	11	14	21	25	30	40	47	50	66	75	88	96	
					100												4-121-24
3	14.5'-16'	SC	36	19	13	15	20	26	38	74	84	87	91	92	95	97	
					100												4-121-28
5	9.5'-11'	SP-SC	28	8	8.5	11	16	21	26	38	46	48	56	60	68	75	
					79	100											4-121-37
9	9.5'-11'	SP-SM	-	NP	6.8	10	21	31	42	60	61	62	72	75	79	82	
					84	100											4-121-44
9	24.5'-26'	SP-SM	-	NP	6.8	14	32	49	66	89	95	97	100				4-121-47

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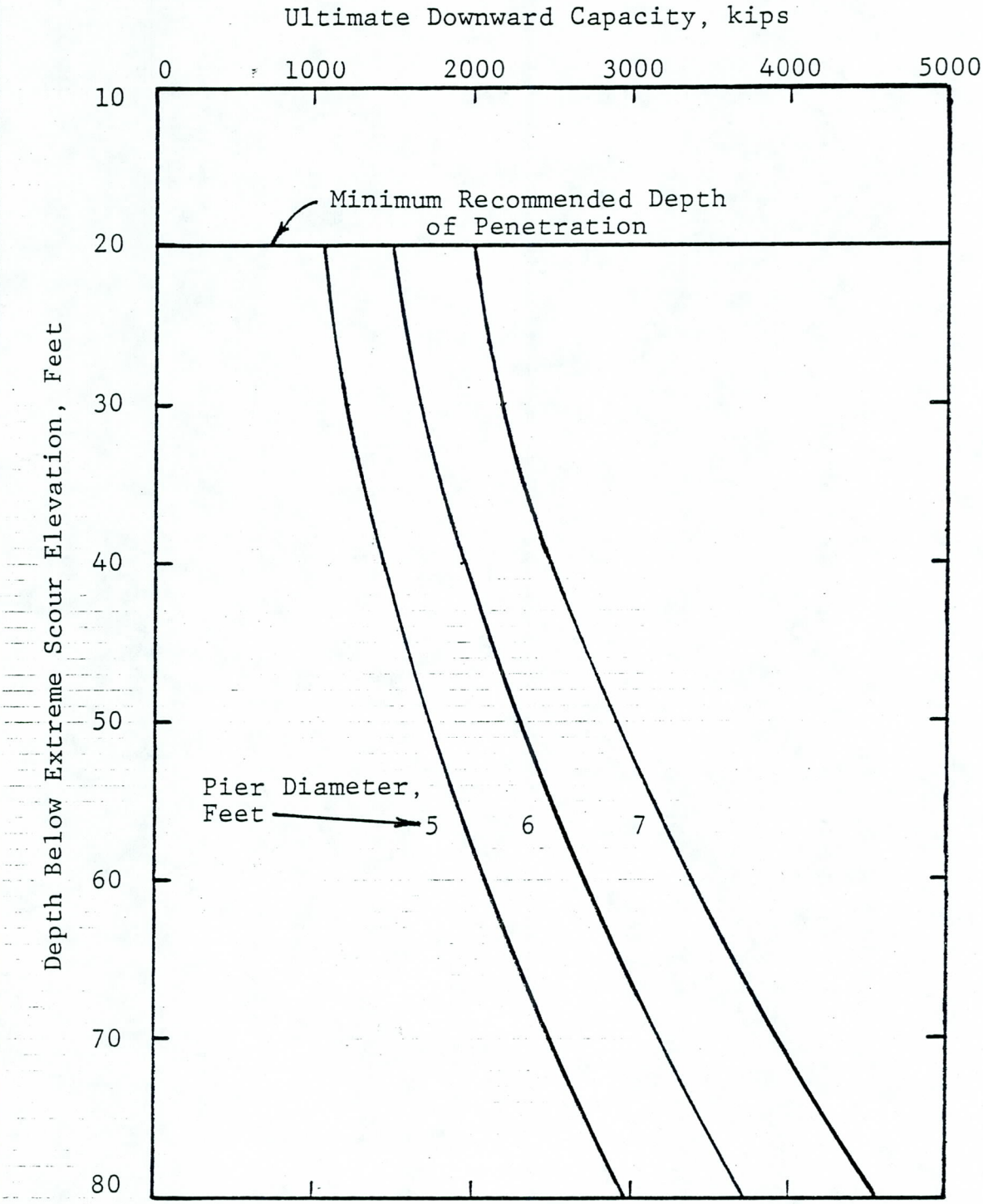
TABULATION OF TEST RESULTS

Job No. E84-121

HOLE NO	DEPTH	MOISTURE CONTENT	LAB NO
1	39'-49'	5.2 %	4-121-6
1	69'-79'	23.3 %	4-121-9
1	79'-89'	19.8 %	4-121-10
11	29'-39'	4.6 %	4-121-14
11	59'-69'	8.1 %	4-121-17
3	40'-45'	0 %	4-121-24
3	14.5'-16'	9.6 %	4-121-28
5	9.5'-11'	6.4 %	4-121-37
9	9.5'-11'	5.2 %	4-121-44
9	24.5'-26'	4 %	4-121-47

FIGURE 1

ULTIMATE DOWNWARD CAPACITIES
OF STRAIGHT, DRILLED, CAST-IN-PLACE
CONCRETE PIERS VERSUS DEPTH
(Criterion 1)



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Project Van Buren Bridge
Job No: E84-121
Computed by: JRF Ckd. by:

FIGURE 2

ULTIMATE DOWNWARD CAPACITIES
OF STRAIGHT, DRILLED CAST-IN-PLACE
CONCRETE PIERS VERSUS DEPTH
(Criterion 2)

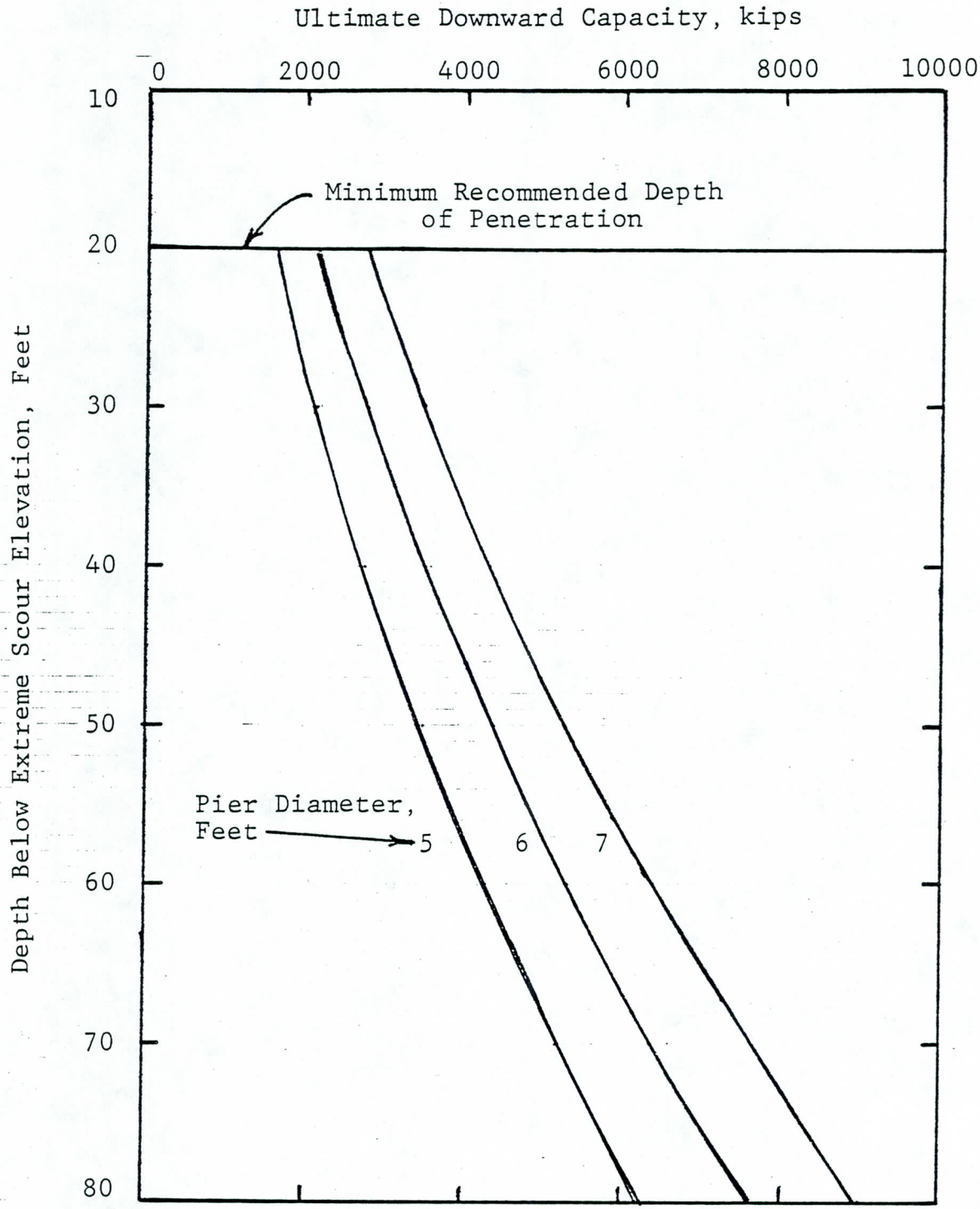
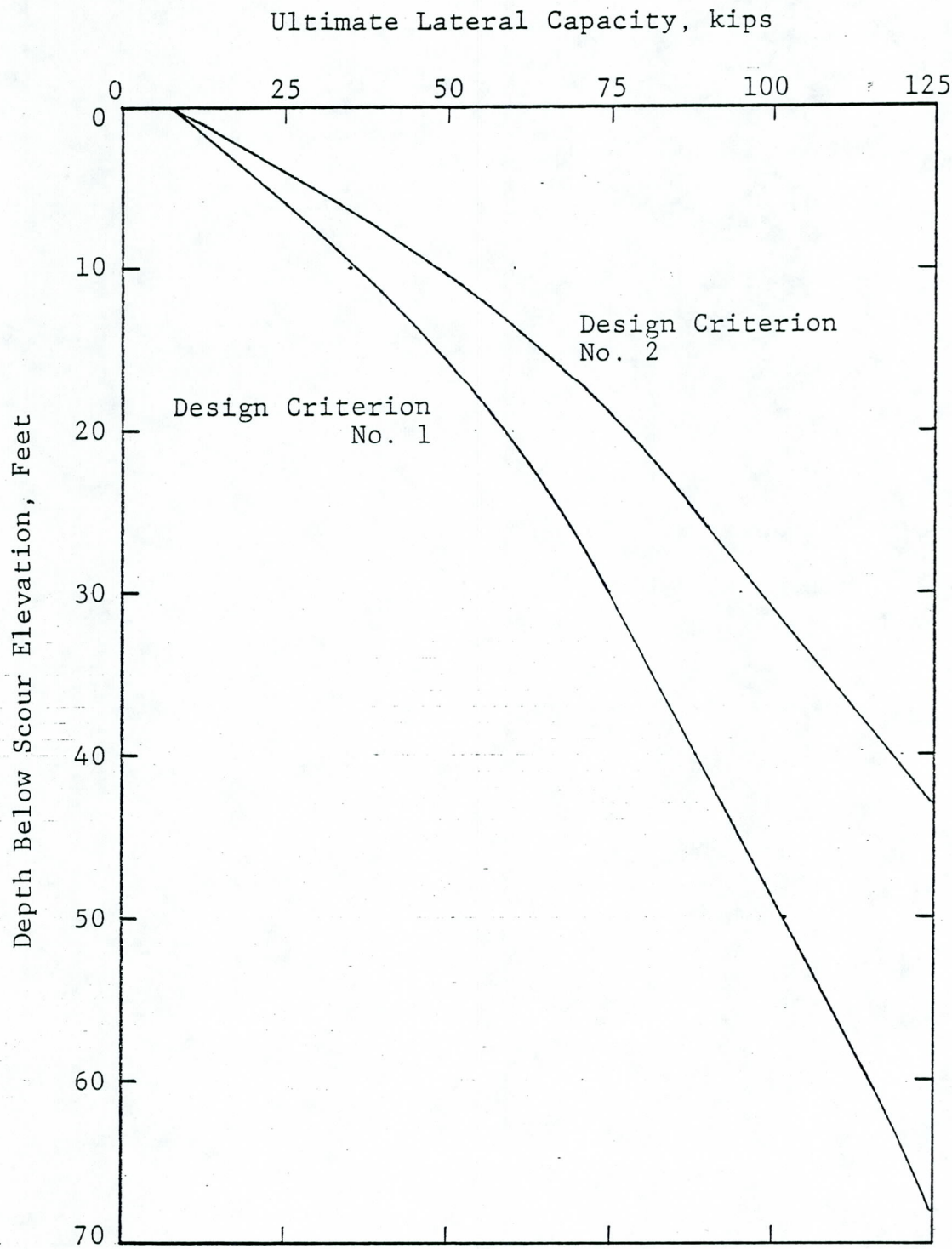


FIGURE 3

ULTIMATE LATERAL CAPACITY
VERSUS DEPTH FOR DRILLED PIERS



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Project Van Buren Bridge
Job No: E84-121
Computed by: JRF Ckd. by:

Proposed Bridge
Van Buren Over Agua Fria River
Maricopa County, Arizona
SHB Job No. E84-121

GUIDE SPECIFICATIONS FOR DRILLED, CAST-IN-PLACE, CONCRETE PIERS UTILIZING SLURRY ASSISTED CONSTRUCTION

1. Design & Construction Considerations

Among the important considerations in design and construction with slurry assisted construction are:

A. Careful control of slurry consistency.

B. Use of a rich concrete mix designed for pumping with the slump in the range of 6 to 8 inches.

C. Use of a steel tremie pipe at least 6 inches in diameter with a high enough capacity pump to operate efficiently at the depths involved.

D. Careful quality control observations to ensure that the bottom of the tremie pipe is kept at least 5.0 feet below the surface of the concrete throughout the pumping operation.

E. Providing at least 6 inches of clearance between the reinforcing steel and the walls of the hole, a minimum of 6 inches between vertical bars, and a minimum of 12 inches between horizontal ties.

F. A 20 percent reduction of bond between the reinforcing steel and concrete is normally used in structural design where the slurry method is used.

A general summary of important elements of specifications for construction of drilled piers by the slurry displacement method is given as follows. These are based on various published criteria including those of the British Federation of Piling Specialists (1975), the Institution of Civil Engineers (1978), Reese and Wright (1977), Boyes (1975), and Xanthakos (1979).

2. Contractor Qualification & Test Pier

Proper construction procedures and techniques are essential in obtaining drilled piers with the load carrying capacity for which they are designed. Therefore, it is critical that the contractor have slurry assisted drilled pier experience and be able to commit experienced people to the project. The foundation contractor should be required to successfully complete a test pier prior to beginning the production piers or the contract should be terminated.

3. Observation of Construction

Continuous observation of the construction of drilled piers is critical and should be made by a representative of the geotechnical engineer. A representative of the geotechnical engineer should verify the nature of materials encountered in pier excavations. The excavation, slurry placement, and concrete placement should be continuously observed to ensure that they meet requirements. A quality control report should be submitted on each pier confirming, in writing, that all details have been observed and meet requirements.

4. Excavation of Drilled Piers

Straight, drilled pier excavations should be advanced with a single flight auger or bucket auger to depths indicated by the plans. Caving of the hole should be prevented at all times by use of a slurry. It should be

verified by observations, soundings, and measurements that excavations are open to the specified depths. It should be ensured that the slurry is properly desanded and meets criteria of Section 5, and that all loose sand is removed from the base of the excavation prior to beginning concreting operations.

5. Properties of Slurry

The slurry should consist of a stable suspension of commercial grade bentonite (API Spec. 13A) in water, and it should be stirred or agitated prior to use so as to maintain a uniform consistency and viscosity. Water used for mixing should be the quality of drinking water with regard to soluble salts; although water contaminated by bacteria is acceptable. Water in which the chemical quality would permit flocculation of the bentonite should not be used. Density of the slurry should be a minimum of 64 and a maximum of 75 pounds per cubic foot. Slurry viscosity by the Marsh Cone method should be in the range of 30 to 90 seconds.

6. Placement of Slurry

The level of bentonite slurry should not be maintained lower than 4.0 feet above the level of standing groundwater or soil zones susceptible to caving. In the event of a sudden loss in bentonite, the boring should be backfilled immediately and instructions from the geotechnical engineer sought.

7. Placement of Concrete

Portland Cement concrete should be of an exceptionally rich mixture which would settle under its own weight and would not mix with the slurry. Concrete used should not contain less than 658 pounds of cement per cubic yard. A slump in the range of 6 to 8 inches is recommended. Admixtures, as needed to prevent segregation of the mix, allow free flow through the placing equipment and, to retard setting during hot weather, should be incorporated into the mix.

Concrete should be placed by means of a gravity-fed tremie pipe or a combination concrete pump and tremie pipe. The tremie pipe should be a rigid, watertight pipe for the full length of the pile and should not be less than 6 inches in diameter if pump-fed or 9 inches in diameter if gravity-fed. The pipe should be equipped with a bottom valve, or other approved device which would prevent mixing of the slurry with the concrete inside the pipe, and which would prevent the intrusion of slurry into the concrete in the event that the tremie pipe has to be removed and replaced.

Reinforcing steel should be in place and the tremie pipe should be inserted to the bottom of the drilled pier before concrete placement is started. Concrete should be placed in a continuous operation in such a manner that the concrete always flows upward within the hole. The delivery pipe should be slowly withdrawn as the elevation of the concrete in the hole rises, but the

discharge end of the pipe should, at all times, be maintained at least 5.0 feet below the surface of the concrete. Raising of the tremie pipe should be done only when the pipe contains a sufficient head of concrete to prevent the formation of a void at the tip.

During concrete placement, the contractor should provide and maintain marking on the tremie pipe, a sounding device, or other adequate methods in which to determine the relative elevation of the concrete surface and the end of the tremie pipe.

8. Disposal of Slurry

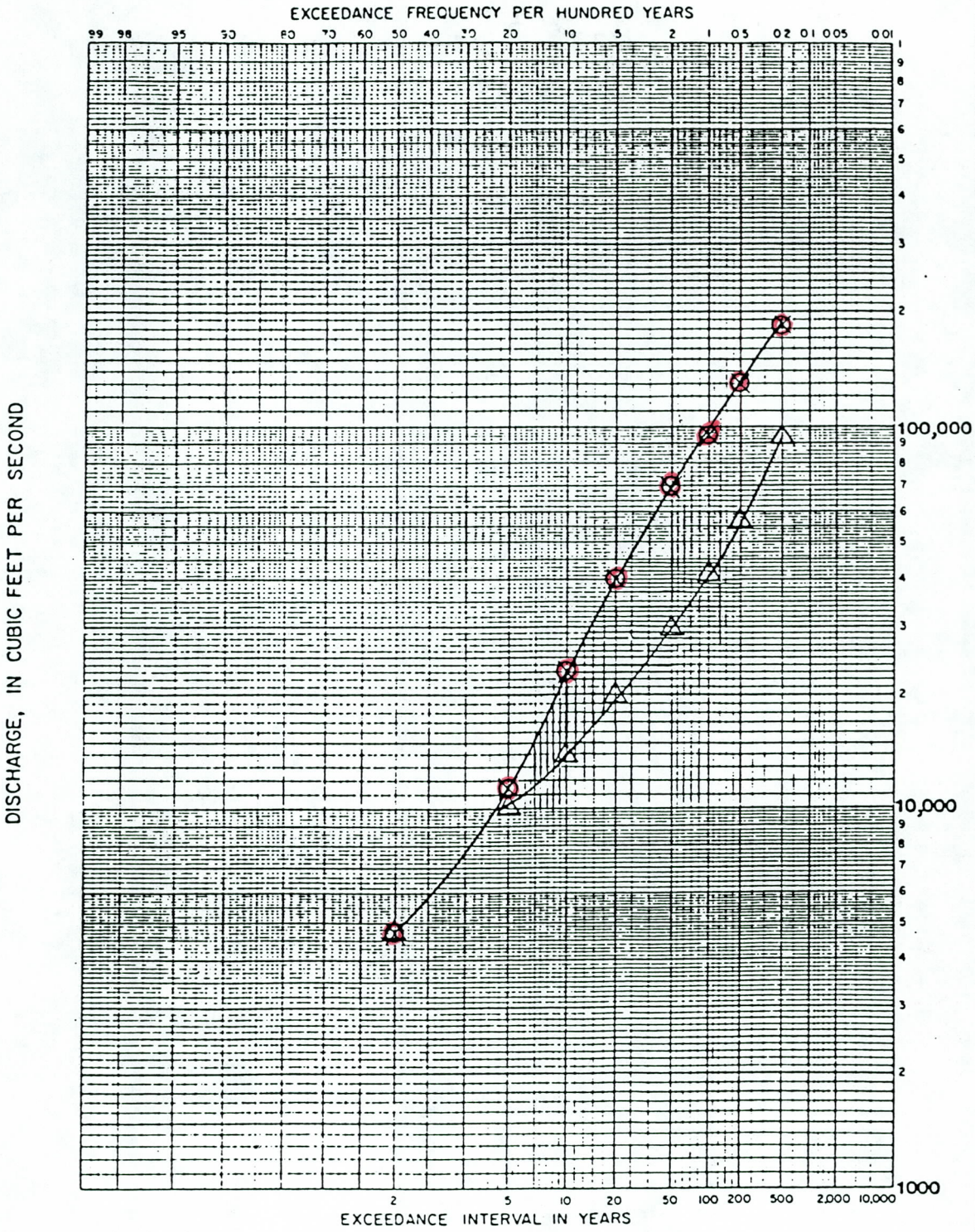
All reasonable steps should be taken to prevent spillage of slurry on the site away from the vicinity of the piers. Discarded slurry suspension should be pumped from the borings and removed from the site.

9. Nondestructive Testing

Nondestructive testing of finished concrete piers should be accomplished by means of geophysical techniques involving gamma ray scattering to verify their continuity. A minimum of four probe holes with a 2-inch I.D. should be cast in the concrete. Either PVC pipe or steel pipe can be used to form the holes. When steel pipe has been used, it can be considered as part of the pier reinforcement.

The objective of the gamma ray logging is to detect

large defects in the interior of the pier created by cave-ins which would jeopardize the integrity of the piers. Thus, casing may be placed on the inside of reinforcing bars with logging being performed by directional devices which are capable of "looking" into the interior of the piers. It should be recognized that gamma ray logging is not capable of detecting all minor defects on the outside of the reinforcement cage. Application of this method to drilled pier quality control is discussed by Priess and others (1978), Priess and Caiserman (1975), and Weltman (1977).



LEGEND:

○ -Present Conditions

× -Future W/O

△ -Future W/

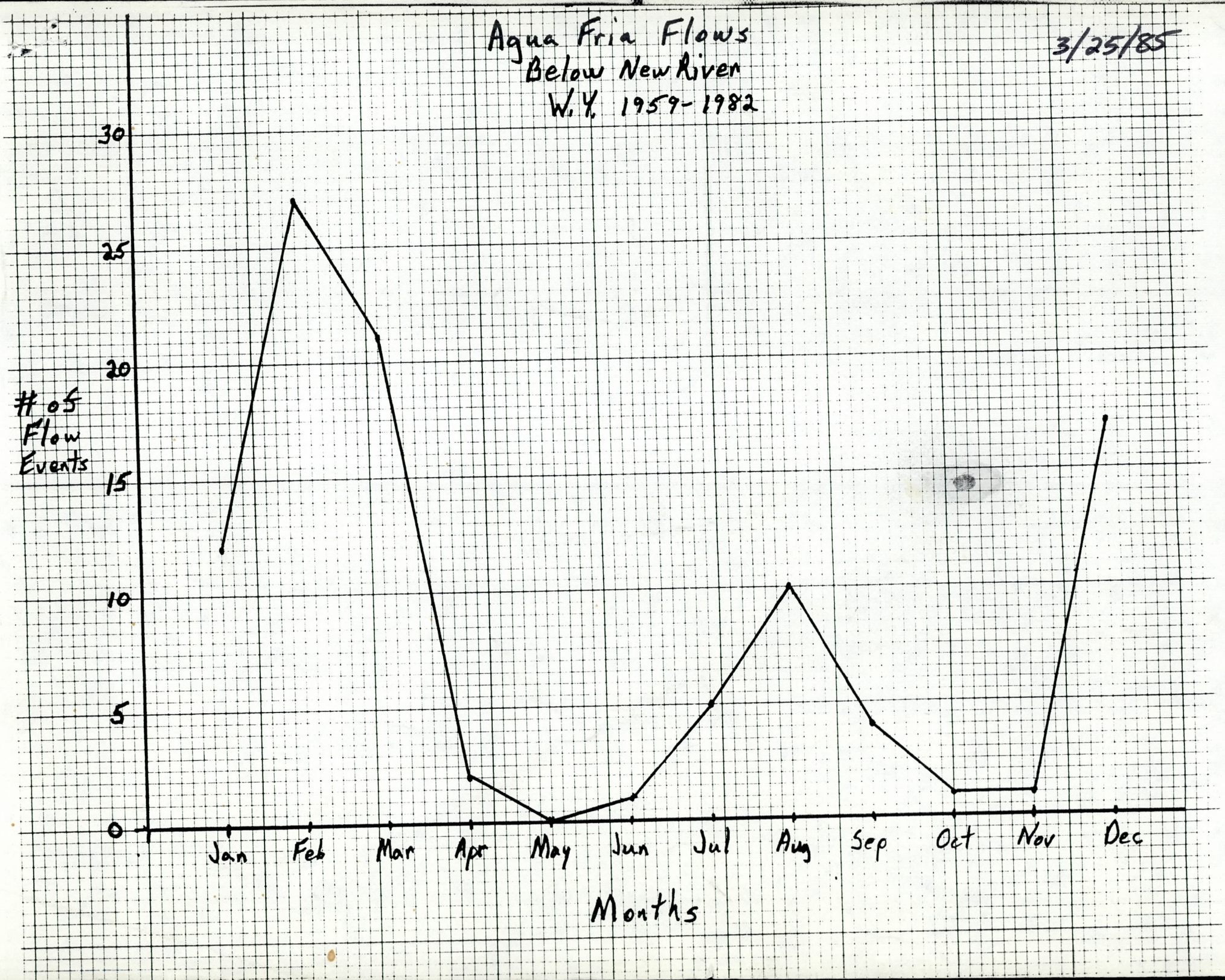
GILA RIVER AND TRIBUTARIES
CENTRAL ARIZONA WATER CONTROL STUDY

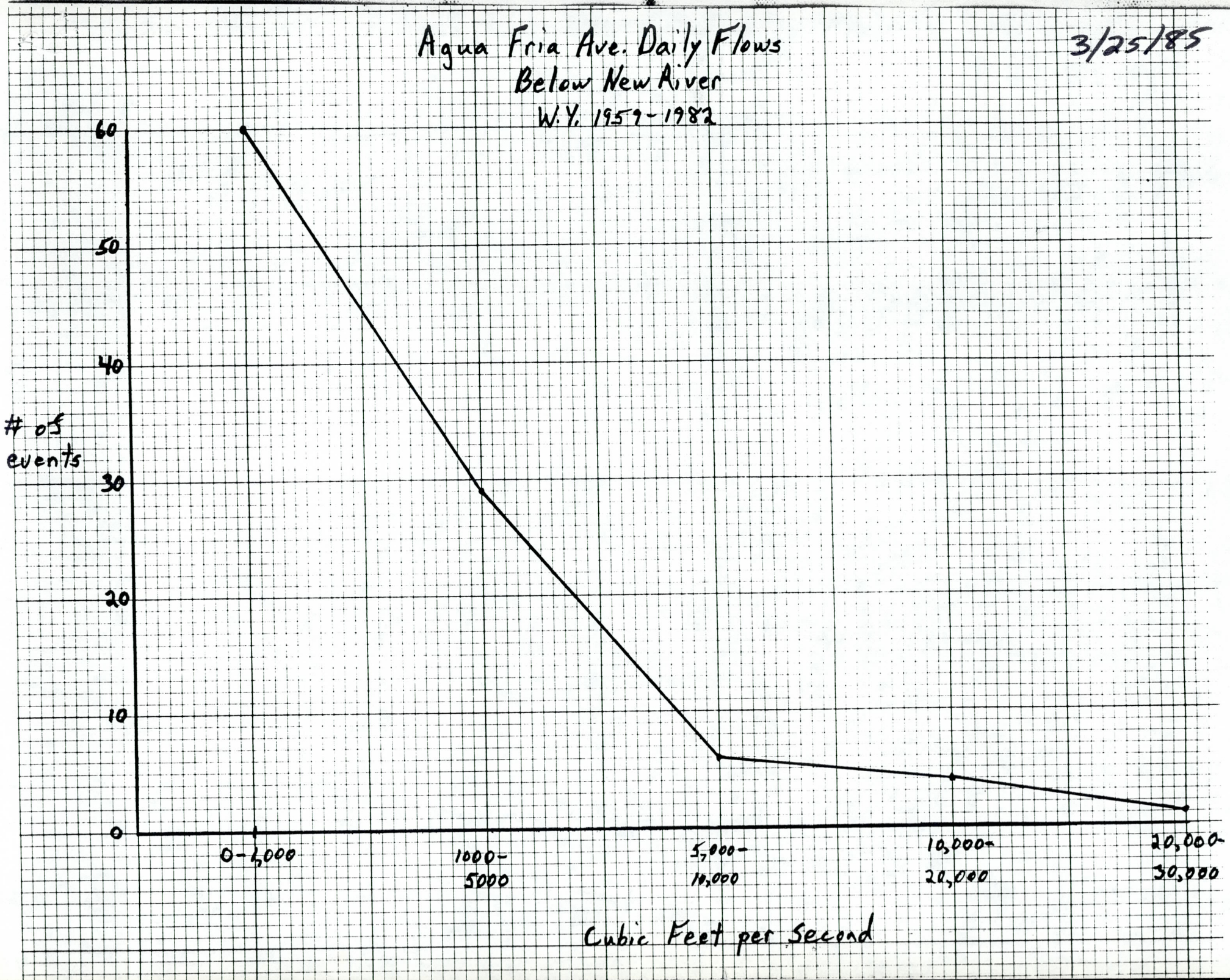
DISCHARGE FREQUENCY CURVES
AGUA FRIA RIVER
below New River
Present, Future W/O, Future W/

U S ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
TO ACCOMPANY REPORT DATED:

3/25/85

FIGURE 4


Agua Fria Flows
Below New River
W.Y. 1959-1982
3/25/85
30
25
20
15
10
5
0
# of Flow Events
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Months


Agua Fria Ave. Daily Flows
Below New River
W.Y. 1959-1982
3/25/85
60
50
40
30
20
10
0
# of events
0-1,000
1000-5000
5,000-10,000
10,000-20,000
20,000-30,000
Cubic Feet per Second