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GEOTECHNICAL INVESTIGATION REPORT

ACDC Canal Bridge

7th Avenue & Arizona Canal

Phoenix, Arizona

Registered Professional Engineer (Civil)
CERTIFICATE NO. 13872
LAWRENCE A. HANSEN
Date Signed 6/1/07
ARIZONA, U.S.A.

SHB Job No. E87-117

Consulting Geotechnical Engineers

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

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APPLIED SOIL MECHANICS • ENGINEERING GEOLOGY • MATERIALS ENGINEERING • HYDROLOGY

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June 1, 1987

HNTB
Anchor Centre II
2207 East Camelback Road
Suite 400
Phoenix, Arizona 85016

SHB Job No. E87-117

Attention: Gray C. Wangelin, P.E.

Re: ACDC Canal Bridge
7th Avenue & Arizona Canal
Phoenix, Arizona

Gentlemen:

Our Geotechnical Investigation Report on the referenced project is herewith submitted. The report includes the results of test drilling, laboratory analysis and recommended criteria for 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 _____
Gary N. Sheppard, Staff Engineer

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

Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 6/1/87 ARIZONA, U.S.A.

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Registered Professional Engineer (Civil) CERTIFICATE NO. 13872 LAWRENCE A. HANSEN Date Signed 6/1/87 ARIZONA, U.S.A.

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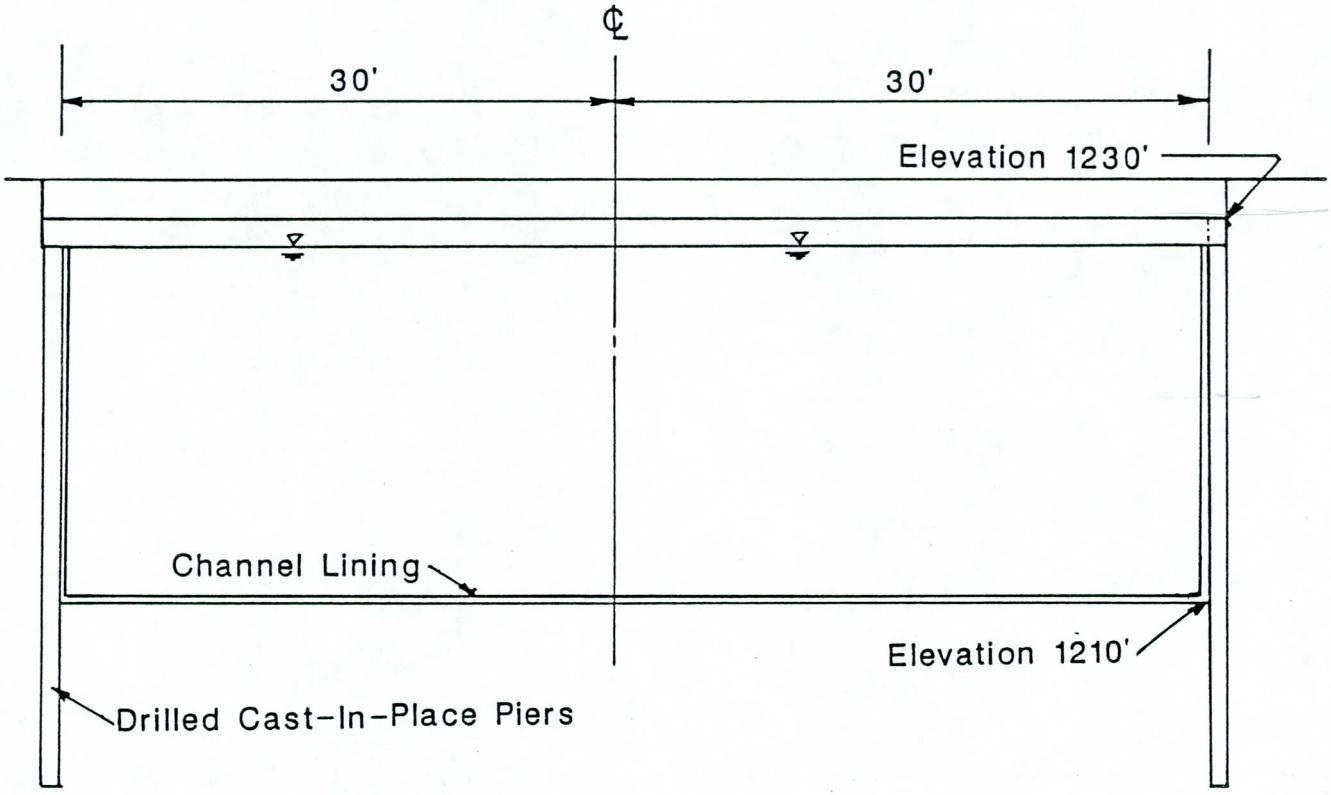
1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the proposed 7th Avenue Bridge over the Arizona Canal diversion channel located in Phoenix, Arizona. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site to provide recommendations for foundation design and abutment support.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided by Gray C. Wangelin of HNTB.

It is understood that a bridge approximately 65 feet long and 64 feet wide is planned. The invert elevation of the Arizona Canal diversion channel at the centerline of the bridge will be approximately 1,210 feet (Corps of Engineers datum). The channel will be concrete lined and rectangular in shape with a 60-foot wide bottom, as shown in Figure 1. The design flow rate is approximately 12,830 cubic feet per second at a depth of 18.5 feet and a velocity of 11.2 feet per second. It is also understood that a sewer line will be relocated parallel to the channel to include an inverted siphon at Dunlap Avenue.



Not To Scale

FIGURE 1

TYPICAL DIVERSION CHANNEL SECTION

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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

Four exploratory borings were drilled to depths of 15 to 40 feet below existing grade. The borings were performed using our CME-55 drill rig advancing a 6 5/8-inch O.D. hollow stem auger. Standard penetration testing and open-end drive sampling were performed at 5-foot intervals in the borings. All boreholes were maintained full of water during standard penetration testing and open-end drive sampling.

The results of the field investigation are presented in Appendix A, which includes a brief description of drilling and sampling equipment and procedures, a site plan showing the boring locations, and logs of the test borings. The field investigation was supervised by Keith Dahlen, staff engineer of this firm.

3.2 Laboratory Analysis

Moisture content determinations were made on selected samples recovered, and the dry density of a selected

2.42-inch drive sample was determined. The results of these tests are shown on the boring logs.

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

4. SITE CONDITIONS & GEOTECHNICAL PROFILE

4.1 Site Conditions

The proposed bridge is located along 7th Avenue between the Arizona Canal located to the south and Mission Lane located to the north. Seventh Avenue is a paved, four-lane roadway that is approximately 50 feet wide and has a curb and gutter. The site area is relatively level and void of vegetative cover.

4.2 Geotechnical Profile

The subsurface soils consist of layers of sandy and silty clays, and clayey sands with lesser deposits of sandy silts, that extend to the depth of the investigation. These soils are of low to medium plasticity and were found to be moderately firm to hard. At the locations of Borings 1 and 2, it is suspected that the upper few feet of soil is man-made fill that is related

to the 7th Avenue road construction. The soils are weakly to moderately lime cemented and it appears that the relative firmness increases with depth, as indicated by the standard penetration test blow count profile included as Figure B-1 in Appendix B.

4.3 Soil Moisture & Groundwater Conditions

No free groundwater was encountered in the borings and soil moisture contents were relatively low throughout the depth of investigation. It is our opinion that in situ moisture conditions are somewhat drier than those reported. Due to maintaining the borehole full of water during standard penetration testing, increased moisture contents were encountered. This is apparently the result of water being forced between the sample and the inner wall of the sample.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

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

Spread-type footings can be used as an alternate to the straight, or belled, cast-in-place, drilled concrete

piers at the abutment locations. The concrete channel lining will protect the near-surface soils from scour. Design criteria for spread-type footings are presented in Section 5.3.

5.2 Cast-In-Place Concrete Piers

5.2.1 Downward Loads

Straight-shafted or belled, drilled, cast-in-place concrete piers are recommended for the support of the foundation loads involved. Safe downward capacities of straight-shafted piers extending a minimum of 10.0 feet below the finished grade elevation are presented in Figure 2 for abutment piers. Capacities shown in Figure 2 are based on side shear resistance only. Safe downward capacities of belled piers extending a minimum of 10 feet below finished grade elevation are shown in Figure 3. Capacities shown in Figure 3 were calculated assuming end-bearing only. Capacities apply to full dead plus live loads. A one-third increase is recommended when considering wind or seismic forces.

The methodology and input design parameters utilized in analysis of drilled pier capacity are presented in Appendix C. Complete design calculations are provided.

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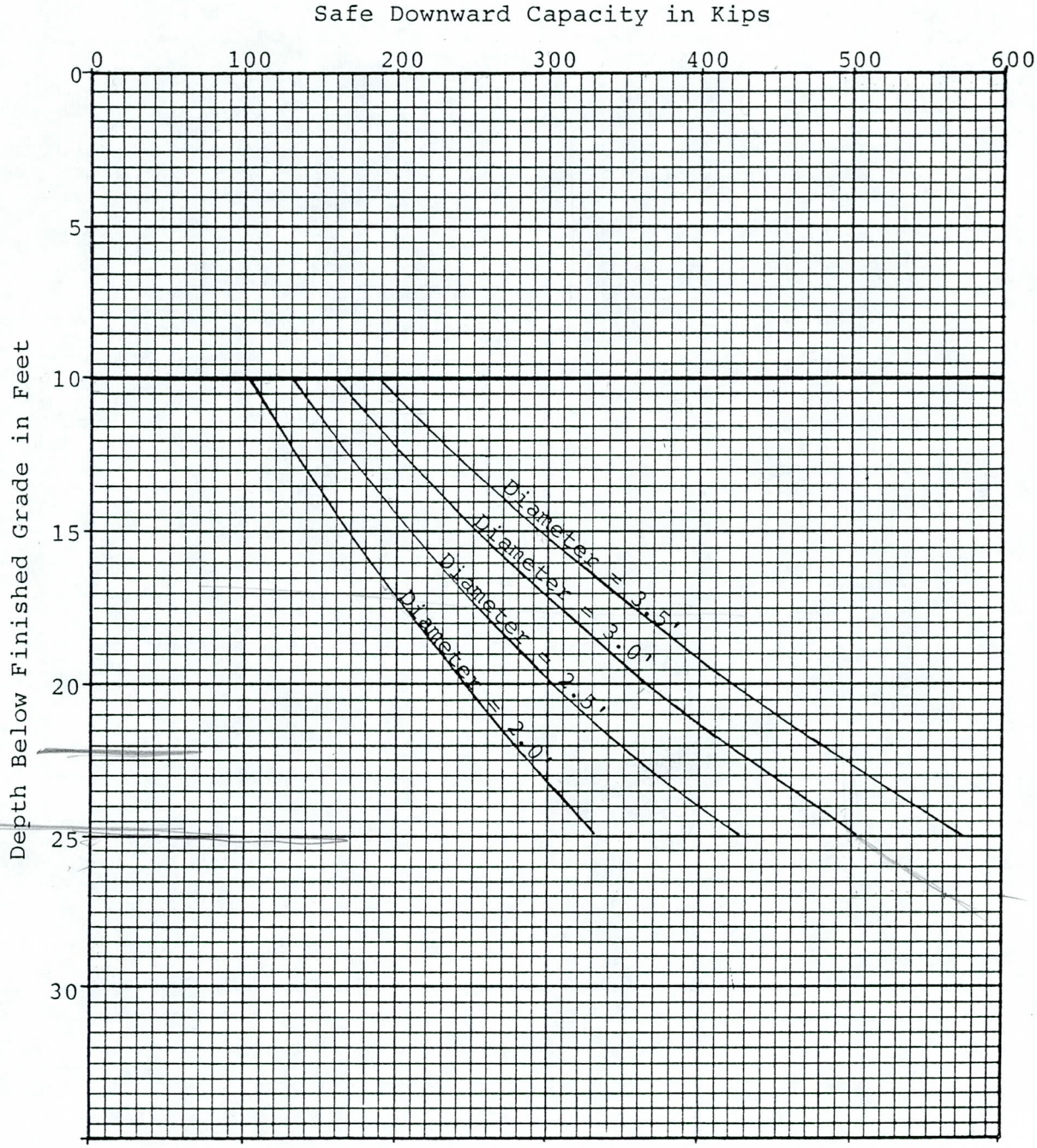
FIGURE 2

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



Side Friction

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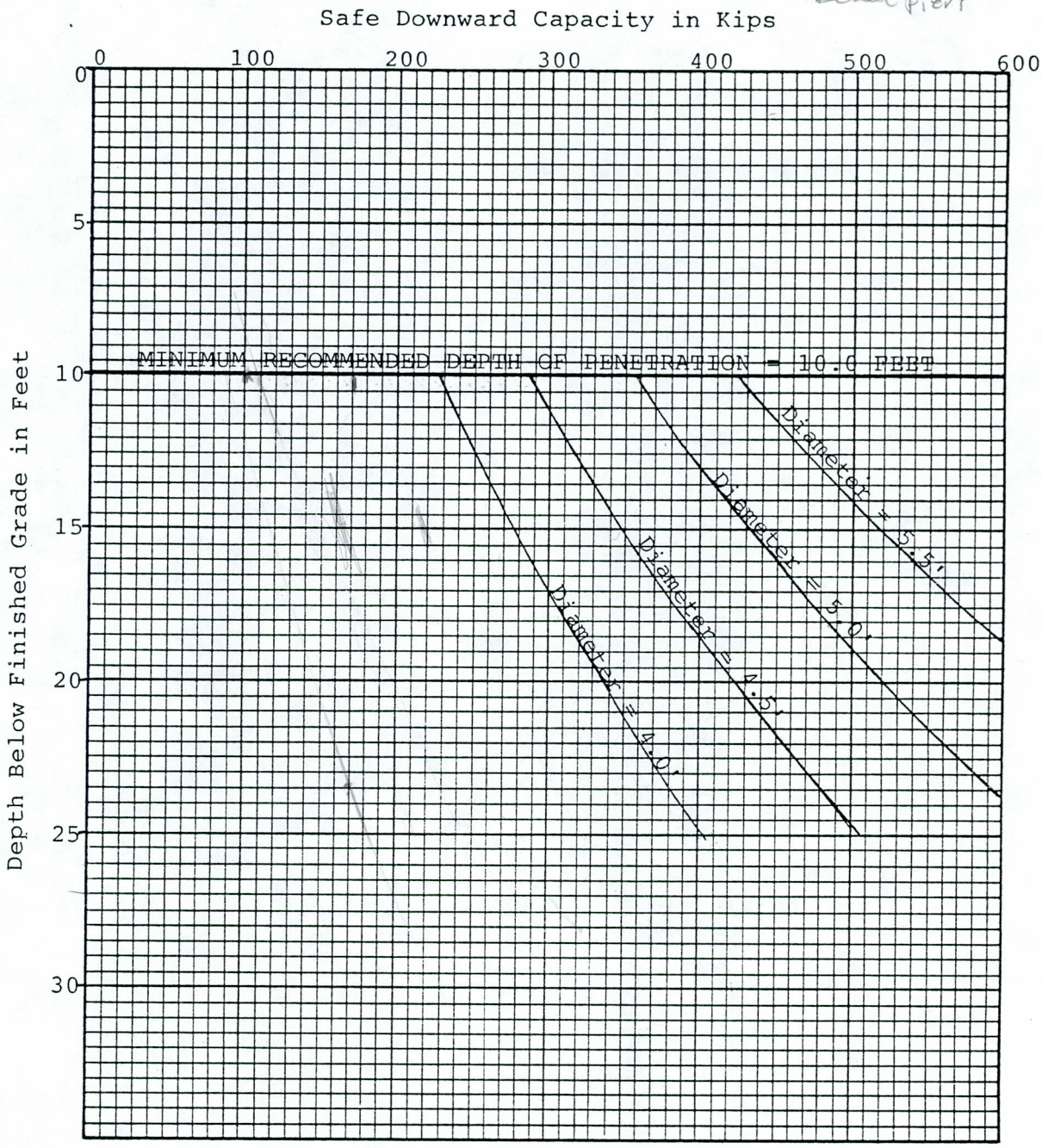
FIGURE 3

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

We are not using Belled piers



End Bearing:

EXA - F.CD
A71 - SPEC. work test.
& Sch.
#8

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$\phi = 30°$ o/c

how about

$c_u = 1000$

5.2.2 Estimated Settlements

Settlements of pier foundations designed and constructed in accordance with criteria presented herein can be estimated using design tables presented in Appendix C. Settlement charts were developed for both the end-bearing and side shear cases using elastic theory. Settlements are presented in terms of inches of settlement per kip of vertical load. Thus, using the charts, the settlement can be quickly estimated for both straight-shafted and belled piers, incorporating the pier diameter and the pier tip elevation.

5.2.3 Resistance to Lateral Loads

The design for lateral loads should be in accordance with procedures detailed by Broms (1965, 1964a, 1964b)*. The soil should be modeled as both cohesive and cohesionless, with the lower allowable lateral load from these procedures to be used for design. Based on our experience with the site soils, conservative strength parameters recommended for use in computing the ultimate lateral resistance are $\phi = 30°$ and $c_u = 1{,}000$ pounds per square foot. The passive earth pressure coefficient for the cohesionless case is 3.0. The in situ unit weight of the soil can be taken as 120 pounds per cubic foot.

*References are listed at end of report.

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---

*References are listed at end of report.

Implementation of Broms' procedures also requires a coefficient of horizontal subgrade reaction, k_h. For the cohesive case, a value of k_hD = 460 pounds per square inch, independent of depth, is recommended. Thus, for a 24-inch diameter pier, k_h = 19 pounds per cubic inch. For the cohesionless case, k_h varies with depth in accordance with the relationship

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

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

The above criteria apply to both straight-shafted and belled piers. For belled piers, the lateral resistance can be conservatively estimated by using the diameter of the shaft for the total length of the pier, thus ignoring the larger diameter of the bell. A more rigorous analysis would include the variation in diameter.

Criteria provided above apply to isolated piers spaced no closer than 3 diameters on center perpendicular to the line of thrust and 6 diameters on center parallel to the line of thrust.

5.2.4 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. Loose material present in the bottom of the holes should be cleaned using the auger or other equipment so that no more than 2 inches of loose material is present after cleaning.

All loose material should be cleaned from the base of drilled-and-belled piers so that undisturbed native soil is exposed throughout. Manual cleaning of belled piers will be necessary for adequate removal of loose disturbed material. This will likely impose requirements of a minimum shaft diameter of 30 inches, to allow access and casing of the shaft.

5.2.5 Placement of Concrete

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

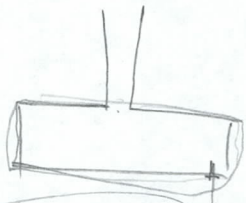
strength standpoint, so that the slump during placement is in the range of 4 to 6 inches.

5.2.6 Inspection & Construction

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

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

5.3 Spread-Type Footings



5.3.1 Downward Loads

As an alternative to drilled piers, spread-type footings will provide safe support of the structure at the abutments. Safe soil bearing pressure for a footing bearing at elevation 1,202, which is approximately 8.0 feet below channel invert finished grade, is 11.3 ksf. Minimum footing depth does not consider scour, as erosion protection is assumed.

5.3.2 Resistance to Lateral Loads

A passive earth pressure of 350 pounds per square foot per foot of depth against footings and walls is recommended for the resistance of lateral loads. A coefficient of friction of 0.45 is recommended for computation of the lateral resistance between the base of foundations and the soil.

5.3.3 Estimated Settlements

Estimated settlements are provided in charts included in Appendix C. Settlements are presented in terms of inches of settlement per kip of vertical load.

5.4 Abutment Wall Design Criteria

Free draining granular backfill should be utilized behind the abutments and in roadway approach fills. This

material should consist of sand and gravel, and have no more than 12 percent passing the no. 200 sieve. This material should be nonplastic when tested in accordance with ASTM D422 and D423. Compaction of the fill should be at least 95 percent of maximum density as determined by ASTM D1557.

The earth pressures against the abutments would depend upon the degree of restraint. Rigid, absolutely restrained abutments would be subjected to earth pressures represented by a hydrostatic load diagram of about 50 pounds per square foot per foot of depth. Rotation or lateral translation of the walls equal to about 0.001 times the height would reduce earth pressures to the active state of about 30 pounds per square foot per foot of depth. Slight lateral translation equal to about 0.0005 times the height would result in an intermediate pressure diagram on the order of 40 pounds per square foot per foot of depth.

REFERENCES

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

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

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

TEST DRILLING EQUIPMENT & PROCEDURES

Drilling Equipment Truck-mounted CME-55 drill rigs powered with 4 or 6 cylinder Ford industrial engines are used in advancing test borings. The 4 cylinder and 6 cylinder engines are capable of delivering about 4,350 and 6,500 foot/pounds torque to the drill spindle, respectively. The spindle is advanced with twin hydraulic rams capable of exerting 12,000 pounds downward force. Drilling through soil or softer rock is performed with 6 1/2 O.D., 3 1/4 I.D. hollow stem auger or 4 1/2 inch continuous flight auger. Carbide insert teeth are normally used on the auger bits so they can often penetrate rock or very strongly cemented soils which require blasting or very heavy equipment for excavation. Where refusal is experienced in auger drilling, the holes are sometimes advanced with tricone gear bits and NX rods using water or air as a drilling fluid. Where auger and tricone gear bits cannot be used to advance the hole due to cobbles or caving conditions, the ODEX (overburden drilling with the eccentric method) is used. A percussion down-the-hole hammer underreams the hole and 5 inch steel casing is introduced into the hole during drilling. The drill bit is eccentric and can be removed from the center of the casing to allow sampling of the material below the bit penetration depth.

Sampling Procedures Dynamically driven tube samples are usually obtained at selected intervals in the borings by the ASTM D1586 procedure. In many cases, 2" O.D., 1 3/8" I.D. samplers are used to obtain the standard penetration resistance. "Undisturbed" samples of firmer soils are often obtained with 3" O.D. samplers lined with 2.42" I.D. brass rings. The driving energy is generally recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the samplers in 6 inch increments. However, in stratified soils, driving resistance is sometimes recorded in 2 or 3 inch increments so that soil changes and the presence of scattered gravel or cemented layers can be readily detected and the realistic penetration values obtained for consideration in design. These values are expressed in blows per foot on the logs. "Undisturbed" sampling of softer soils is sometimes performed with thin walled Shelby tubes (ASTM D1587). Where samples of rock are required, they are obtained by NX diamond core drilling (ASTM D2113). Tube samples are labeled and placed in watertight containers to maintain field moisture contents for testing. When necessary for testing, larger bulk samples are taken from auger cuttings.

Continuous Penetration Tests Continuous penetration tests are performed by driving a 2" O.D. blunt nosed penetrometer adjacent to or in the bottom of borings. The penetrometer is attached to 1 5/8" O.D. drill rods to provide clearance to minimize side friction so that penetration values are as nearly as possible a measure of end resistance. Penetration values are recorded as the number of blows of a 140 pound 30 inch free fall drop hammer required to advance the penetrometer in one foot increments or less.

Boring Records Drilling operations are directed by our field engineer or geologist who examines soil recovery and prepares boring logs. Soils are visually classified in accordance with the Unified Soil Classification System (ASTM D2487) with appropriate group symbols being shown on the logs.

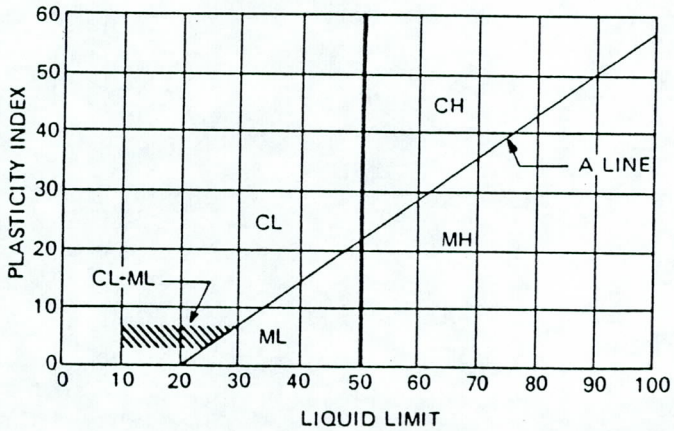
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified by the Unified Soil Classification system on the boring logs presented in this report. Grain-size analysis and Atterberg Limits Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, see "The Unified Soil Classification System" Corp of Engineers, US Army Technical Memorandum No. 3-357 (Revised April 1960) or ASTM Designation: D2487-66T.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 seive)			SW	Well graded sands, gravelly sands.
					SP	Poorly graded sands, gravelly sands.
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS (LIMITS PLOT BELOW "A" LINE & HATCHED ZONE ON PLASTICITY CHART)	SILTS OF LOW PLASTICITY (Liquid Limit Less Than 50)			ML	Inorganic silts, clayey silts with slight plasticity.
		SILTS OF HIGH PLASTICITY (Liquid Limit More Than 50)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
	CLAYS (LIMITS PLOT ABOVE "A" LINE & HATCHED ZONE ON PLASTICITY CHART)	CLAYS OF LOW PLASTICITY (Liquid Limit Less Than 50)			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		CLAYS OF HIGH PLASTICITY (Liquid Limit More Than 50)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity.

NOTE: Coarse grained soils with between 5% & 12% passing the No. 200 sieve and fine grained soils with limits plotting in the hatched zone on the plasticity chart to have double symbol.

PLASTICITY CHART



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Cobbles	Above 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to ¾ in.
Fine gravel	¾ in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

TERMINOLOGY USED TO DESCRIBE THE RELATIVE DENSITY, CONSISTENCY OR FIRMNESS OF SOILS

The terminology used on the boring logs to describe the relative density, consistency or firmness of soils relative to the standard penetration resistance is presented below. The standard penetration resistance (N) in blows per foot is obtained by the ASTM D1586 procedure using 2" O.D., 1 3/8" I.D. samplers.

1. Relative Density. Terms for description of relative density of cohesionless, uncemented sands and sand-gravel mixtures.

N	Relative Density
0-4	Very loose
5-10	Loose
11-30	Medium dense
31-50	Dense
50+	Very dense

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

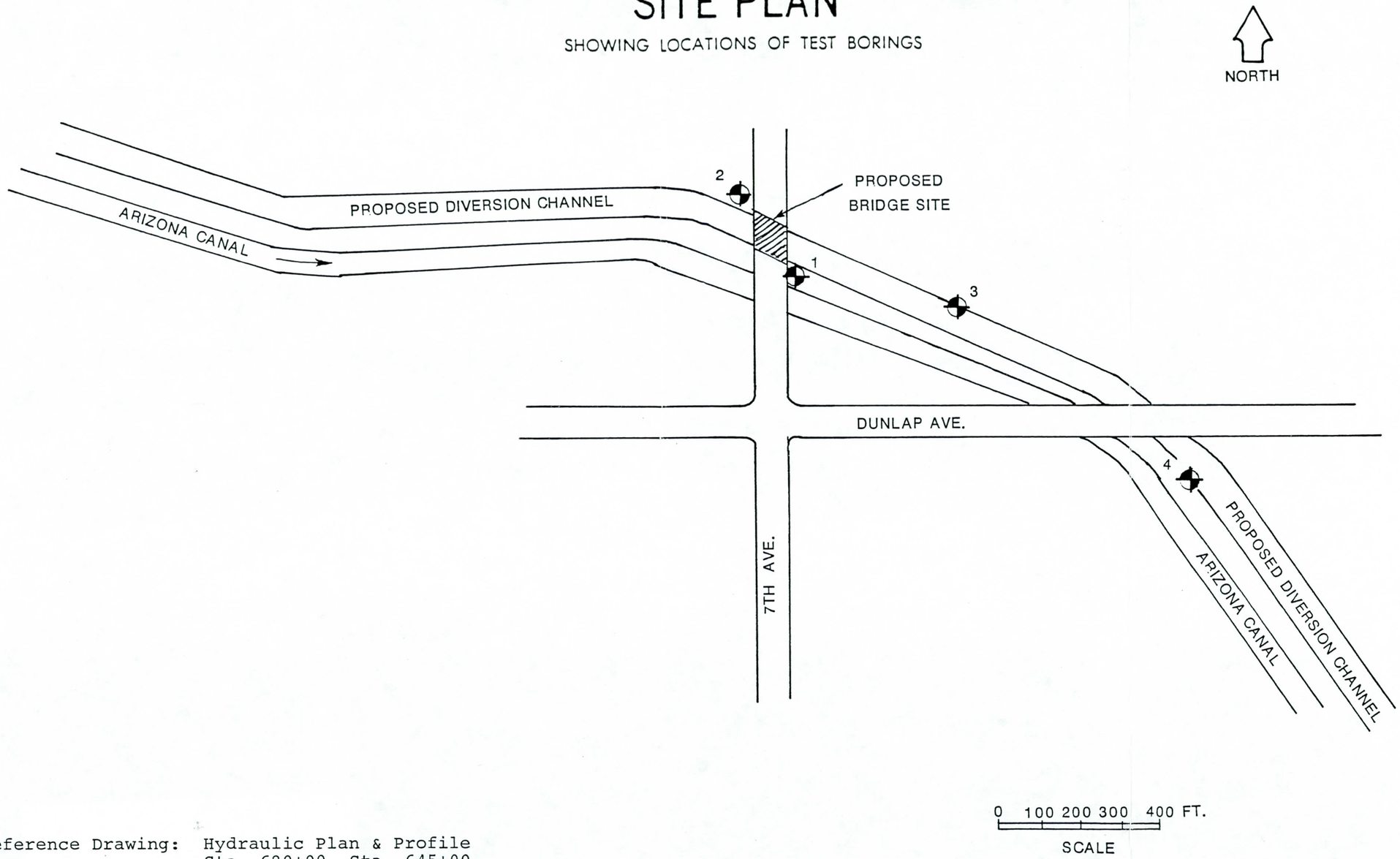
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.
16-30	Very stiff	Readily indented with thumbnail.
30+	Hard	Indented only with difficulty by thumbnail.

3. Relative Firmness. Terms for description of partially saturated and/or cemented soils which commonly occur in the Southwest including clays, cemented granular materials, silts and silty and clayey granular soils.

N	Relative Firmness
0-4	Very soft
5-8	Soft
9-15	Moderately firm
16-30	Firm
31-50	Very firm
50+	Hard

SITE PLAN

SHOWING LOCATIONS OF TEST BORINGS



Reference Drawing: Hydraulic Plan & Profile
Sta. 680+00, Sta. 645+00
U.S. Army Corps of Engineers
Los, Angeles District

ACDC Canal Bridge
7th Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E87-117

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1183

$$\begin{array}{r} 1234.63 \\ -\ \ 41.00 \\ \hline 1193.63 \end{array}$$

this is only for
40' blow. ou cye is 51

PROJECT ACDC Canal Bridge

JOB NO. E87-117 DATE 5-4-87

LOG OF TEST BORING NO. 1

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1234.63'

DATUM City of Phoenix

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	28			SM	slightly moist very firm	FILL SILTY SAND, some fine grained gravel, poorly graded, weakly lime cemented, low plasticity, light brown note: traces of concrete note: filled hole with H_2O prior to sampling
5			☒	S	32		14	CH		
10			☒	S	44					
15			☒	S	42		14		slightly moist very firm	SANDY CLAY, some silt, fine to medium grained, weakly lime cemented, medium to high plasticity, light brown
20			☒	S	45			SC	slightly moist to moist very firm to hard	CLAYEY SAND, some silt, trace of fine grained gravel, predominantly fine to medium grained, weakly lime cemented, low to medium plasticity, light brown note: interbedded with CL lenses below 15'
25			☒	S	53					
30			☒	S	57		6	SM-SC	slightly moist hard	SILTY SAND TO CLAYEY SAND, considerable fine grained gravel, poorly graded, angular, weakly lime cemented, low plasticity, light brown
35			☒	S	91			CL	slightly moist to moist hard	SILTY CLAY, some to considerable fine grained sand, weakly lime cemented, low plasticity, light brown note: grading to SC below 40'
40			☒	S	77					
45										Stopped auger at 39'6" Stopped sampler at 41'

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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CONSULTING GEOTECHNICAL ENGINEERS

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PROJECT ACDC Canal Bridge

JOB NO. E87-117 DATE 5-4-87

LOG OF TEST BORING NO. 2

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1233.39'

DATUM City of Phoenix

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	13			ML	slightly moist moderately firm	SANDY SILT, trace to some clay, fine grained, weakly lime cemented, low plasticity, light brown
5			☒	S	26			CL	slightly moist moderately firm	SANDY CLAY, considerable silt, predominantly fine to medium grained, weakly lime cemented, low plasticity, light brown
10			☒	S	13		10			
15			☒	U	54			SC-SM	slightly moist very firm	CLAYEY SAND, considerable silt, gap graded, weakly lime cemented, low plasticity, light brown
20			☒	S	38		11	CL	slightly moist very firm	SILTY CLAY, trace of fine grained sand, weakly lime cemented, low plasticity, light brown
25			☒	U	73			SC	slightly moist hard	CLAYEY SAND, some silt & fine grained gravel, gap graded, weakly lime cemented, low plasticity, light brown
30			☒	S	50				slightly moist hard	SANDY CLAY, some to considerable silt, fine to medium grained, weakly lime cemented, low to medium plasticity, light brown
35			☒	S	55			CL		
40			☒	S	74					
45										Stopped auger at 39'6" Stopped sampler at 41'

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 ACDC Canal Bridge

JOB NO. E87-117 DATE 5-4-87

LOG OF TEST BORING NO. 3

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1232.75'

DATUM City of Phoenix

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			SM	slightly moist	SILTY SAND, fine grained, weakly lime cemented, low plasticity to nonplastic, light brown
5			☒	S	33				moderately firm	
								CL	slightly moist	SANDY CLAY, some to considerable silt, fine to medium grained, weakly lime cemented, low plasticity, light brown
10			☒	S	75		5		very firm to hard	
								CL	slightly moist	SILTY CLAY, trace of fine grained sand, weakly lime cemented, low to medium plasticity, light brown
15			☒	S	75				hard	
20										Stopped auger at 14'6" Stopped sampler at 16'

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 ACDC Canal Bridge

JOB NO. E87-117 DATE 5-4-87

LOG OF TEST BORING NO. 4

RIG TYPE CME-55

BORING TYPE 6½" Hollow Stem Auger

SURFACE ELEV. 1235.64'

DATUM City of Phoenix

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	12			CL-ML	slightly moist to moist moderately firm	SILTY CLAY, trace of fine grained sand, weakly lime cemented, low plasticity, brown
5			X	S	15		12			
10			X	S	26			SC	slightly moist moderately firm to firm	CLAYEY SAND, some silt, trace of fine grained gravel, predominantly fine to medium grained, weakly lime cemented, medium plasticity, brown to light brown
15			X	S	17		19	CL	moist firm to very firm	SILTY CLAY, trace of fine grained sand, weakly lime cemented, low plasticity, light brown note: considerable poorly graded sand in lenses
20			X	S	36					
25			X	S	56			CL	slightly moist to moist very firm to hard	SANDY CLAY, some silt, trace of fine grained gravel, poorly graded, weakly lime cemented, low plasticity, light brown note: considerable sand below 31', probably grades to SC
30			X	S	32		17			
35			X	S	63					
40										Stopped auger at 34'6" Stopped sampler at 36'

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.

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TABULATION OF TEST RESULTS

Job No. E87-117
W/O 1

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	4.5'-6'	CH	54	31	62	72	77	79	82	88	92	94	97	98	100		7-117-2
#1	14.5'-16'	SC	33	11	41	51	56	60	64	72	81	86	94	97	100		7-117-4
#1	29.5'-31'	SM-SC	15	6	34	44	50	55	59	69	77	82	91	94	98	100	7-117-7
#2	9.5'-11'	CL	33	17	61	69	73	76	79	87	92	94	99	99	100		7-117-12
#2	19.5'-21'	CL	28	16	58	71	78	80	83	89	93	95	97	99	100		7-117-14
#3	9.5'-11'	CL	27	16	55	63	68	71	73	81	86	89	95	95	97	100	7-117-21
#4	4.5'-6'	SC	38	25	45	59	68	72	76	83	89	91	95	97	100		7-117-24
#4	14.5'-16'	CL	30	13	59	71	82	81	84	91	94	95	98	99	100		7-117-26
#4	29.5-31	CL	28	9	69	79	84	87	89	93	96	97	100				7-117-29

SERGENT, HAUSKINS, & BECKWITH

GEOTECHNICAL ENGINEERS

PROJECT: ACDC CANAL BRIDGE
SAMPLE: #2 @ 24.5'-25.5'

JOB # E87-117
DATE 5/12/87
W.O. # 1
LAB # 15

CONSOLIDATION TEST

INSITU MOIST. CONTENT	8.8%
SAT. MOIST. CONTENT	14.7%
MOISTURE PICK-UP	5.9%
DRY DENSITY	121.2 pcf

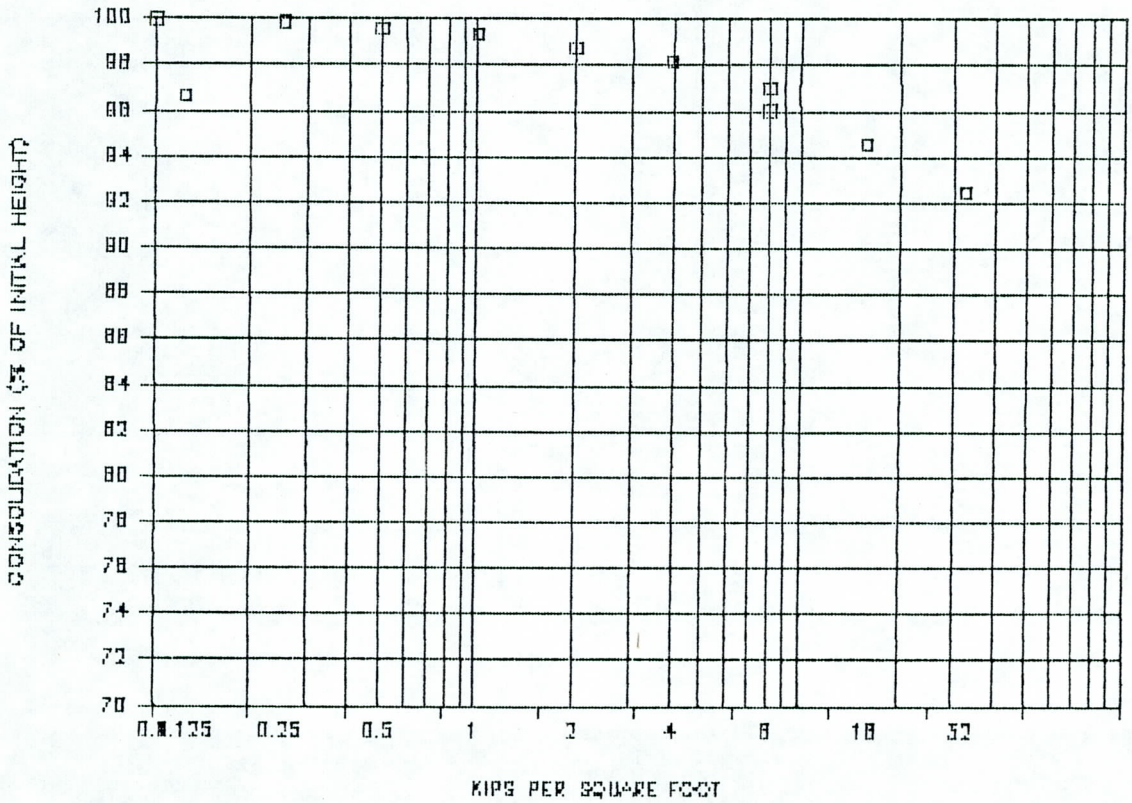
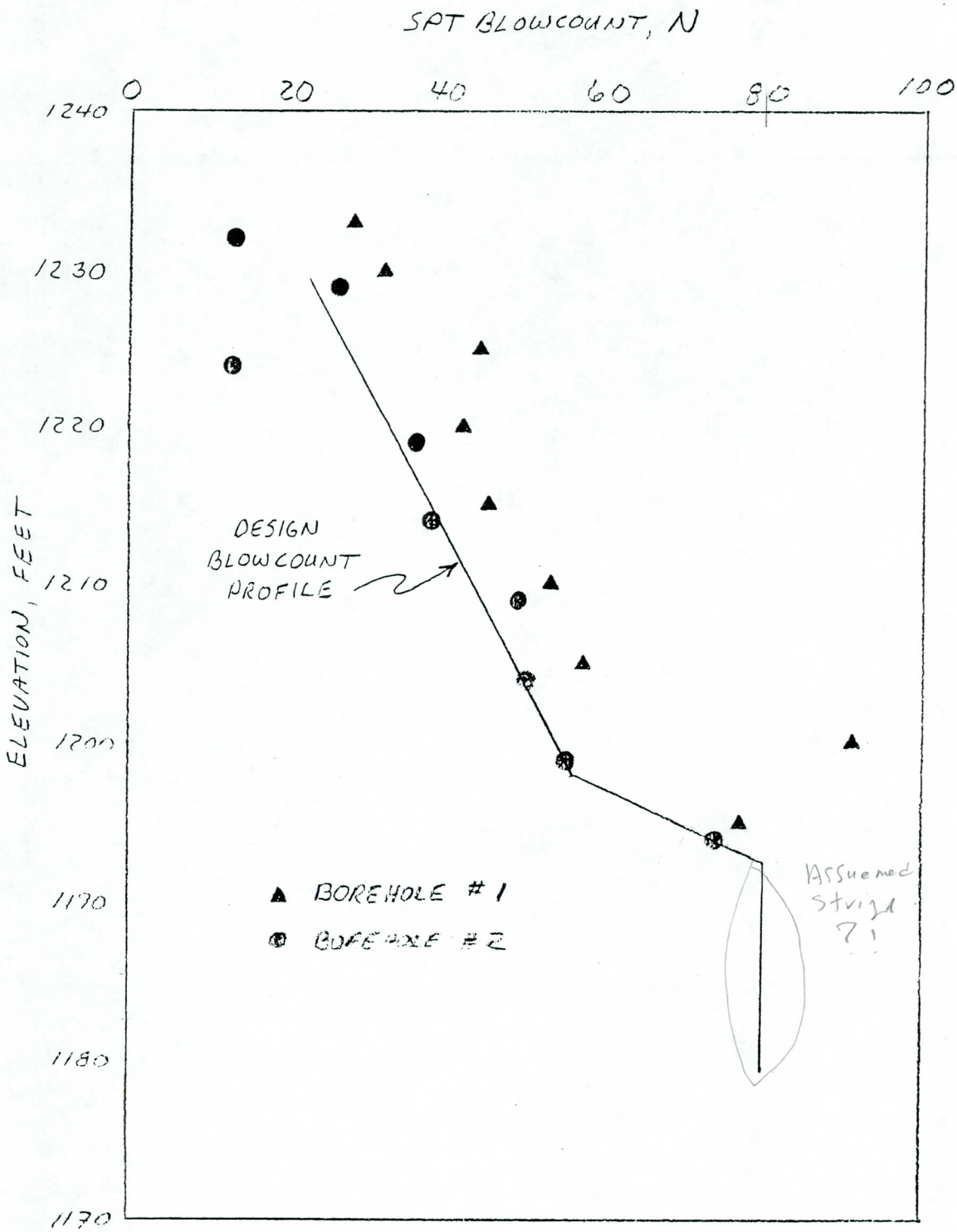


FIGURE B-1



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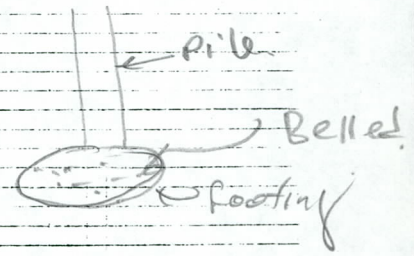
Project ACDC CANAL BRIDGE @ 7th AVE
Job No: E87-117
Computed by: GAJS Ckd. by: NHW
Date 5/12/87 Page 1 of 1

We are not using Belled Piers

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

SAFE DOWNWARD CAPACITY



CONSIDERING THE FUNCTION OF A BELL - USE END BEARING ONLY

RECALL FROM STRAIGHT PIER ANALYSIS:

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

WHERE d = DIAMETER OF THE BELL, FEET

TABLE C-4

PIER TIP ELEVATION FEET	Q_E KSF	CAPACITY, P_D, KIPS $d = 4.0$ FEET	$d = 4.5$ FEET	$d = 5.0$ FEET	$d = 5.5$ FEET
1200	54	226.2	286.3	353.4	427.7
1195	69	289.0	365.8	451.6	546.4
1190	80	335.1	424.1	523.6	633.6

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

A. CONSIDER END-BEARING ONLY

$$\delta = \mu_0 \mu_1 \left(q d / E_s \right)$$

WHERE: δ = SETTLEMENT

q = AVERAGE BEARING PRESSURE $= Q/(\pi d^2/4)$

d = PIER DIAMETER

E_s = SOIL MODULUS

μ_0, μ_1 = DIMENSIONLESS PARAMETERS DEPENDENT ON DEPTH

THEREFORE: $\delta = \mu_0 \mu_1 \left(4Q / \pi d E_s \right)$

WHERE Q IS THE APPLIED AXIAL LOAD
FOR A CIRCULAR FOUNDATION, $\mu_1 = 0.62$.
μ_0 IS DEPENDENT ON THE RATIO D/d
WHERE D = DEPTH TO FOOTING BELOW FINISHED GRADE.

FIGURE C-2 SHOWS VARIATION OF μ_0 WITH DEPTH

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

THE FINAL WORKING RELATIONSHIP FOR SETTLEMENT CALCULATION IS:

$$\delta = 0.62 \mu_0 \left(4/\pi \right) \left(Q / d E_s \right)$$

$$= 0.789 \mu_0 Q / d E_s$$

Settlement Calculations:

a) From Table C.5 (End Bearing)

Tip Ele = 1190, P = 146.6 K

δ = Settlement = (0.0009) 146.6

= 0.132

b) From Table C.6 (Side Shear)

δ = Settlement = (0.0001) 146.6 K

= 0.015"

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

$$\delta = 9.47 \mu_o Q / d E_s$$

TABLE C-5 PRESENTS ESTIMATED SETTLEMENTS FOR ABUTMENT PIERS. SETTLEMENTS ARE PRESENTED AS INCHES/KIP VERTICAL LOAD

EXAMPLE:

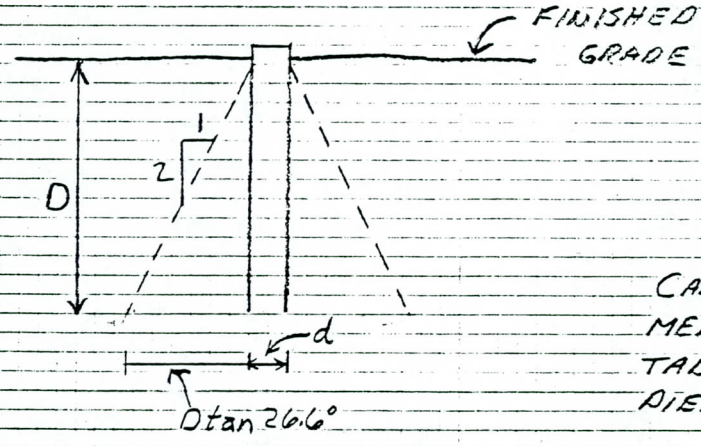
PIER TIP ELEVATION = 1200
PIER DIAMETER = 2.0'
VERTICAL LOAD = 200

$$\delta = (0.0021)(200) = 0.42 \text{ INCHES}$$

TABLE C-5 CAN BE USED FOR EITHER STRAIGHT SHAFTED OR BELLED PIERS SINCE ONLY END-BEARING IS CONSIDERED

B. CONSIDER SIDE SHEAR ONLY

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



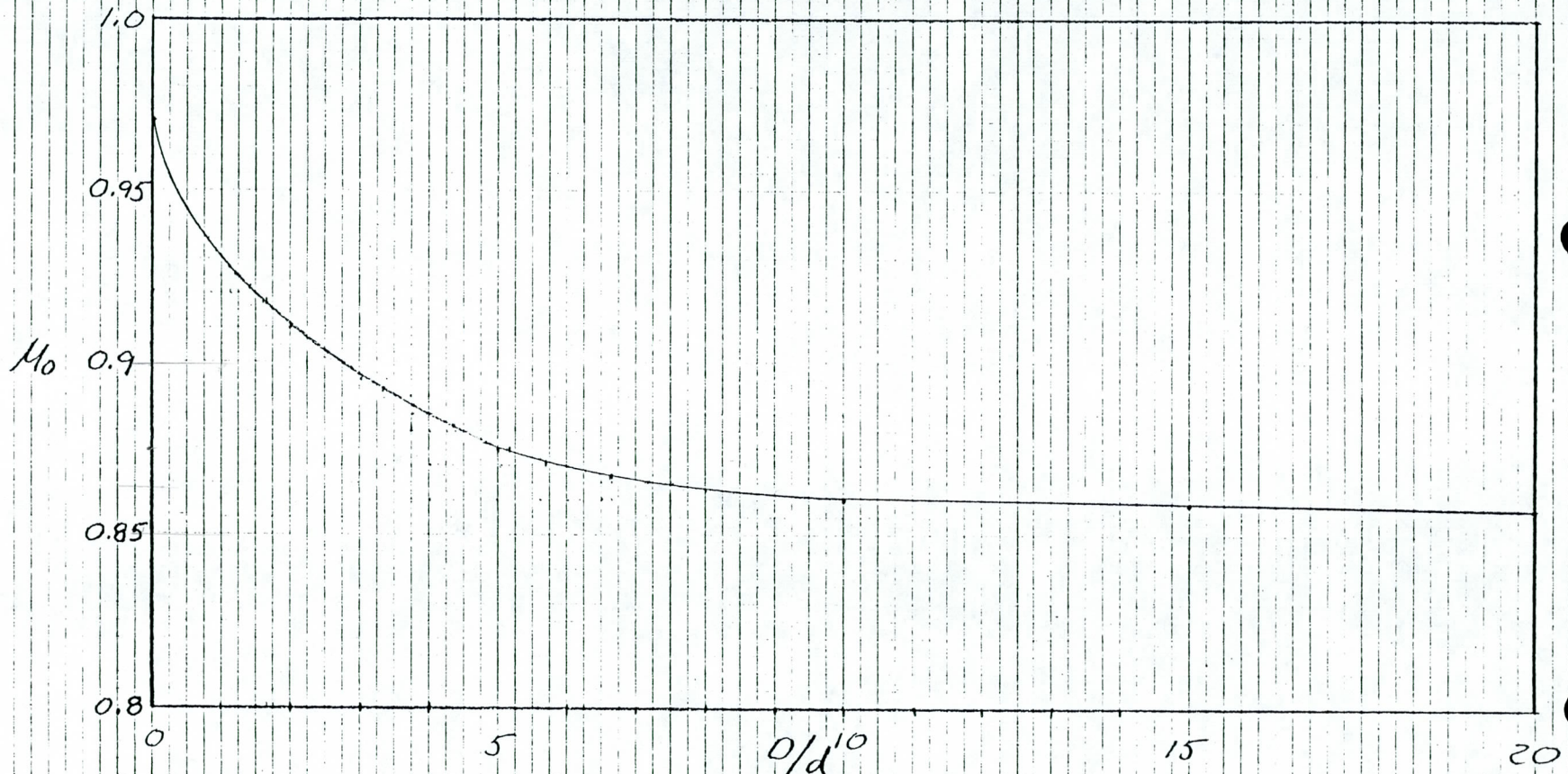
CALCULATIONS OF SETTLEMENT IS AS PREVIOUS EXAMPLE TABLE C-6 IS FOR ABUTMENT PIERS.

$$d^* = 2D \tan 26.6° + d$$

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

FIGURE C-2

VARIATION OF μ_o WITH D/d



D = DEPTH TO FOOTING BELOW FINISHED GRADE

d = PIER DIAMETER

TABLE C-5

NORMALIZED SETTLEMENT IN INCHES/KIP VERTICAL LOAD
FOR ABUTMENT PIERS (FINISHED GRADE 1210 ±)
ASSUMING END BEARING ONLY

PIER TIP ELEVATION FEET	E_s KSF	SETTLEMENT, IN/KIP d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET	d = 4.0 FEET	d = 4.5 FEET	d = 5.0 FEET
1200	2000	0.0021	0.0017	0.0014	0.0012	0.0011	0.0010	0.0009
1195	2800	0.0015	0.0012	0.0010	0.0009	0.0008	0.0007	0.0006
1190	3200	0.0013	0.0010	0.0009	0.0007	0.0006	0.0006	0.0005

TABLE C-6

NORMALIZED SETTLEMENT IN INCHES/KIP VERTICAL LOAD FOR ABUTMENT PIERS (FINISHED GRADE 1210±) ASSUMING SIDE SHEAR ONLY

PIER TIP ELEVATION FEET	DEPTH FEET	E_s KSF	SETTLEMENT, IN/KIP d = 2.0' FEET	d = 2.5' FEET	d = 3.0' FEET	d = 3.5' FEET
1202	8	1800	0.0005	0.0004	0.0004	0.0004
1197	13	2600	0.0002	0.0002	0.0002	0.0002
1192	18	3300	0.0001	0.0001	0.0001	0.0001
1187	23	3300	0.0001	0.0001	0.0001	0.0001

IV SAFE CAPACITY OF ALTERNATIVE SPREAD-TYPE FOOTING FOR ABUTMENTS.

ASSUME $\phi = 30°$ $\gamma_T = 120$ PCF
$C = 1{,}000$ PSF

GENERAL BEARING CAPACITY RELATIONSHIP

$q_{ULT} = CN_c + \gamma D$ D = EMBEDDED DEPTH = 8.0 FEET (ELEVATION 1202±)

FROM NAVFAC DM-7.2, 1982
FIGURE 1, P. 7.2-131 $N_c = 33$

$q_{ULT} = (1)(33) + (0.120)(8)$
$= 34$ KSF

ALLOWABLE BEARING PRESSURE
USE FACTOR OF SAFETY = 3.0

$q_{ALL} = 34/3 = \underline{\underline{11.3 \text{ KSF}}}$

this was assumed shallow foundation.

IV SAFE CAPACITY OF ALTERNATIVE SPREAD-TYPE FOOTING FOR ABUTMENTS.

ASSUME $\phi = 30°$ $\gamma_T = 120$ PCF

$C = 1{,}000$ PSF

GENERAL BEARING CAPACITY RELATIONSHIP

$q_{ULT} = CN_c + \gamma D$ D = EMBEDDED DEPTH = 8.0 FEET (ELEVATION 1202±)

FROM NAVFAC DM-7.2, 1982 FIGURE 1, P. 7.2-131 $N_c = 33$

$q_{ULT} = (1)(33) + (0.120)(8)$

$= 34$ KSF

ALLOWABLE BEARING PRESSURE USE FACTOR OF SAFETY = 3.0

$q_{ALL} = 34/3 = \underline{\underline{11.3 \text{ KSF}}}$

DESIGN OF DRILLED, CAST IN PLACE CONCRETE PIERS

I. SAFE DOWNWARD CAPACITY

A. CONSIDER END BEARING ONLY

$P_0 = Q_E A_E / F.S.$ = DESIGN CAPACITY, KIPS

Q_E = ULTIMATE END BEARING

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

WHERE d = PIER DIAMETER, FEET

F.S. = FACTOR OF SAFETY = 3.0

$$P_0 = 0.262\, Q_E\, d^2$$

B. CONSIDER SIDE SHEAR ON

$$P_0 = \sum_{i=1}^{n} Q_{si} A_{si} L_i / F.S$$

Q_{si} = ULTIMATE

A_{si} = UNIT S

L_i = SEGME

F.S. = 3.0

$$P_0 = 1.05\, d \sum_{i=1}^{n} Q_{si} L_i$$

4/4.
Is that Allowable load. Re. bond IF do D = 3 ?
Pile.
How about D = 3?

NOTE: FINAL DESIGN RECOMMENDATIONS USE SAFE DOWNWARD CAPACITIES CONSIDERING SIDE SHEAR ONLY, WHICH RESULTS IN MUCH GREATER CAPACITIES THAN CONSIDERING END BEARING ALONE.

DESIGN OF DRILLED, CAST IN PLACE CONCRETE PIERS

I. SAFE DOWNWARD CAPACITY

A. CONSIDER END BEARING ONLY

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

Q_E = ULTIMATE END BEARING

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

WHERE d = PIER DIAMETER, FEET

F.S. = FACTOR OF SAFETY = 3.0

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

B. CONSIDER SIDE SHEAR ONLY

$P_D = \sum_{i=1}^{n} Q_{si} A_{si} L_i / F.S.$ = DESIGN CAPACITY, KIPS

Q_{si} = ULTIMATE SIDE SHEAR

A_{si} = UNIT SIDE AREA = πd

WHERE: d = PIER DIAMETER, FEET

L_i = SEGMENT LENGTH, FEET

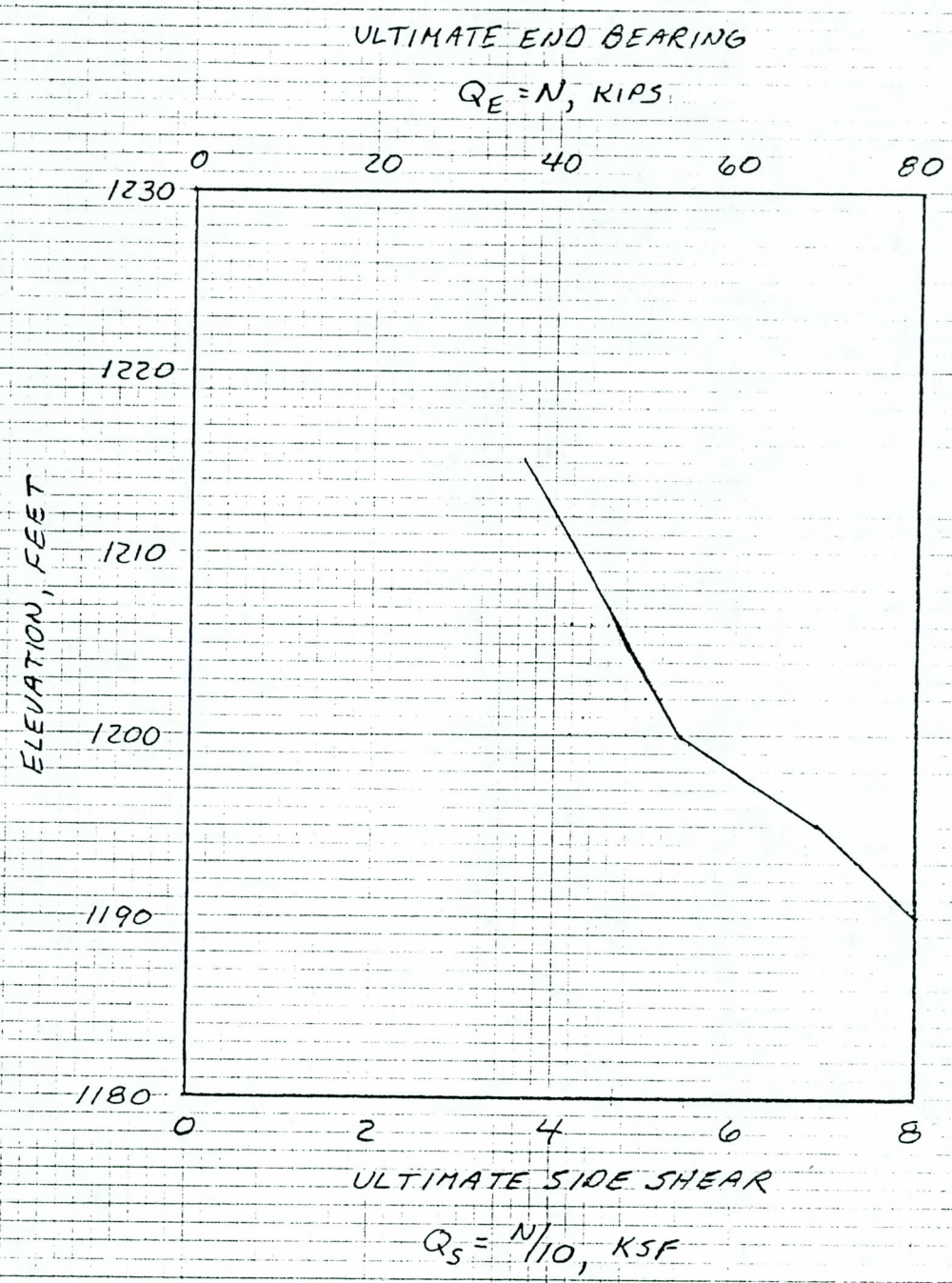
F.S. = 3.0

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

NOTE: FINAL DESIGN RECOMMENDATIONS USE SAFE DOWNWARD CAPACITIES CONSIDERING SIDE SHEAR ONLY, WHICH RESULTS IN MUCH GREATER CAPACITIES THAN CONSIDERING END BEARING ALONE.

FIGURE C-1

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



$$P_0 = 0.262\, Q_E\, d^2$$

$$= 0.262\,(80)\,(3^2) = 188.5^K$$

check

In ksf

$$P_0 = 1.05\, d \sum_{i=1}^{n} Q_{si} L_i$$

$d = 3.0$, $L_i = 5.0$, $Q_{si} = 7.80$

$$P_0 = (1.05)(3.0)(7.8)\,5.$$

$$= 122.9^K$$

TABLE C-1

DESIGN VERTICAL DOWNWARD CAPACITY
OF DRILLED PIERS BASED ON END BEARING

PIER TIP ELEVATION FEET	Q_E	CAPACITY, P_D, KIPS d=2.0 FT	d=2.5 FT	d=3.0 FT	d=3.5 FT
1200	54	56.6	88.4	127.2	173.2
1195	69	72.3	112.9	162.6	221.3
1190	80	83.8	130.9	188.5	256.6

PC1

1183 - 1183

$\frac{188.5}{A = 7.0} = 27$ KSF bearing

SAFE: Allowable.

TABLE C-2

INCREMENTAL VERTICAL DOWNWA
CAPACITY OF DRILLED PIERS
BASED ON SIDE SHEAR

ELEVATION INCREMENT FEET	Q_{si} KSF	L_i FT	INCREMEN d=2.0 FT	d= F		
1207-1202	4.90	5	51.5	64.3	77.2	90.0
1202-1197	5.68	5	59.6	74.6	89.5	104.4
1197-1192	6.95	5	73.0	91.2	109.5	127.7
1192-1187	7.80	5	81.9	102.4	122.9	143.3

399 K

TABLE C-1

DESIGN VERTICAL DOWNWARD CAPACITY OF DRILLED PIERS BASED ON END BEARING

PIER TIP ELEVATION FEET	Q_E	CAPACITY, P_D, KIPS d=2.0 FT	d=2.5 FT	d=3.0 FT	d=3.5 FT
1200	54	56.6	88.4	127.2	173.2
1195	69	72.3	112.9	162.6	221.3
1190	80	83.8	130.9	188.5	256.6

PCL

TABLE C-2

INCREMENTAL VERTICAL DOWNWARD CAPACITY OF DRILLED PIERS BASED ON SIDE SHEAR

ELEVATION INCREMENT FEET	Q_{si} KSF	L_i FT	INCREMENTAL CAPACITY P_{Di} KIPS d=2.0 FT	d=2.5 FT	d=3.0 FT	d=3.5 FT
1207-1202	4.90	5	51.5	64.3	77.2	90.0
1202-1197	5.68	5	59.6	74.6	89.5	104.4
1197-1192	6.95	5	73.0	91.2	109.5	127.7
1192-1187	7.80	5	81.9	102.4	122.9	143.3

399K

TABLE C-3

End bearing cap or skin friction

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

Skin friction

PIER TIP ELEVATION FEET	EMBEDDED LENGTH FEET	CAPACITY P_o, KIPS d = 2.0 FEET	d = 2.5 FEET	d = 3.0 FEET	d = 3.5 FEET
1202	8	82.3	102.9	123.5	144.1
1197	13	141.9	177.5	213.0	248.5
1192	18	214.9	268.7	322.5	376.2
1187	23	296.8	371.1	445.4	519.5

☆ NOTE: DESIGN VALUES OBTAINED BY SUMMING INCREMENTAL CAPACITIES (TABLE C-2) TO EACH DEPTH OR ELEVATION

V SETTLEMENT ANALYSIS
OF CONTINUOUS SPREAD-TYPE
FOOTINGS FOR ABUTMENTS

THE CHRISTIAN, J.T. & CARRIL
SETTLEMENT ANALYSIS USED FOR
DRILLED PIERS CAN BE APPLIE

Our CASE is Continous Spread

Our CASE spreated footing

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

BUT FOR A CONTINUOUS FOO

δ = SETTLEMENT
μ_1 = 1.7 FOR L/B > 10
q = AVERAGE BEARING
B = FOOTING WIDTH
E_s = SOIL MODULUS
μ_0 = PARAMETER, AS SHOWN IN FIGURE C-2

ASSUMING PROPOSED EMBEDMENT DEPTH OF 8 FEET

$$\delta = 0.255 \mu_0 Q/E_s \text{, INCHES}$$

FOOTING WIDTH FEET	SETTLEMENT IN/KIP
4	.0001
5	.0001
8	.0001
10	.0001

* E_s = 1,800 KSF AT ELEVATION 1202±

NOTE: SETTLEMENTS WILL VARY FOR DIFFERENT WIDTHS OF FOOTINGS BUT THE CHANGE IS INSIGNIFICANT FOR THE SOIL CONDITIONS ENCOUNTERED

V SETTLEMENT ANALYSIS OF CONTINUOUS SPREAD-TYPE FOOTINGS FOR ABUTMENTS

THE CHRISTIAN, J.T. & CARRIER, W.D. SETTLEMENT ANALYSIS USED FOR THE DRILLED PIERS CAN BE APPLIED

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

BUT FOR A CONTINUOUS FOOTING:

δ = SETTLEMENT
$\mu_1 = 1.7$ FOR $L/B > 10$
q = AVERAGE BEARING PRESSURE = Q/LB WHERE L = FOOTING LENGTH
B = FOOTING WIDTH
E_s = SOIL MODULUS
μ_0 = PARAMETER, AS SHOWN IN FIGURE C-2

ASSUMING PROPOSED EMBEDMENT DEPTH OF 8 FEET

$$\delta = 0.255 \mu_0 Q/E_s \text{, INCHES}$$

FOOTING WIDTH FEET	SETTLEMENT IN/KIP	
4	.0001	
5	.0001	* E_s = 1,800 KSF
8	.0001	AT ELEVATION
10	.0001	1202±

NOTE: SETTLEMENTS WILL VARY FOR DIFFERENT WIDTHS OF FOOTINGS BUT THE CHANGE IS INSIGNIFICANT FOR THE SOIL CONDITIONS ENCOUNTERED

SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

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June 18, 1987

HNTB
Anchor Centre II
2207 East Camelback Road
Suite 400
Phoenix, Arizona 85016

SHB Job No. E87-117
Addendum No. 1

Attention: Gray C. Wangelin, P.E.

Re: ACDC Canal Bridge
7th Avenue & Arizona Canal
Phoenix, Arizona

Gentlemen:

This addendum presents an evaluation of the proposed sewer realignment along the north side of the Arizona Canal Diversion Channel between 7th Avenue and Dunlap Avenue.

Details of the proposed construction were provided by Gray C. Wangelin, P.E. of HNTB. It is understood that a 12 to 18-inch sanitary sewer line will be relocated parallel to the north side of the diversion channel. The sewer line will be relocated prior to construction of the diversion channel. The purpose of this letter is to determine the minimum horizontal distance that the sewer needs to be set

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PHOENIX (602) 272-6848 | TUCSON (602) 792-2779 | ALBUQUERQUE (505) 884-0950 | SANTA FE (505) 471-7836 | SALT LAKE CITY (801) 266-0720 | EL PASO (915) 778-3369

back from the proposed diversion channel so as not to be adversely affected by the proposed construction.

It is our opinion that the diversion channel's temporary slopes will be stable at 3/4:1 (horizontal to vertical). Parameters assumed for stability analysis are as follows:

γ = 120 pcf
c = 300 pcf
ϕ = 30°
H = 20 feet

Using simplified slope stability and assuming a circular failure surface exiting at the toe of the excavation, a factor of safety of 1.44 was determined for the 20-foot slope under dry conditions.

At 7th Avenue, the approximate elevation of the sewer line will be 1,225 feet, which is about 5 feet below existing site grade. Figure 1 is a schematic of the sewer line and diversion channel. From this figure, it was determined that a minimum horizontal set back of 14 feet is required in order not to affect the stability of the sewer line during construction of the diversion channel.

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

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

Respectfully submitted,
Sergent, Hauskins & Beckwith Engineers

By _____
Gary N. Sheppard

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13872
LAWRENCE A.
HANSEN
Date Signed 6/18/87
ARIZONA, U.S.A.

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

Copies: Addressee (3)

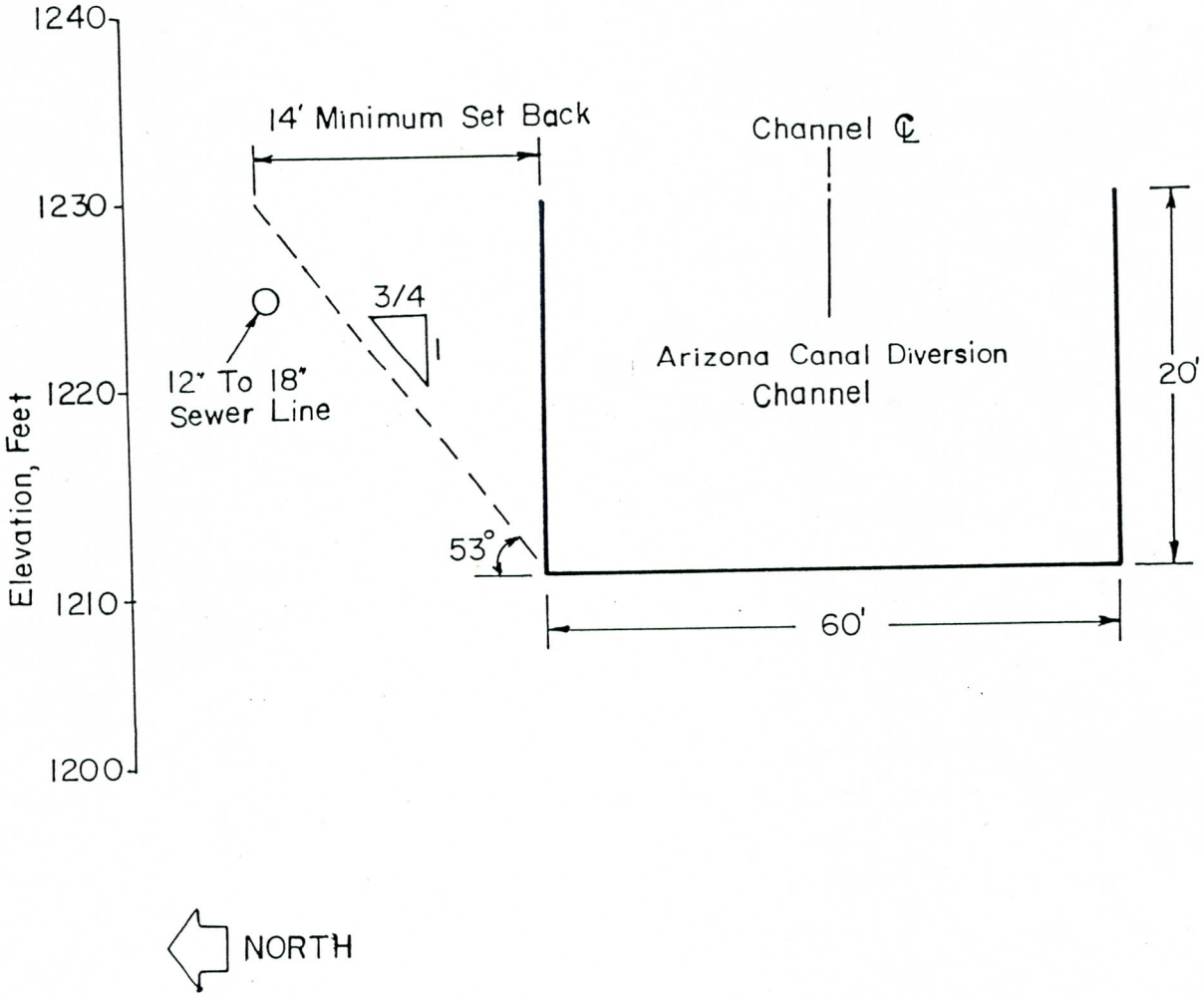

FIGURE 1

ACDC Canal Bridge
7th Avenue & Arizona Canal
Phoenix, Arizona
SHB Job No. E87-117
Addendum No. 1

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