SERGENT, HAUSKINS & BECKWITH

This is the Revised Report 10-31-88

GEOTECHNICAL INVESTIGATION REPORT

Bridge Over Skunk Creek at 83rd Avenue

Peoria, Arizona

Registered Professional Engineer CERTIFICATE NO. 12176 NORMAN HAROLD WETZ Date Signed 10/17/88 ARIZONA, U.S.A.

SHB Job No. E88-9
Report No. 1
Revision No. 1

Consulting Geotechnical Engineers

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SERGENT, HAUSKINS & BECKWITH CONSULTING GEOTECHNICAL ENGINEERS

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October 17, 1988

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One Gateway Center
426 North 44th Street, Suite 252
Phoenix, Arizona 85008

SHB Job No. E88-9
Report No. 1
Revision No. 1

Attention: Donald R. Davis, P.E.
President

Re: Bridge Over Skunk Creek
at 83rd Avenue
Peoria, Arizona

Gentlemen:

Our revised Geotechnical Investigation Report for the Bridge Structure planned for the referenced project is herewith submitted. The report includes the results of the exploratory drilling and laboratory analysis, and recommended criteria for foundation design. Additional calculations are provided in this revised report.

Our Geotechnical Investigation Report addressing drop structure foundations, channel bank stabilization and other earthwork elements of the project will be submitted at a later date.

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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

Registered Professional Engineer (Civil)
CERTIFICATE NO. 12176
NORMAN HAROLD WETZ
Date Signed 10/17/88
ARIZONA, U.S.A.

And ______________________
Norman H. Wetz, P.E.

Registered Professional Engineer (Civil)
CERTIFICATE NO. 13872
LAWRENCE A. HANSEN
Date Signed 10/17/88
ARIZONA, U.S.A.

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

Copies: Addressee (3)

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SHB Job No. E88-9
Report No. 1
Revision No. 1

Registered Professional Engineer
CERTIFICATE NO.
12176
NORMAN HAROLD
WETZ
Date Signed 12/17/88
ARIZONA, U.S.A.

1. INTRODUCTION

This report is submitted pursuant to a geotechnical investigation made by this firm of the site of the new 83rd Avenue Bridge over Skunk Creek located in Peoria, Arizona. The object of this investigation was to evaluate the physical properties of the subsoils underlying the site to provide recommendations for estimated depth of scour, foundation design and abutment support.

2. PROJECT DESCRIPTION

Preliminary details of the proposed construction were provided to us by Lloyd W. Miller, P.E., and Donald R. Davis, P.E., of Hoffman-Miller Engineers, Inc. It is understood that a new bridge is planned over Skunk Creek at 83rd Avenue. The bridge will be about 70 feet wide and 625 feet long with five spans.

3. INVESTIGATION

3.1 Subsurface Exploration

Six exploratory borings were drilled to depths of 80 to 120 feet below existing grade. The borings were performed using our CME-75 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 equipment and procedures, a site plan showing the boring locations, and logs of the test borings. The field investigation was supervised by Kenneth D. Donnelson, staff geologist, of this firm.

3.2 Laboratory Analysis

The moisture content and the dry density of selected samples recovered were determined. The results of these test are presented on the boring logs. Grain-size analysis, Atterberg limits and direct shear tests 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 at the site of the existing two-lane 83rd Avenue Bridge crossing Skunk Creek. The site is typical of an Arizona ephemeral stream bed. Topographic relief is on the order of 18 feet from the abutment locations to the deepest portion of the existing channel. The site is relatively void of vegetative cover, though some areas contain sparse natural desert vegetation. Previous construction noted is the existing 83rd Avenue Bridge and the Outer Loop Bridge, presently under construction downstream of the project site.

4.2 Geotechnical Profile

The subsurface soils encountered at the site consist predominantly of stratified deposits of silty and clayey sands with some gravels, cobbles and occasional boulders. These deposits extended to the full depth of our investigation. These soils are of low to medium plasticity and were found to be firm to hard and very weakly to strongly lime cemented. There are a few zones of relatively clean sand and sand and gravel materials in the upper 5 to 20 feet of the borings. These soils are nonplastic and are medium dense to very dense.

4.3 Soil Moisture & Groundwater Conditions

No free groundwater was encountered in the borings at the time of the investigation and soil moisture contents were low to moderate throughout the depth of the 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 measured. This is apparently the result of water being forced between the sample and the inner wall of the sampler. A temporarily high groundwater table can be expected to exist during and after flows in the channel due to surface water permeating the soils.

5. DISCUSSION & RECOMMENDATIONS

5.1 Analysis of Results

5.1.1 Abutment Piers

Drilled pier foundations bearing a minimum of 10 feet below finished channel grade are recommended for the abutments. It is assumed that bank protection will protect near-surface soils from scour at the bridge abutments. Design criteria for abutment drilled piers are presented in Section 5.2. The use of spread footings for the abutment piers was not analyzed due to the potential of different settlements between the abutments and channel piers.

5.1.2 Channel Piers

Drilled pier foundations bearing a minimum of 43 feet below finished channel grade are recommended for the channel piers. Analysis is based on estimated scour depths of surface soils surrounding the channel piers. Design criteria for channel drilled piers are also presented in Section 5.2. Analyses of potential scour are presented in Section 5.3.

5.2 Drilled, Cast-in-Place Concrete Piers

5.2.1 Downward Loads

Straight-shafted, drilled, cast-in-place concrete piers are recommended for the support of the foundation loads involved. Safe downward capacities of straight shafted piers are presented in Figure 1 for abutment piers and Figure 2 for channel piers. Capacities shown are based on end-bearing only. These 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 the analysis of drilled pier capacity are presented in Appendix C. Complete design calculations are also provided in Appendix C.

5.2.2 Estimated Settlements

Settlements of pier foundations were estimated using two methods outlined in the NAVFAC Design Manual 7.1 and 7.2, (1982)*. The first method utilizes the Schmertmann procedure as outlined in NAVFAC Design Manual 7.1. Estimated settlements based on this method are provided in Appendix C, pages 6 thru 11. These settlements appear to be somewhat conservative

*References are listed at the end of this report.

PROJECT 83rd Avenue Bridge Over Skunk Creek JOB NO. E88-9

FIGURE 1

SAFE DOWNWARD CAPACITIES FOR STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE ABUTMENT PIERS BASED ON END-BEARING ONLY

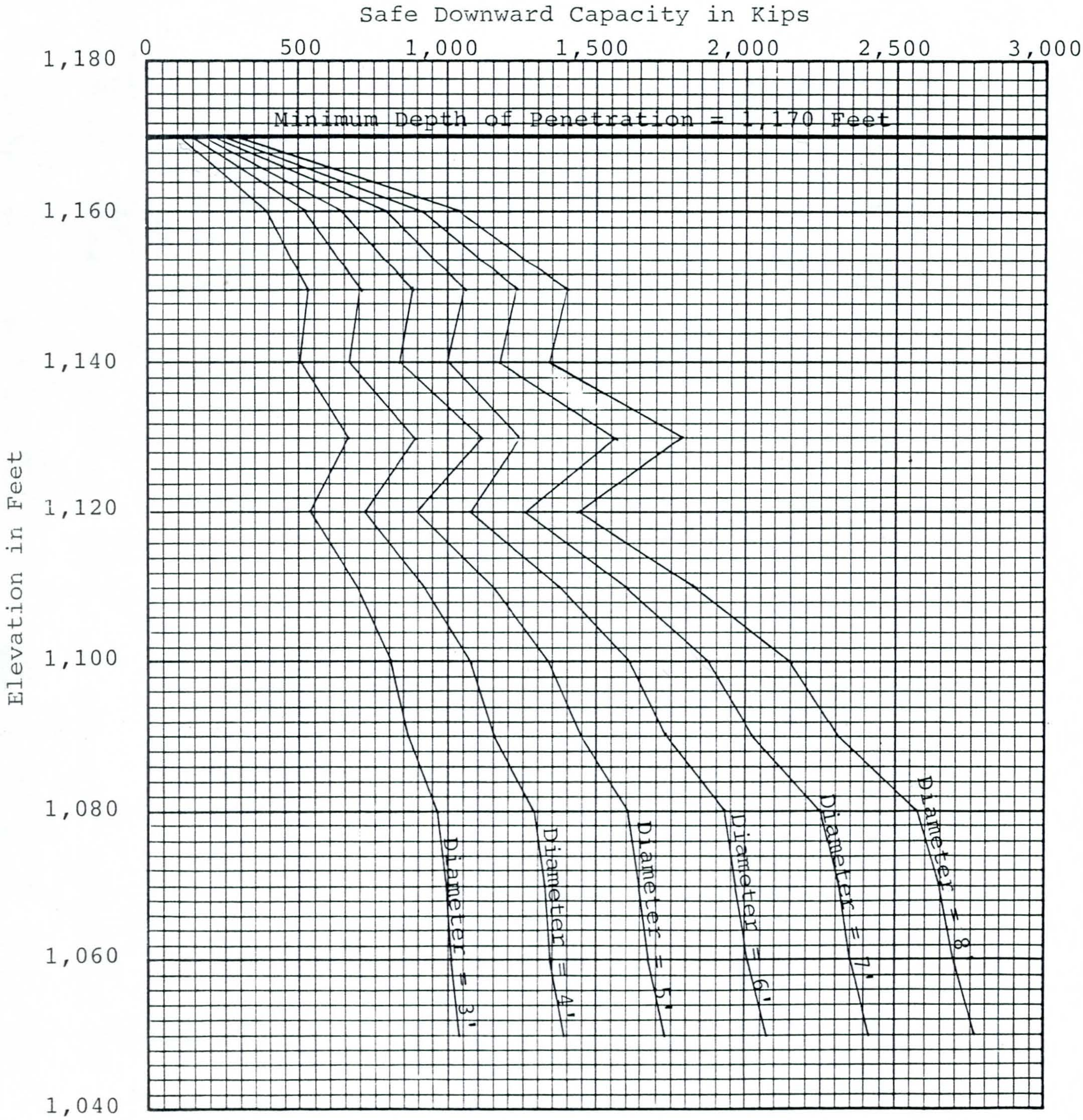

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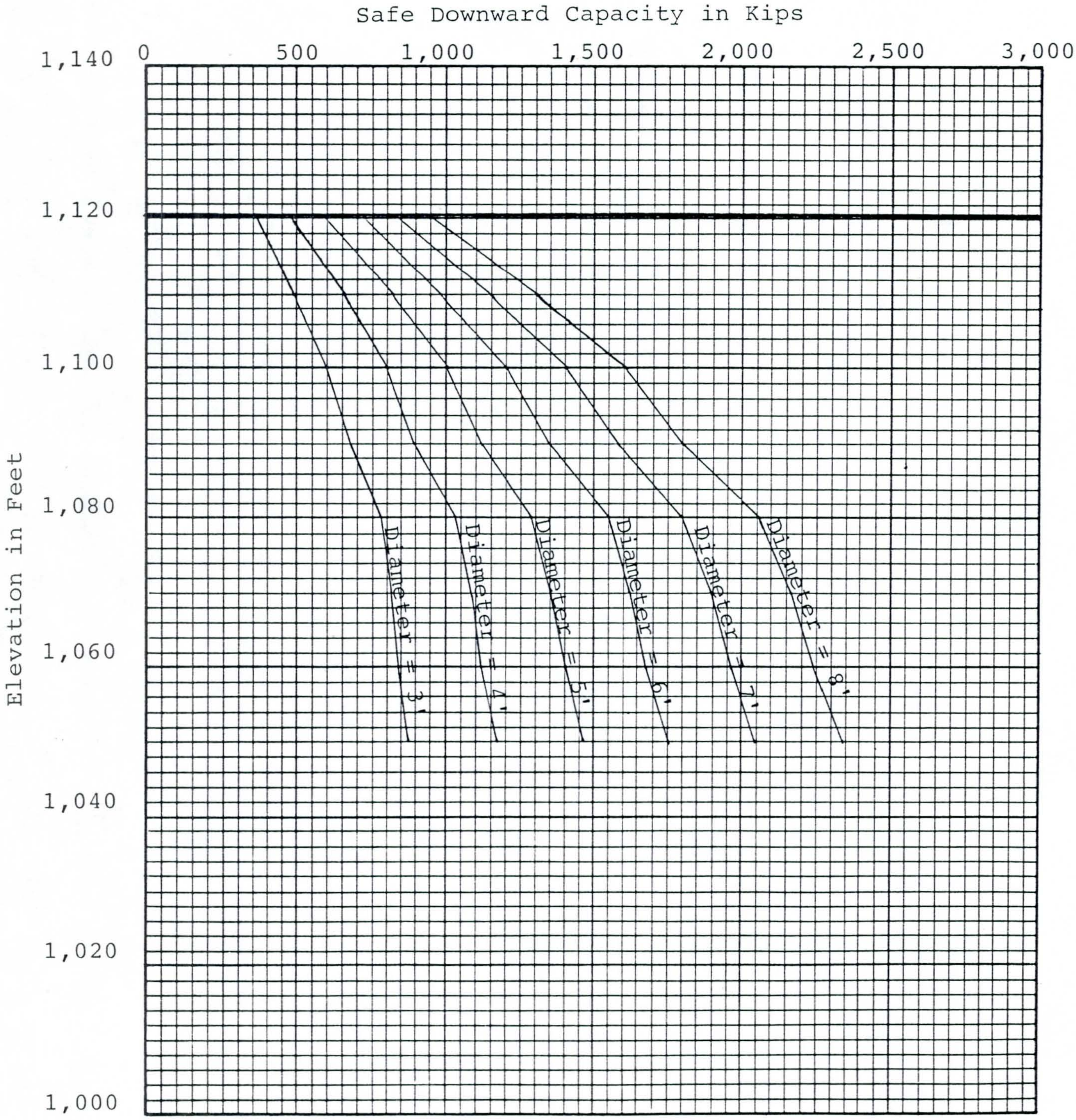
FIGURE 2

SAFE DOWNWARD CAPACITIES FOR STRAIGHT, DRILLED, CAST-IN-PLACE CONCRETE ABUTMENT PIERS BASED ON END-BEARING ONLY



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based on load test data and local experience with similar soils.

Therefore, we analyzed settlements of drilled piers utilizing the Vesic procedure outlined in NAVFAC Design Manual 7.2. Settlement charts were developed for both the end-bearing and side shear cases. Settlements are presented in terms of inches of settlement per kip of vertical load. Using the charts, the settlements can be estimated for straight-shafted piers considering both pier diameter and the pier tip elevation. These values as presented in Appendix C, pages 12 thru 17, appear to be more realistic and are recommended for use in design.

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 and results of direct shear tests, conservative strength parameters recommended for use in computing the ultimate lateral resistance are 0 = 25° and c = 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 110 pounds per cubic foot.

Implementation of Broms' procedures also requires a coefficient of horizontal subgrade reaction, k_h. For the cohesive case, a value of k_hD = 460 pounds per square inch, independent of depth, is recommended. Thus, for a 36-inch diameter pier, k_h = 13 pounds per cubic inch. For the cohesionless case, k_h varies with depth in accordance with the relationship

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

where z is the 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.

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 3 inches of loose material is present after cleaning.

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 5 to 7 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 inspecter to ensure that it meets requirements. An inspection report should be submitted for each pier stating, in writing, that all details have been inspected and meet requirements.

It appears that straight-shafted, drilled pier excavations may require casing or slurry methods for advancement for the upper portion of the drilled piers. Since some caving is anticipated, concrete quantities may exceed the neat volumes indicated by the plans. Guide specifications for drilled, cast-in-place concrete piers utilizing slurry-assisted construction are provided in Appendix D.

5.3 Scour Analysis

The maximum scour within a channel is the sum of the general scour that occurs across the stream bed as a result of constrictions, the local scour that occurs at the obstructions of the foundations and any long-term degradation/aggradation processes that may be taking place. Thus, complete analysis of potential scour requires details of the type of channel being proposed and the configuration of any pier bents that may be used for bridge support.

For the analysis of scour depth, the depth of general scour was estimated using methods outlined by Pemberton and Lara (1984), which is based on the Neill equation utilizing the competent mean velocity. A depth of 20 feet was calculated, assuming an average stream velocity of 11.0 feet per second, a mean particle diameter of 1.5 millimeters and a total flow of 35,000 cubic feet per second. It is assumed in the calculations that the abutments for the bridge structure will be oriented

design Q

parallel to the channel and that the channel will have either a straight line configuration or a moderate bend.

Many procedures have been developed for estimating the depth of local scour adjacent to bridge piers. These have been summarized by Anderson (1974) and Laursen (1980). Typically, these procedures involve calculation of the equilibrium local scour depth. The various formulas available were applied to the case of support for the bridge structure involving 3- to 8-foot diameter piers. Where required by the analysis, the mean grain-size of the channel bed material being transported was assumed equal to 1.5 millimeters. Based on the conceptual plan provided, the piers will be oriented along the flow direction with three piers per bent.

Analysis results are summarized in Table 1, which lists the depth of local scour below the mean stream bed elevation. The various procedures predict a maximum depth of local scour varying from as little as 6 1/2 feet to as much as 14 1/2 feet for a 3-foot diameter pier. Table 2 presents total scour depths calculated as the sum of local and general scours. Local scour values used are based on the averages from the various methods.

5.4 Abutment Walls

5.4.1 Backfill

Because of the potential for flow of water adjacent to

TABLE 1

Summary of Local Scour Depth Predictions

Maximum Scour Depth in Feet

Procedure	Notes	D=3′	D=4′	D=5′	D=6′	D=7′	D=8′	
Blench	1,2	10.54	12.15	13.47	14.62	15.62	16.53	
Inglis-Poona	1,2	13.61	15.45	16.96	18.27	19.42	20.45	
Laursen II	1,2	6.32	7.28	8.13	8.90	9.61	10.26	
Neill	1,2	6.64	8.13	9.50	10.79	12.02	13.20	
~~Shen I~~	~~1,2~~	7.91 ~~14.30~~	9.45 ~~17.32~~	10.85 ~~20.10~~	12.15 ~~22.70~~	13.39 ~~25.16~~	14.52 ~~27.50~~	equilibrium scour
Shen II	1,2	11.00	13.32	15.46	17.45	19.34	21.14	
FHWA	3	7.51	9.05	10.46	11.78	13.02	14.20	
~~Laursen III~~	~~2~~	~~6.34~~	~~7.31~~	~~8.16~~	~~8.93~~	~~9.64~~	~~10.29~~	clear water scour

Notes:
1. Procedure cited in Anderson (1974).
2. Procedure cited in Laursen (1980).
3. Procedure cited in FWHA Training & Design Manual(1975).

TABLE 2

Total Scour Depths for Design

Pier Diameter (Feet)	General Scour (Feet)	Local Scour (Feet)		Total Scour (Feet)
3	20	9.53	7.2	29.53
4	20	11.25	9.6	31.25
5	20	12.78	11.4	32.78
6	20	14.18	13.7	34.18
7	20	15.48	14.8	35.48
8	20	16.70	16.3	36.70

the approach fill and relatively rapid drawdown, a clean, granular, free-draining backfill is recommended for use behind the abutment wingwall and retaining walls in conjunction with a weephole system. The backfill should meet the following grading requirements as determined by ASTM D422:

Sieve Size (Square Opening)	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.

5.4.2 Lateral Earth Pressure

The earth pressure against abutment walls will depend upon the degree of restraint. With the recommended backfill and drainage conditions presented in Section 5.4.1, rigid, absolutely restrained abutments will be subjected to earth pressures represented by a hydrostatic load diagram of about 50 pounds per square foot per foot of depth. Lateral translation or rotation of the wall equal to about 0.001 times the height would reduce earth pressures to the active state of about 35 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.

REFERENCES

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Pemberton, E.L. and Lara, J.M., 1984, Computing Degradation and Local Scour, Technical Guideline for Bureau of Reclamation, Engineering and Research Center, Denver, Colorado.

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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.

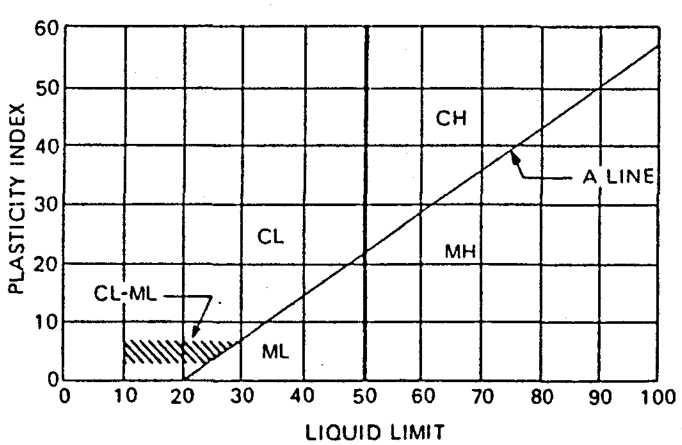
UNIFIED SOIL CLASSIFICATION SYSTEM

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.

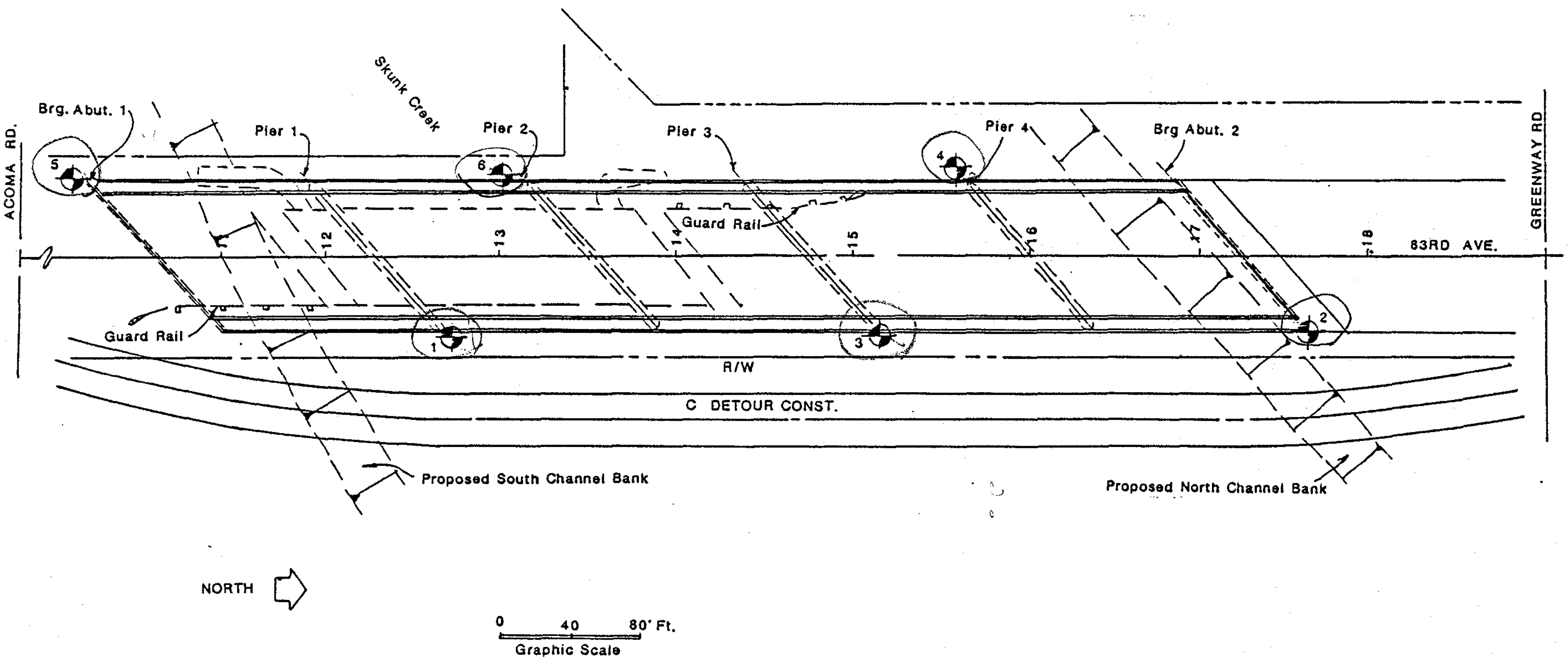
N	Relative Consistency	Remarks
0-2	Very soft	Easily penetrated several inches with fist.
3-4	Soft	Easily penetrated several inches with thumb.
5-8	Medium stiff	Can be penetrated several inches with thumb with moderate effort.
9-15	Stiff	Readily indented with thumb, but penetrated only with great effort.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



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PROJECT 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-4 & 4-6-88

LOG OF TEST BORING NO. 1

location:	SE Abutment
RIG TYPE	CME-75
BORING TYPE	6 5/8" Hollow Stem Auger
SURFACE ELEV.	1164.8'
DATUM	U.S.G.S.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0				S	84			SM-SW
5				S	120		10	
								SC
10				S	50/5½"		5	
15				S	104			CL
20				S	90			SC
25				S	50/3"			SP-SM
30				S	50/5½"			SC
35				S	84			SM-SP
40				U	87			SP-SM
45				S	50/4"			SM
								SC
50								

REMARKS	VISUAL CLASSIFICATION
moist very dense	SAND & GRAVEL, some silt & cobbles, well graded, angular, non-plastic, light brown
moist hard	CLAYEY SAND, considerable gravel, some cobbles, well graded, moderately to strongly lime cemented, medium plasticity, brown note: hole 1 continued on 4-6-88
moist hard	SANDY CLAY, weakly to moderately lime cemented, medium plasticity, light reddish brown to light brown
moist hard	CLAYEY SAND, predominantly fine to medium grained, weakly to moderately lime cemented, low plasticity, brown
moist hard	SILTY SAND, predominantly fine to medium grained, weakly lime cemented, nonplastic, brown
moist hard	CLAYEY SAND, some gravel, well graded, weakly lime cemented, low plasticity, brown mottled w/some orange & green
moist hard	SILTY SAND, predominantly fine to medium grained, weakly to moderately lime cemented, nonplastic, tan
moist hard	SAND, considerable gravel, some silt, predominantly fine to medium grained, weakly to moderately lime cemented, nonplastic, tan

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 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-4 & 4-6-88

LOG OF TEST BORING NO. 1

RIG TYPE CME-75

BORING TYPE 6 5/8" Hollow Stem Auger

SURFACE ELEV. 1164.8'

DATUM U.S.G.S.

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
50				S	84			SC
55				S	89			
								SM
60				S	47 (no recovery)			
65				S	50/2"			GM
70				S	54			CL
75				S	50/4"			SM
				S	50/5½"			GC
80								

REMARKS	VISUAL CLASSIFICATION
slightly moist hard	(depth 43' to 48') SILTY SAND, some to considerable gravel, predominantly fine grained, moderately lime cemented, nonplastic, light brown
moist hard	(depth 48' to 53') CLAYEY SAND, some gravel, weakly to moderately lime cemented, low plasticity, tan
moist hard to very firm	SILTY SAND, predominantly fine to medium grained, weakly lime cemented, nonplastic, brown
dry hard	SILTY SAND & GRAVEL, poorly graded, angular, moderately lime cemented, nonplastic, light brown
very moist hard	SANDY CLAY, some gravel, weakly lime cemented, medium plasticity, brown
moist hard	SILTY SAND, some gravel, well graded, weakly lime cemented, nonplastic, brown
moist hard	CLAYEY SAND & GRAVEL, poorly graded, weakly to moderately lime cemented, low plasticity, light brown
	Stopped auger at 79'6" Sampler refused at 79'11½"

GROUND WATER

DEPTH	HOUR	DATE

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 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-4 & 4-7-88

LOG OF TEST BORING NO. 2

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1177.4'
DATUM U.S.G.S.

Depth in Feet	Continuous Penetration Resistance	Graphical Log	Sample	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0				S	50/5"			GC
5				S	12			
9								GP-GM
10				S	66		5	
15				S	50/5½"			GM-GP
20				S	144			SC
25				S	50/5"			
27								SC
30				S	85			CL
35				S	54 (no recovery)			
40				U	100/10"	116	14	
44								SM
45				S	50/1" (no recovery)			
50				S	50/3" (no recovery)			

REMARKS	VISUAL CLASSIFICATION
slightly moist; hard	CLAYEY SAND & GRAVEL, some cobbles, fine to coarse grained, well graded, subrounded, weakly to moderately lime cemented, low plasticity, brown
slightly moist; medium dense to very dense	SAND & GRAVEL, some silt, poorly graded, nonplastic, brown
slightly moist; very dense	SILTY SAND, GRAVEL & COBBLES, poorly graded, subrounded, nonplastic, brown
moist; hard	CLAYEY SAND, some gravel, predominantly medium to coarse grained, angular, weakly to moderately lime cemented, low plasticity, light reddish brown
moist; hard	CLAYEY SAND, predominantly fine grained, weakly to moderately lime cemented, low plasticity, light brown
moist; hard	SANDY CLAY, weakly to moderately lime cemented, medium plasticity, brown
moist; hard	SILTY SAND, some gravel, trace of clay, predominantly fine to medium grained, weakly lime cemented, nonplastic, brown

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 83rd Avenue Bridge Over Skunk Creek

LOG OF TEST BORING NO. 2

JOB NO. E88-9 DATE 4-4 & 4-7-88

RIG TYPE	CME-75
BORING TYPE	6 5/8" Hollow Stem Auger
SURFACE ELEV.	1177.4'
DATUM	U.S.G.S.

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								SM		
55			X	S	21 (no recovery)				moist to very moist firm to hard	CLAYEY SAND, some gravel, predominantly fine grained, weakly lime cemented, medium plasticity, brown
60			X	S	54		22	SC		
65			X	S	50/5½"				moist hard	CLAYEY SAND, some to considerable gravel, predominantly fine to medium grained, weakly to moderately lime cemented, low plasticity, brown
70			X	S	50/5"			SC		
75				S	50/5"					
80				S	50/6"			CL	slightly moist hard	SANDY CLAY, trace of gravel, predominantly fine grained, moderately lime cemented, medium plasticity, light brown

Stopped auger at 79'6"
Sampler refused 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.
U – 3" O.D. 2.42" I.D. tube sample.
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JOB NO. E88-9 DATE 4-4-88

LOG OF TEST BORING NO. 3

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1178.0'
DATUM U.S.G.S.

Depth in Feet	Continuous Penetration Resistance	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
0–5		S	38			SM	slightly moist dense	SILTY SAND, some gravel & cobbles, well graded, nonplastic, brown note: considerable gravel from 3½' to 5'
5–8		S	28			SC	very moist firm	CLAYEY SAND, considerable gravel, well graded, weakly lime cemented, low plasticity, brown
8–14		S	41			SM	very moist very firm	SILTY SAND, some gravel, well graded, subrounded, weakly lime cemented, nonplastic, brown
14–23		S	50/6"			GP	very moist very dense	SANDY GRAVEL & COBBLES, some silt, well graded, nonplastic, brown
		S	50/2"					
23–34		S	50/1"			GC	very moist hard	CLAYEY SAND & GRAVEL, some cobbles, well graded, weakly to moderately lime cemented, medium plasticity, brown
		S	82			SC	moist hard	CLAYEY SAND, predominantly fine to medium grained, weakly to moderately lime cemented, low plasticity, brown
34–38		U	75	99	23	SM-SC	moist hard	SILTY SAND, some clay, predominantly fine grained, weakly to moderately lime cemented, low plasticity, brown
38–50		S	50/6"				moist to very moist hard	CLAYEY SAND, predominantly fine to medium grained, weakly lime cemented, medium plasticity, brown note: some cobbles from 42' to 43'
		S	50/5"		31	SC		
50		S	50/5½"			SW		

GROUND WATER

DEPTH	HOUR	DATE
	none	

SAMPLE TYPE

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

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LOG OF TEST BORING NO. 3

JOB NO. E88-9 DATE 4-4-88

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1178.0'
DATUM U.S.G.S.

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								SW		(depth 49½' to 52')
									slightly moist hard	SAND & GRAVEL, well graded, weakly lime cemented, nonplastic, light brown
55				S	23 (no recovery)					
									moist firm to hard	SANDY CLAY, weakly lime cemented, low to medium plasticity, brown note: trace of gravel below 64'
60				S	36		27	CL		
65				S	78					
									moist hard	CLAYEY SAND, predominantly fine grained, weakly lime cemented, medium plasticity, brown
70				S	76			SC		
75				S	50/4"					
									moist hard	CLAYEY SAND & GRAVEL, trace of cobbles, well graded, weakly to moderately lime cemented, low plasticity, brown
								GC		
				S	50/3"					
80										Stopped auger at 79'6" Sampler refused at 79'8"

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE
A – Auger cuttings. B – Block sample
S – 2" O.D. 1.38" I.D. tube sample.
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JOB NO. E88-9 DATE 4-5-88

LOG OF TEST BORING NO. 4

RIG TYPE CME-75

BORING TYPE 6 5/8" Hollow Stem Auger

SURFACE ELEV. 1171.8'

DATUM U.S.G.S.

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	70			GM	slightly moist very dense	SILTY SAND & GRAVEL, well graded, nonplastic, brown
5				S	52			SM	slightly moist to moist very dense	SILTY SAND, some gravel, well graded, nonplastic, brown
10				S	50/3"					
15				S	50/3½"			SP-SM	slightly moist hard	SAND, some silt & gravel, predominantly medium to coarse grained, weakly lime cemented, nonplastic, light brown note: occasional cobbles
20				S	50/5"			GC	moist hard	CLAYEY SAND & GRAVEL, some cobbles, fine to coarse grained subrounded gravel, well graded, weakly lime cemented, low plasticity, brown
25				S	50/2"					
30				S	79			SC	moist hard	CLAYEY SAND, trace of gravel, predominantly fine grained, moderately lime cemented, low plasticity, brown
35				U	100	94	26	CL	moist hard	SANDY CLAY, trace of gravel, moderately to strongly lime cemented, medium plasticity, brown
40				S	50/5"					
45				S	50/5½"					
50				S	50/5½"					

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 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-5-88

LOG OF TEST BORING NO. 4

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1171.8'
DATUM U.S.G.S.

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								SW	dry very dense	SAND & GRAVEL, well graded, nonplastic, grayish brown
55				S	44		13		moist very firm to hard	CLAYEY SAND, considerable gravel, predominantly fine to medium grained, weakly to moderately lime cemented, medium plasticity, light brown to brown
60				U	67					
65				S	50/5½"			SC		
70				S	50/5½"					
75				S	50/4"					
80				S	50/4"					
85										Stopped auger at 79'6" Sampler refused at 80'4"

GROUND WATER

DEPTH	HOUR	DATE

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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JOB NO. E88-9 DATE 4-11-88

LOG OF TEST BORING NO. 5

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1178.1'
DATUM U.S.G.S.

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	49		10	SC	moist dense	CLAYEY SAND, some to considerable gravel, predominantly fine to medium grained, medium plasticity, brown
				A						
5				S	15			SM	moist medium dense	SILTY SAND, some gravel, well graded, nonplastic, brown
10				S	29			SM-SW	moist medium dense	SAND & GRAVEL, some silt, well graded, non-plastic, brown
				A			4		moist hard	CLAYEY SAND & GRAVEL, some cobbles, well graded, weakly lime cemented, medium plasticity, brown
15				S	50/0" (no recovery)					
								GC		
20				S	50/5"		5			
25				S	50/3"		11	GC	very moist hard	CLAYEY SAND & GRAVEL, trace of cobbles, well graded, weakly lime cemented, low plasticity, brown
30				S	50/5½"		36	SC	moist hard	CLAYEY SAND, predominantly fine grained, moderately lime cemented, low plasticity, brown
35				S	49		28		moist very firm	CLAYEY SAND, predominantly fine grained, weakly to moderately lime cemented, medium plasticity, brown note: trace of gravel
								SC		
40				U	100/4"	99	25			
45				S	50/5"		11	SC	moist hard	CLAYEY SAND, well graded, weakly lime cemented, low plasticity, brown
								SM		
50				S	50/5"		12			

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 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-11-88

LOG OF TEST BORING NO. 5

RIG TYPE CME-75

BORING TYPE 6 5/8" Hollow Stem Auger

SURFACE ELEV. 1178.1'

DATUM U.S.G.S.

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								SM		(depth 47' to 53')
55				S	50/5"		11	SC	moist hard	SILTY SAND, some gravel, well graded, weakly lime cemented, nonplastic, light brown
60				S	31		24		moist hard	CLAYEY SAND, considerable gravel, predominantly fine to medium grained, weakly lime cemented, low plasticity, brown
65				S	50		16		moist very firm to hard	CLAYEY SAND, trace of gravel, predominantly fine grained, weakly to moderately lime cemented, low plasticity, brown
70				S	87		23	SC		
75				S	50/3"		11			
80				S	55		29	CL	moist hard	SANDY CLAY, weakly to moderately lime cemented, low plasticity, brown
85				U	100/11"	87	32		moist hard	CLAYEY SAND, trace of gravel, predominantly fine grained, weakly to moderately lime cemented, low plasticity, brown
90				S	115		20	SC		
95				S	50/6"		17			Stopped auger at 94'6" Sampler refused at 95'6"
100										

GROUND WATER

DEPTH	HOUR	DATE

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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JOB NO. E88-9 DATE 4-12-88

LOG OF TEST BORING NO. 6

RIG TYPE CME-75

BORING TYPE 6 5/8" Hollow Stem Auger

SURFACE ELEV. 1162.9'

DATUM U.S.G.S.

Depth in Feet	Continuous Penetration Resistance	Sample Type	Blows per foot 140 lb. 30" free-fall drop hammer	Dry Density Lbs. per cu. ft.	Moisture Content Per Cent of Dry Wt.	Unified Soil Classification
0						
5		S	50/5"		5	GW-GM
10		S	50/5"		4	
15		S	50/4"		30	SM
20		S	50/5½"		33	
25		S	50/4"		13	SC
30		S	29		20	CL
35		U	100/7"	122	13	GC
40		S	50/5½"		13	
45		S	59		22	SC
50		S	50/4"		13	

REMARKS	VISUAL CLASSIFICATION
moist very dense	SILTY SAND, GRAVEL & COBBLES, well graded, subrounded, low plasticity, brown
moist hard	SILTY SAND, trace of clay, predominantly fine grained, weakly to moderately lime cemented, nonplastic, light orangish brown
moist hard	CLAYEY SAND, trace of gravel, predominantly fine grained, weakly to moderately lime cemented, low plasticity, light reddish brown
moist firm	SANDY CLAY, considerable silt, weakly lime cemented, medium plasticity, light brown
moist hard	CLAYEY SAND & GRAVEL, trace of cobbles, well graded, angular, moderately lime cemented, low plasticity, brown
moist hard	CLAYEY SAND, trace of gravel, predominantly fine to medium grained, moderately lime cemented, low to medium plasticity, brown

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 83rd Avenue Bridge Over Skunk Creek

JOB NO. E88-9 DATE 4-12-88

LOG OF TEST BORING NO. 6

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1162.9'
DATUM U.S.G.S.

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										
55				S	50/4"		14	SC		
60				S	50/4"		11			
									moist hard	SANDY CLAY, some gravel, moderately to strongly lime cemented, low plasticity, brown note: micaceous
65				S	50/6"		18	CL		
									moist hard	CLAYEY SAND, some gravel, predominantly fine to medium grained, moderately lime cemented, medium plasticity, brown
70				S	106		17			
75				S	50/4"		13			
80				S	50/5½"		15			
								SC		
85				S	50/6"		10			
90				S	50/3"		8			
95				S	50/0"	(no recovery)				
								CL		
100				S	50/3"		17			

GROUND WATER

DEPTH	HOUR	DATE

SAMPLE TYPE

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

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LOG OF TEST BORING NO. 6

JOB NO. E88-9 DATE 4-12-88

RIG TYPE CME-75
BORING TYPE 6 5/8" Hollow Stem Auger
SURFACE ELEV. 1162.9'
DATUM U.S.G.S.

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
100								CL	considerably moist hard	(depth 97' to 106') CLAYEY SAND, trace of gravel, predominantly fine to medium grained, weakly to moderately lime cemented, medium plasticity, brown
105				S	50/2"		12			
110				S	50/3" (no recovery)			GC	very moist hard	CLAYEY SAND & GRAVEL, well graded, subangular, moderately lime cemented, medium plasticity, brown note: some cobbles below 113'
115				S	50/3"		10			
				S	50/3"		9			
120										Stopped auger at 119'6" Sampler refused at 119'9"

GROUND WATER

DEPTH	HOUR	DATE

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. E88-9
W/O 1

HOLE NO	DEPTH	UNIFIED CLASS	L.L.	P.I.	#200 .75"	#100 1"	#50 1.5"	#40 2"	#30 2.5"	1.19 #16 3"	2.0 #10 3.5"	#8 4"	#4 6"	.25" 8"	.375" 10"	.5" 12"	LAB NO	#10
1	4'6"-6'	SC	33	17	15 89	18 89	22 100	26	31	46	54	57	69	75	80	88	8-9-2	
2	9'6"-11'	GP-GM	NV	NP	8.8 68	12 68	19 100	24	28	37	42	43	49	53	57	59	8-9-6	
3	44'6"-45'	SC	47	26	47	58	69	74	81	88	93	94	100				8-9-17	
3	59'6"-61'	CL	36	16	51	66	84	92	96	98	99	99	100				8-9-19	
4	54'6"-56'	SC	46	23	33 79	40 79	49 100	54	60	66	70	71	73	75	77	78	8-9-34	
1	39'6"-40'6"	SP-SM	NV	NP	8.0 100	13	29	42	52	61	66	67	73	76	80	85	8-9-45	
2	59'6'-61'	SC	46	24	43 85	50 100	60	66	71	75	77	78	79	79	80	81	8-9-57	
5	6"-2'	SC	39	20	45 86	52 100	58	61	65	72	76	78	83	84	86	86	8-9-62	
5	19'6"-20'6"	GC	35	20	13 65	15 77	18 100	20	24	31	38	40	47	50	55	58	8-9-66	
6	89'6"-90'6"	SC	33	16	27 87	32 87	43 100	51	59	65	70	72	79	81	83	84	8-9-100	
6	104'6"-105'		NA	NA	37 100	43	52	58	64	73	78	80	87	89	92	98	8-9-102	

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REPORT OF LABORATORY TESTS	DATE 4/26/88
PROJECT: 83 AVE. BRIDGE -SKUNK CREEK	JOB NO. E88-9
LOCATION:#1 @ 39'6" TO 40'6"	W.O.NO. 1
	LAB NO. 45

DIRECT SHEAR TEST(SATURATED)ASTM D-3080

POINT NO. 1 (NORMAL STRESS 2.998 KSF)

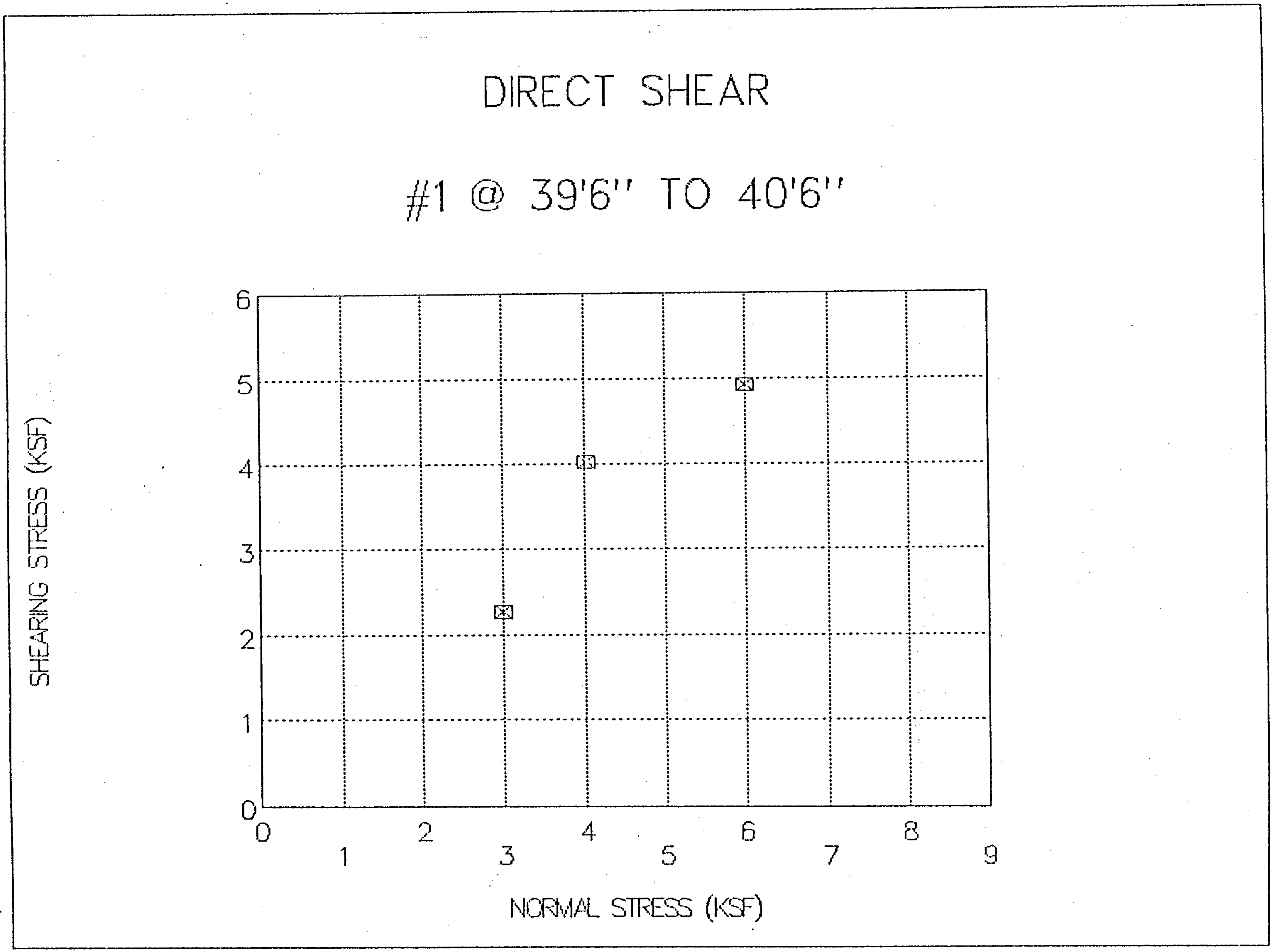
Initial Moisture Content	10.8%
Dry Density γ_d	103.1 LB/CU FT
Moisture at Saturation	23.9%
Maximum Vertical Deformation @ T max.	0.012 IN
Shearing Stress, T max.	2.3 KSF

POINT NO. 2 (NORMAL STRESS 4.02 KSF)

Initial Moisture Content	14.6%
Dry Density	102.9 LB/CU FT
Moisture at Saturation	21.2%
Maximum Vertical Deformation @ T max.	0.002 IN
Shearing Stress, T max.	4.0 KSF

POINT NO. 3 (NORMAL STRESS 6.003 KSF)

Initial Moisture Content	11.1%
Dry Density	104.3 LB/CU FT
Moisture at Saturation	23.2%
Maximum Vertical Deformation @ T max.	-0.017 IN
Shearing Stress, T max.	4.9 KSF


DIRECT SHEAR
#1 @ 39'6" TO 40'6"
SHEARING STRESS (KSF)
0
1
2
3
4
5
6
0
1
2
3
4
5
6
7
8
9
NORMAL STRESS (KSF)

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REPORT OF LABORATORY TESTS	DATE 4/26/88
PROJECT: 83 AVE. BRIDGE -SKUNK CREEK	JOB NO. E88-9
LOCATION:#4 @ 59'6" TO 60'6"	W.O.NO. 1
	LAB NO. 35

DIRECT SHEAR TEST(SATURATED)ASTM D-3080

POINT NO. 1 (NORMAL STRESS 2.998 KSF)

Initial Moisture Content	21.7%
Dry Density	101.0 LB/CU FT
Moisture at Saturation	25.2%
Maximum Vertical Deformation @ T max.	0.007 IN
Shearing Stress, T max.	2.4 KSF

POINT NO. 2 (NORMAL STRESS 6.003 KSF)

Initial Moisture Content	20.2%
Dry Density	103.2 LB/CU FT
Moisture at Saturation	23.1%
Maximum Vertical Deformation @ T max.	0.011 IN
Shearing Stress, T max.	5.2 KSF

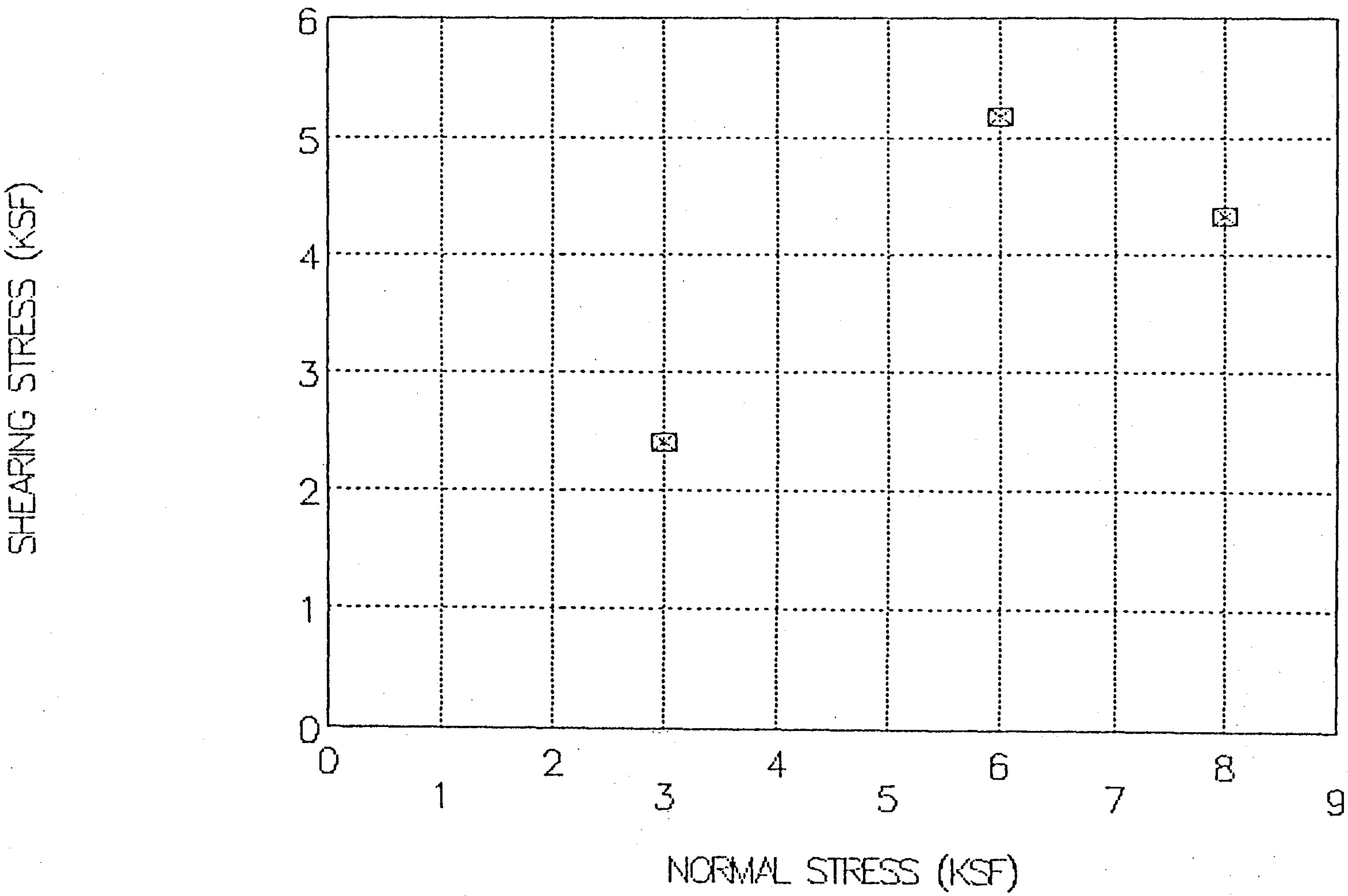
POINT NO. 3 (NORMAL STRESS 8.01 KSF)

Initial Moisture Content	17.5%
Dry Density	107.0 LB/CU FT
Moisture at Saturation	19.2%
Maximum Vertical Deformation @ T max.	-0.007 IN
Shearing Stress, T max.	4.3 KSF

$S = .252 = \frac{wG}{e}$ $\frac{W_s}{V_s} = 101 = \rho_s$ $S =$

DIRECT SHEAR

#4 @ 59'6" TO 60'6"



DESIGN OF DRILLED, CAST IN PLACE, CONCRETE PIERS FOR ABUTMENTS AND CHANNEL PIERS

SAFE DOWNWARD CAPACITY

* FROM NAVFAC 7.2 pg 200-201

A. CONSIDER END-BEARING ONLY

$$q_s = \frac{1}{9}\left(\frac{0.4\,\bar{N}\,D}{B}\right)2 \quad KSF \qquad \frac{1}{3}^{*}$$

WHERE :

$\bar{N} = C_N \cdot N$

N = STANDARD PENETRATION RESISTANCE NEAR PILE TIP.

$C_N = 0.77 \log_{10} \frac{20}{p}$ WHERE p = EFFECTIVE OVERBURDEN STRESS AT PILE TIP.

D = DEPTH OF PILE TIP (FT)

B = DIAMETER OF PILE TIP (FT)

$$q_{SAFE} = \frac{.09\,\bar{N}\,D}{B}\left(\frac{\pi B^2}{4}\right) = .07\,\bar{N}(D)(B)$$

B CONSIDER SIDE SHEAR ONLY

$$f_s = .5\left(\frac{N}{50}\right).333(2) \quad KSF$$

$$= .0067\,N_{AVE}$$

$$q_s = .0067\,N_{AVE}\,(D)\,\pi B$$

WHERE D = DEPTH OF EMBEDMENT

B = PIER DIAMETER

$$q_s = .02094\,N_{AVE}\,(D)(B)$$

FIGURE C-1 PRESENTS A SUMMARY OF AVE. BLOW COUNTS N_{AVE}

FIGURE C-2 PRESENTS A SUMMARY OF CORRECTED BLOW COUNTS $\bar{N}$

TABLES C-1 THROUGH C-4 PRESENT THE ESTIMATED SAFE CAPACITIES

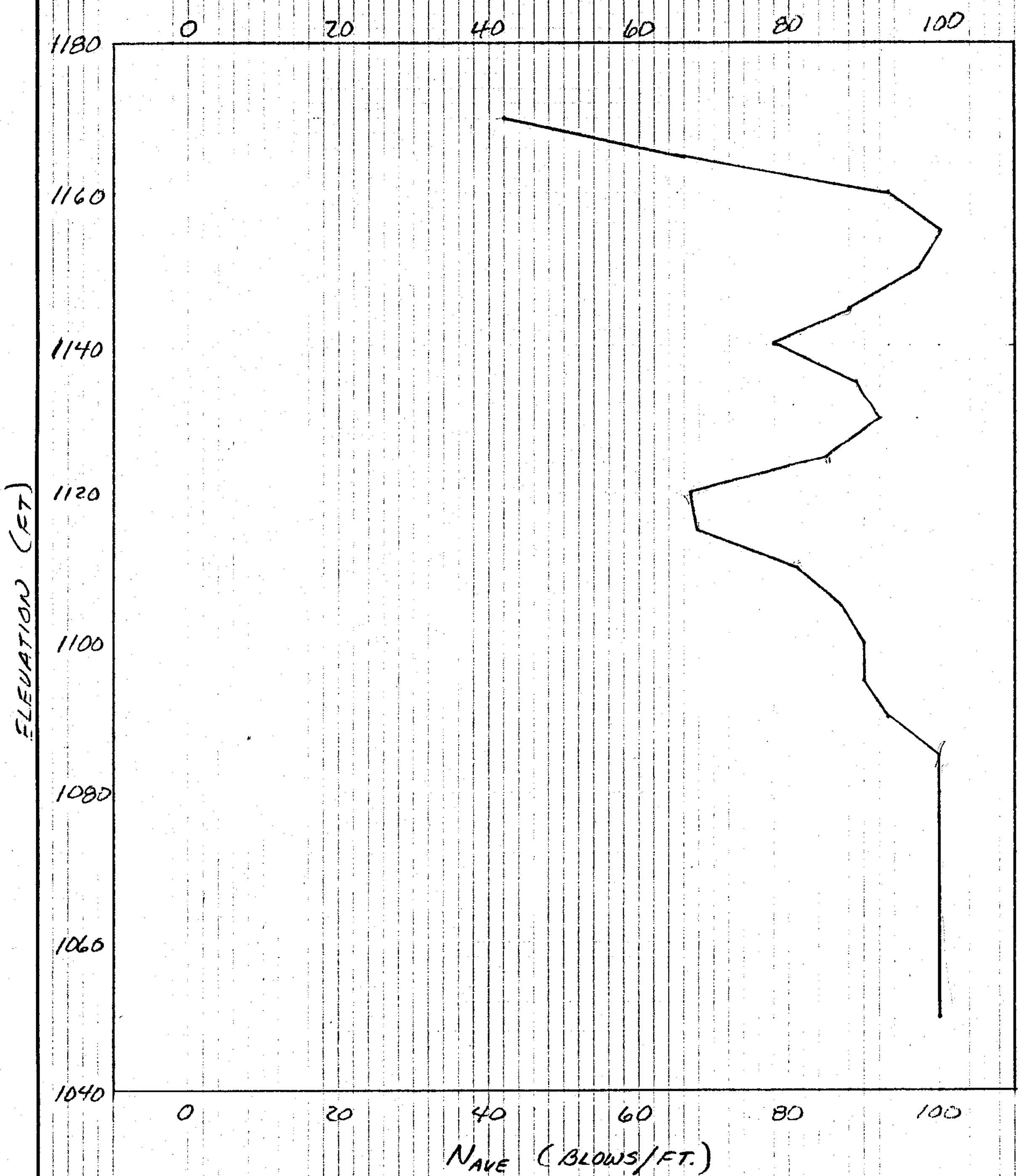


FIG C-1 AVERAGE BLOWCOUNT PROFILE FOR ABUTMENTS AND CHANNEL PIERS FROM BOREHOLES 1 THROUGH 6

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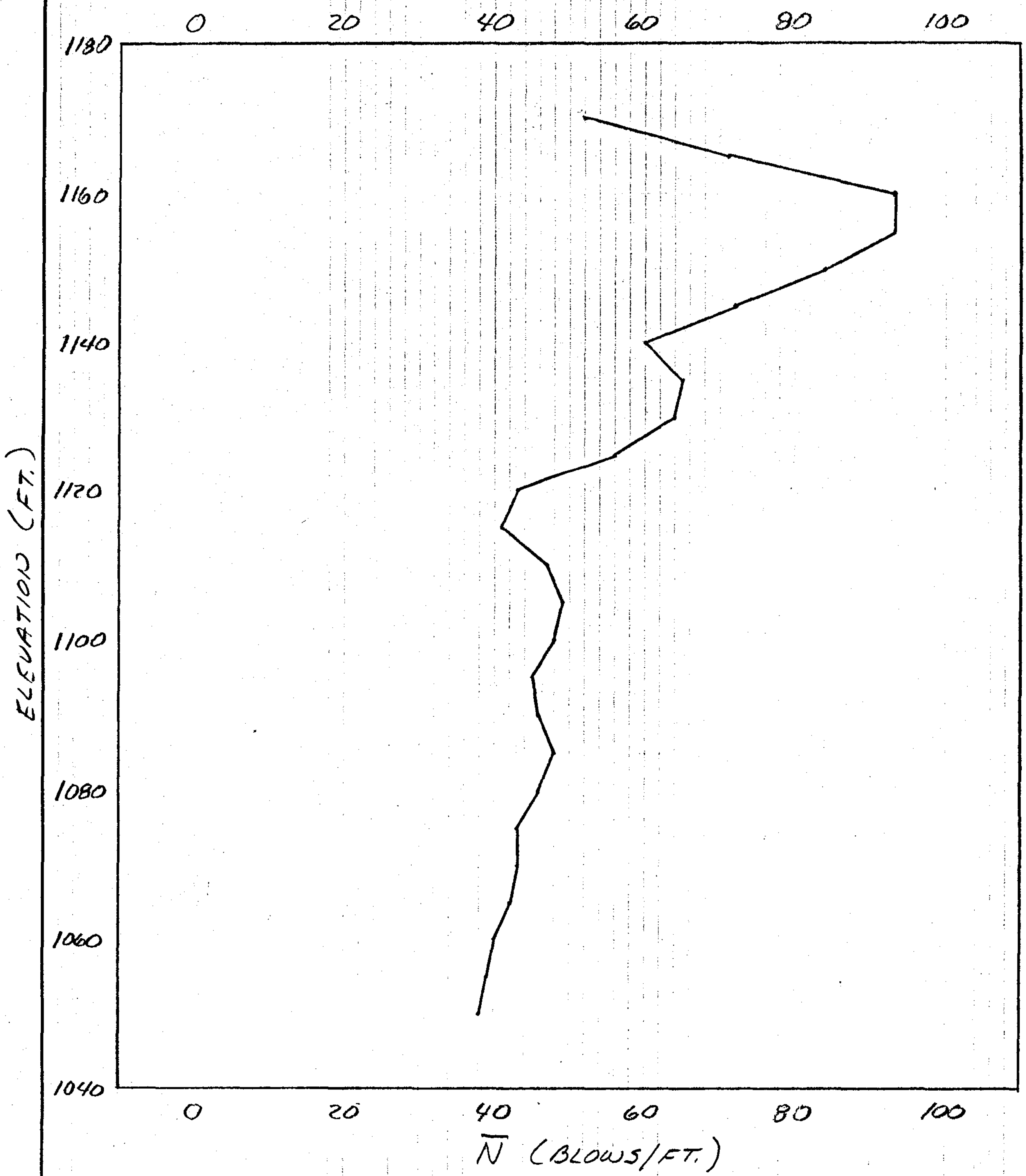

FIG C-2 CORRECTED BLOWCOUNT PROFILE FOR ABUTMENTS AND CHANNEL PIERS FROM BOREHOLES 1 THROUGH 6

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TABLE C-1 SAFE DOWNWARD CAPACITY FOR ABUTMENTS BASED ON END BEARING ONLY

PIER TIP ELEV. FT	$\overline{N}$ BLOWS/FT	CAPACITY, KIPS					
		B=3' 7.1	4' 12.6	5' 19.6	6' 28.3	7' 38.5	8' 50.3
1170	52	109	145	182	218	254	290
1160	93	390	519	649	779	909	1039
1150	84	528	704	880	1056	1232	1407
1140	60	503	670	838	1005	1173	1340
1130	64	670	894	1117	1340	1564	1787
1120	43	540	721	901	1081	1261	1441
1110	47	689	919	1148	1378	1608	1838
1100	48	804	1072	1340	1609	1877	2145
1090	46	867	1156	1445	1734	2023	2312
1080	46	963	1285	1606	1927	2248	2569
1070	43	991	1321	1651	1981	2312	2642
1060	40	1005	1340	1676	2011	2346	2681
1050	38	1035	1380	1724	2069	2414	2759

TABLE C-2 SAFE DOWNWARD CAPACITY FOR ABUTMENTS BASED ON SIDE SHEAR ONLY

PIER TIP ELEV. FT	N_{AVE} BLOWS/FT	CAPACITY, KIPS					
		B=3'	4'	5'	6'	7'	8'
1170	42	26	35	44	53	62	70
1160	71	89	119	149	178	208	238
1150	79	149	199	248	298	347	397
1140	79	199	265	331	397	463	529
1130	82	258	343	429	515	601	687
1120	79	298	397	496	596	695	794
1110	79	347	463	579	695	811	926
1100	81	407	543	679	814	950	1086
1090	82	464	618	773	927	1082	1236
1080	84	528	704	880	1055	1231	1407
1070	85	587	783	979	1175	1371	1566
1060	87	656	875	1093	1312	1530	1749
1050	88	719	958	1198	1437	1677	1916

TABLE C-3 SAFE DOWNWARD CAPACITY FOR CHANNEL PIERS BASED ON END BEARING ONLY

PIER TIP ELEV. FT	$\bar{N}$ BLOWS/FT.	CAPACITY, KIPS					
		B=3' 7.1	4' 12.6	5' 19.6	6' 28.3	7' 38.5	8' 50.3
1150	84	176	235	293	352	411	469
1140	60	251	335	419	503	586	670
1130	64	402	536	670	804	938	1072
1120	43	360	480	600	721	841	961
1110	47	492	656	820	984	1148	1313
1100	48	603	804	1005	1206	1407	1609
1090	46	674	899	1124	1349	1574	1798
1080	46	771	1028	1285	1542	1798	2055
1070	43	811	1081	1351	1621	1891	2161
1060	40	838	1117	1396	1676	1955	2234
1050	38	876	1167	1459	1751	2043	2335

TABLE C-4 SAFE DOWNWARD CAPACITY FOR CHANNEL PIERS BASED ON SIDE SHEAR ONLY

PIER TIP ELEV. FT	N_{AVE} BLOWS/FT	CAPACITY, KIPS					
		B=3'	4'	5'	6'	7'	8'
1150	95	60	80	100	119	139	159
1140	89	112	149	186	224	261	298
1130	90	170	226	283	339	396	452
1120	85	214	285	356	427	498	570
1110	85	267	356	445	534	623	712
1100	85	320	427	534	641	748	854
1090	86	378	504	630	756	882	1008
1080	88	442	590	737	885	1032	1179
1070	89	503	671	839	1006	1174	1342
1060	90	565	754	942	1131	1319	1508
1050	91	629	838	1048	1258	1467	1677

NOTE: DUE TO SCOUR RESTRAINTS, MINIMUM DEPTH OF EMBEDMENT IS AT ELEVATION 1120

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Computed by: GNS Ckd. by: NHW
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SETTLEMENT ANALYSIS
(Schmertman's Method)

A. INSTANTANEOUS SETTLEMENTS*

$$\Delta H_i = \frac{\cancel{2}q\, B^2}{K_{vi}\,(B+1)^2}$$

*FROM NAVFAC DM-7.1 FIG. 6 PAGE 219

WHERE:

ΔH_i = IMMEDIATE SETTLEMENT OF PIER IN FT

q = PIER UNIT LOAD IN KSF

B = PIER DIAMETER IN FT

K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION IN KIPS/FT^3

TO PRESENT SETTLEMENT IN TERMS OF INCH/KIP VERT. LOAD

$$\Delta H_i(\text{in})/\text{KIP} = \frac{2(q\ \text{KIP}/\text{FT}^2)(B^2\ \text{FT}^2)}{(K_{vi}\ \text{KIP}/\text{FT}^3)((B+1)^2\ \text{FT}^2)\left(\frac{\pi B^2}{4}\ \text{FT}^2\right)}\left(\frac{12\ \text{IN}}{1\ \text{FT}}\right)$$

$$= \frac{2\,\cancel{B^2}(12)(4)}{K_{vi}\,(B+1)^2\,\pi\,\cancel{B^2}} = \frac{30.56}{K_{vi}\,(B+1)^2}$$

K_{vi} = 500 KIPS/FT^3 (FOR RELATIVE DENSITY = 30%)

$$\Delta H_i\ (\text{INCHES})/\text{KIP} = \frac{0.06112}{(B+1)^2}$$

FIGURE C-3 IS A SUMMARY OF INSTANTANEOUS SETTLEMENTS FOR PIER DIAMETERS OF 3' TO 8' AND PIER LOADS OF 0 TO 1600 KIPS

B LONGTERM SETTLEMENTS*

$$\Delta H = C_1 C_2 \Delta P \sum_{0}^{2B} \left(\frac{I_z}{E_s}\right) \Delta z$$

* FROM NAVFAC DM-7.1 SCHMERTMANN'S METHOD PAGES 220-222

WHERE:

ΔH = TOTAL LONG-TERM SETTLEMENT (INCHES)

$C_1 = 1 - 0.5(P_o/\Delta P)$; $C_1 \geq 0.5$

$C_2 = 1 + 0.2 \log(10t)$

P_o = OVERBURDEN PRESSURE AT FOUNDATION LEVEL (KSF)

ΔP = NET FOUNDATION PRESSURE INCREASE (KSF)

t = ELAPSED TIME IN YEARS

ASSUMPTIONS

AVE. DEPTH OF PIER TIP = 42 FOOT
AVE. SOIL DENSITY = 100 PCF
$E_s/N = 12$ FOR SANDS & GRAVELS ; $N = 100$ BLOWS
$t = 10$ YEARS

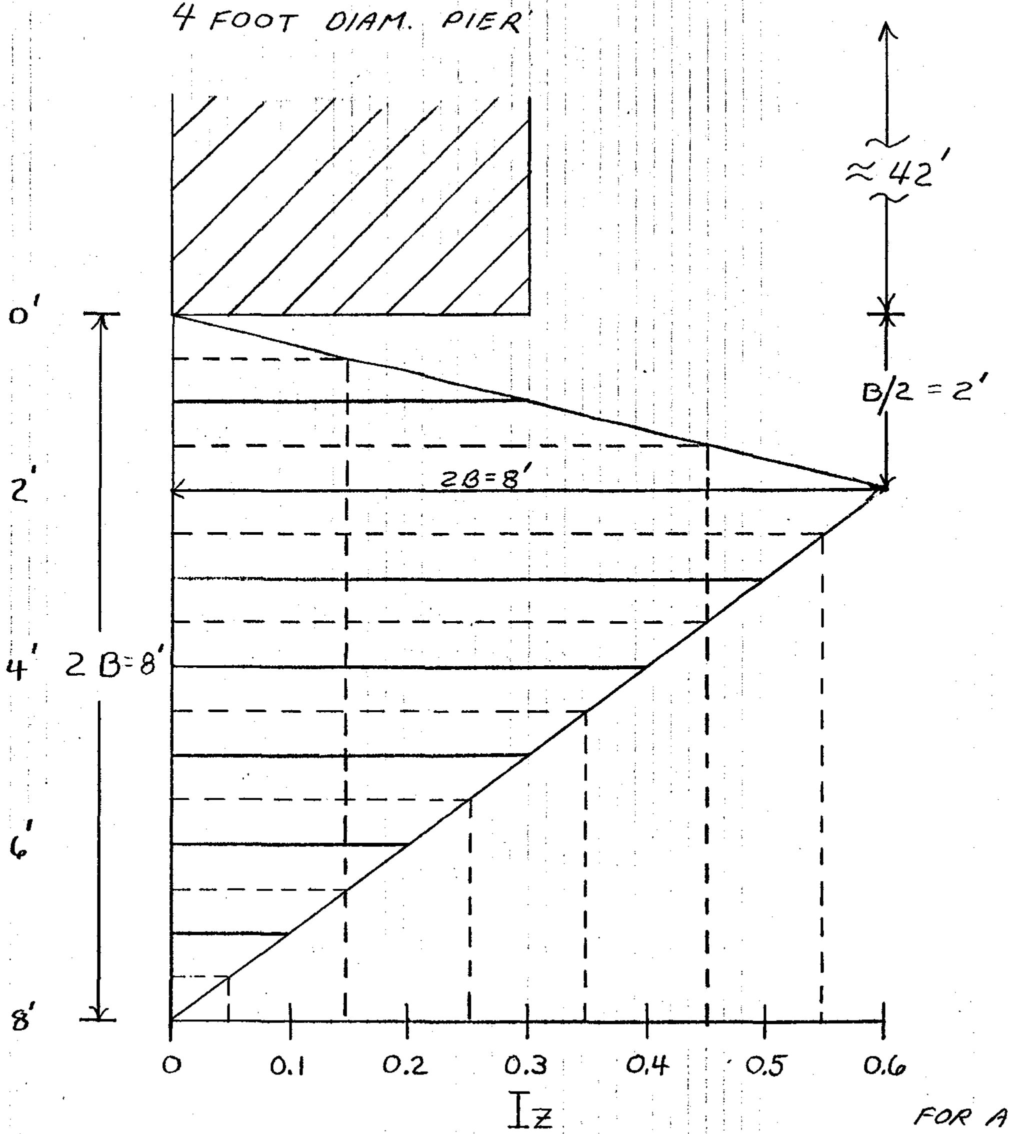
A SAMPLE CALCULATION IS PRESENTED

A LAYERED SYSTEM DOES NOT EXIST FOR A DEPTH OF 2B BELOW THE PIER TIP, HOWEVER ANALYSIS WAS COMPLETED USING ONE-FOOT INCREMENTS FOR INCREASED ACCURACY. THE SAMPLE CALCULATION IS FOR A 4-FOOT DIAMETER PIER CARRYING A 1000 KIP LOAD.

FIGURE C-4 IS A SUMMARY OF LONG-TERM SETTLEMENTS FOR PIER DIAMETERS OF 3 TO 8 FEET AND PIER LOADS OF 0 TO 1600 KIPS

FIGURE C-5 IS A SUMMARY OF TOTAL ESTIMATED SETTLEMENTS (THE SUM OF INSTANTANEOUS AND LONG-TERM SETTLEMENTS) FOR PIER DIAMETERS OF 3 TO 8 FEET AND PIER LOADS OF 0 TO 1600 KIPS.

SAMPLE CALCULATION - LONG TERM SETTLEMENTS



FOR A 1000 KIP LOAD:

LAYER	ΔZ (IN)	N	Es/N	Es (KSF)	Zc (IN.)	Iz	$\frac{I_z}{E_s}\Delta Z$ (IN/KSF)
1	12	100	12	2400	6	0.15	0.00075
2	12	100	12	2400	18	0.45	0.00225
3	12	100	12	2400	30	0.55	0.00275
4	12	100	12	2400	42	0.45	0.00225
5	12	100	12	2400	54	0.35	0.00175
6	12	100	12	2400	66	0.25	0.00125
7	12	100	12	2400	78	0.15	0.00075
8	12	100	12	2400	90	0.05	0.00025
							0.01200

$P_o = 42(100) = 4.20 \text{ KSF}$

$\Delta P = 1000/12.57 = 80 \text{ KSF}$

$C_1 = 1 - 0.5\left(\frac{4.2}{80}\right) = 0.97$

$C_2 = 1 + 0.2 \text{ LOG}(10 \cdot 10)$
$= 1.4$

$\Delta H = (0.97)(1.4)(80)(0.012)$
$= \underline{\underline{1.3''}}$

FIG. C-3 INSTANTANEOUS SETTLEMENT

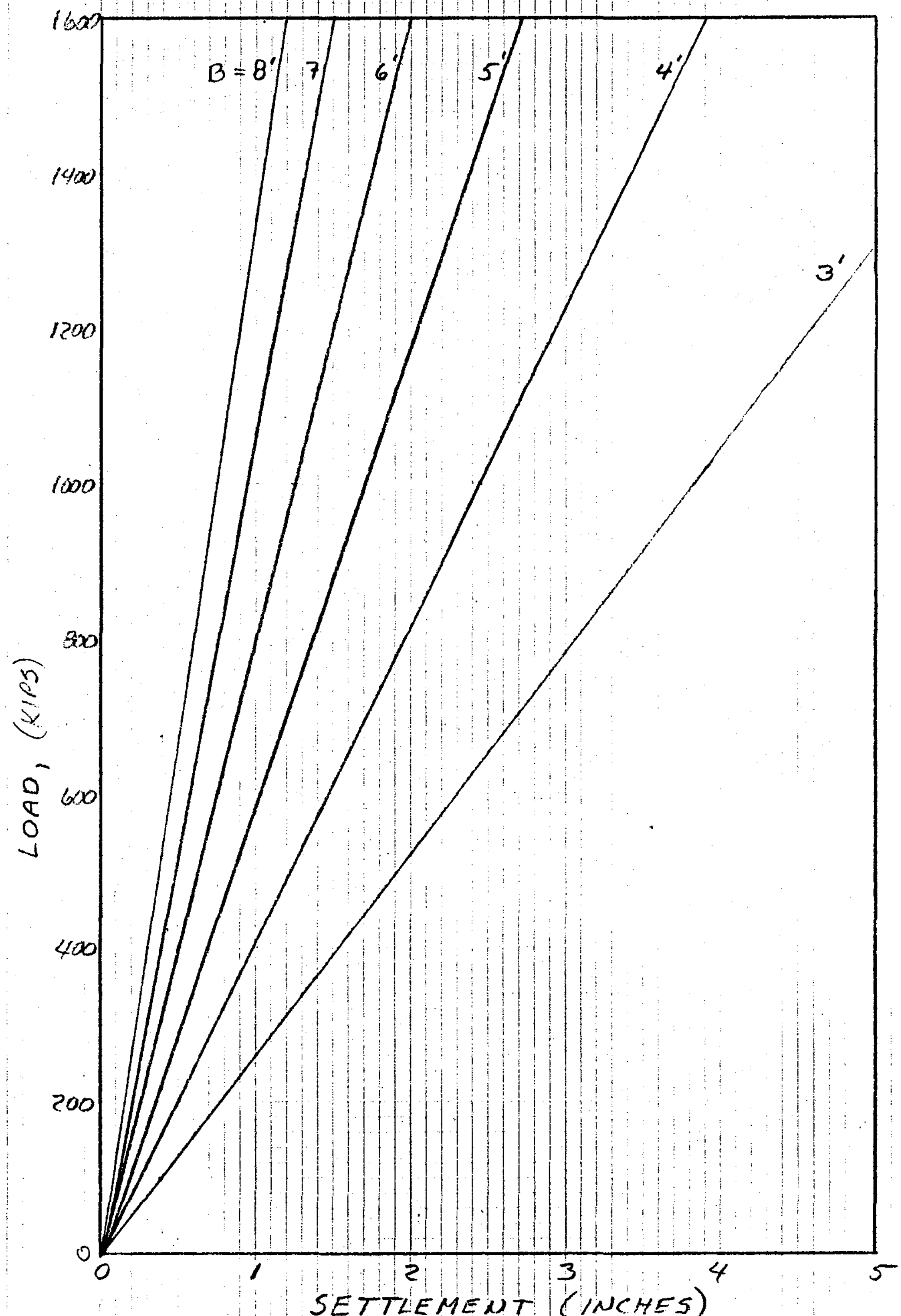


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FIG C-4 LONG-TERM SETTLEMENT

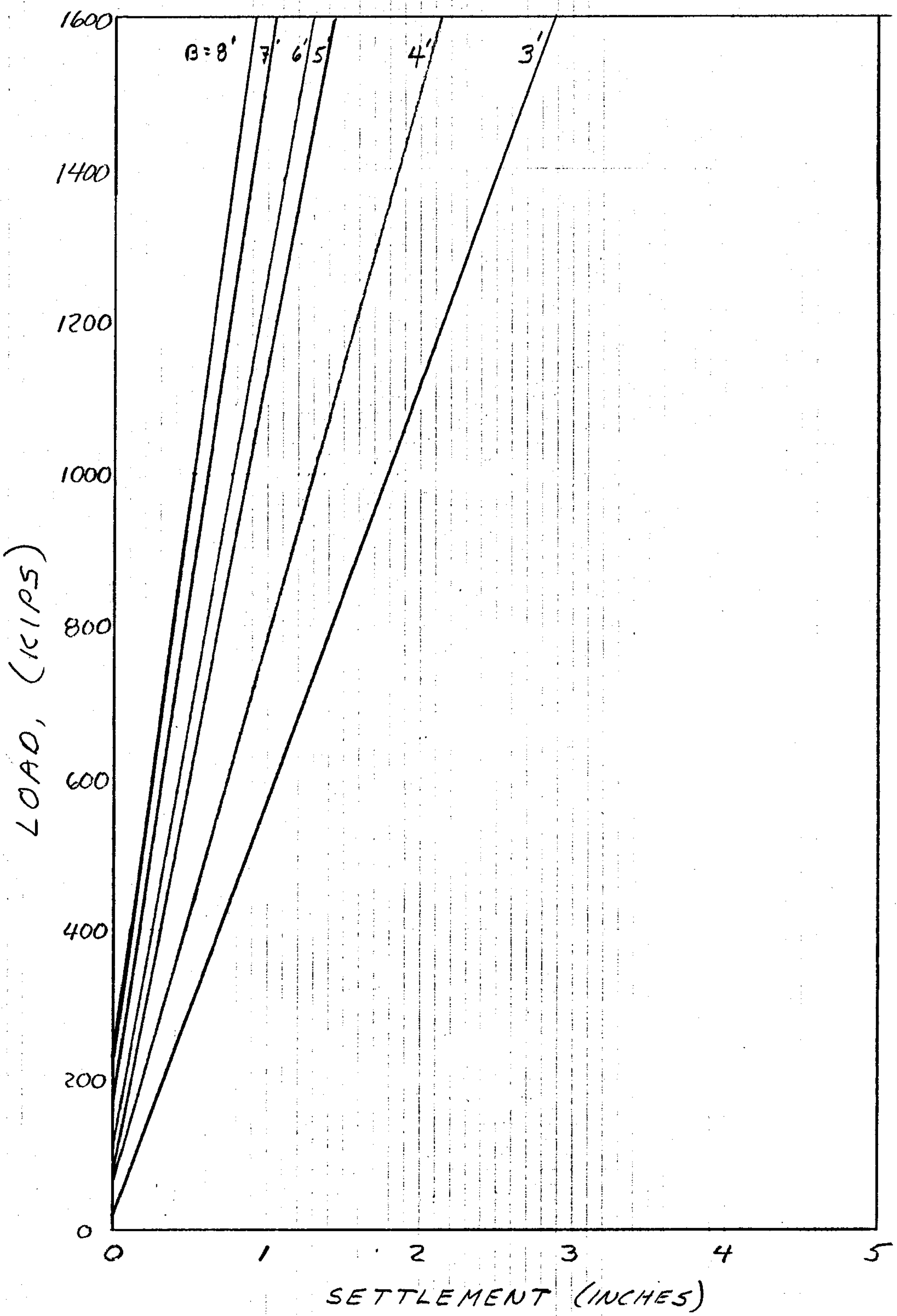

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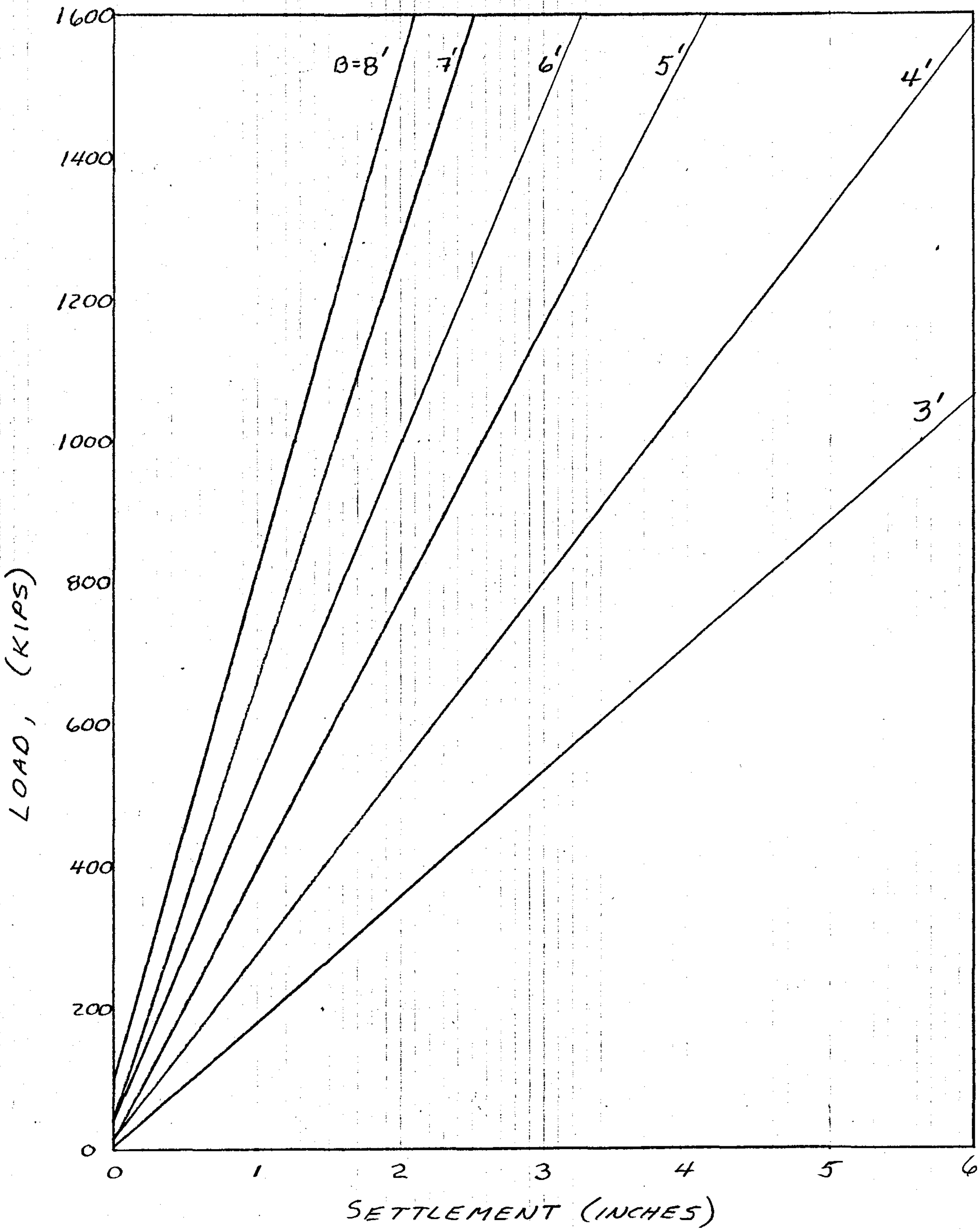
FIG. C-5 TOTAL SETTLEMENT



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ALTERNATE SETTLEMENT ANALYSIS FROM NAVFAC
DM 7.2
PG 207-209

A. CONSIDER END-BEARING ONLY

$$SETTLEMENT = Q_P\left(\frac{L}{AE_P} + \frac{C_P}{3Bq_s}\right)12$$

WHERE:

Q_P = POINT LOAD AT PIER TIP (KIPS)

L = PIER LENGTH (FT)

A = PIER CROSS SECTIONAL AREA (FT^2)

E_p = MODULUS OF ELASTICITY FOR PIER
FOR CONCRETE $E_p = 3\times10^6$ PSI = 4.32×10^5 KSF

C_p = EMPIRICAL COEFFICIENT
FOR DENSE SANDS = $C_p = 0.02$

B = PIER DIAMETER (FT)

q_o = ULTIMATE END BEARING CAPACITY = $3\times q_{SAFE}$

SAMPLE CALCULATION FOR ABUTMENT

4 FT DIAMETER PIER = B
1000 KIP LOAD = Q_P

$L = 50$ FT

$A = 12.57\ FT^2$

$C_p = 0.02$

$$SETTLEMENT = 1000\left[\left(\frac{50}{12.57(4.32\times10^5)}\right) + \frac{.02}{4(3\times894)}\right]12$$

$= 0.13$ INCHES

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B CONSIDER SIDE SHEAR ONLY

$$SETTLEMENT = Q_S \left(\frac{\alpha_S L}{A E_P} + \frac{C_S}{3 D g_S} \right) 12$$

WHERE:

Q_S = SHAFT FRICTION LOAD (KIPS)

L = PIER LENGTH (FT)

A = PIER CROSS SECTIONAL AREA (FT^2)

E_P = MODULUS OF ELASTICITY OF PIER
FOR CONCRETE = 3×10^6 PSI = 4.32×10^5 KSF

$C_S = (0.93 + 0.16\ D/B) C_P$

D = DEPTH OF EMBEDMENT (FT)

B = PIER DIAMETER (FT)

g_o = ULTIMATE CAPACITY BASED ON SIDE SHEAR (KIPS)

α_S = 0.5 FOR PARABOLIC DISTRIBUTION

SAMPLE CALCULATION FOR ABUTMENT

4 FT. DIAMETER PIER = B

1000 KIP LOAD = Q_S

$L = 50'$

$A = 12.57\ FT^2$

$C_P = 0.02$

$$SETTLEMENT = 1000 \left[\left(\frac{0.5 (50)}{12.57 (4.32 \times 10^5)} \right) + \left(\frac{(0.93 + 0.16 (50/4)).02}{3 (50)(343)} \right) \right] 12$$

= .07 INCHES

TABLES C-5 THROUGH C-8 PRESENT SETTLEMENTS IN INCHES/KIP $\times 10^{-4}$ FOR ABUTMENT AND CHANNEL PIERS BASED ON END BEARING AND SIDE SHEAR.

TABLE C-5 SETTLEMENT IN INCHES/KIP $\times 10^{-4}$ FOR ABUTMENTS BASED ON END BEARING ONLY

PIER TIP ELEV FT	DEPTH OF EMBED. FT	SETTLEMENT INCHES/KIP $\times 10^{-4}$					
		B = 3'	4'	5'	6'	7'	8'
1170	10	2.84	1.60	1.02	0.71	0.52	0.40
1160	20	1.47	0.83	0.53	0.37	0.27	0.21
1150	30	1.68	0.95	0.61	0.42	0.31	0.24
1140	40	2.10	1.18	0.76	0.53	0.39	0.30
1130	50	2.36	1.33	0.85	0.59	0.43	0.33
1120	60	2.85	1.60	1.03	0.71	0.52	0.40
1110	70	3.14	1.76	1.13	0.78	0.58	0.44
1100	80	3.48	1.95	1.25	0.87	0.64	0.49
1090	90	3.84	2.16	1.38	0.96	0.71	0.54
1080	100	4.21	2.37	1.51	1.05	0.77	0.59
1070	110	4.59	2.58	1.65	1.15	0.84	0.65
1060	120	4.98	2.80	1.79	1.25	0.92	0.70
1050	130	5.37	3.02	1.93	1.34	0.99	0.76

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TABLE C-6 SETTLEMENT IN INCHES/KIP $\times 10^{-4}$ FOR ABUTMENTS BASED ON SIDE SHEAR ONLY.

PIER TIP ELEV. FT.	DEPTH OF EMBED FT.	SETTLEMENT, INCHES/KIP $\times 10^{-4}$					
		B = 3'	4'	5'	6'	7'	8'
1170	10	4.70	3.15	2.34	1.86	1.53	1.32
1160	20	1.29	0.80	0.56	0.43	0.34	0.28
1150	30	1.04	0.67	0.42	0.30	0.23	0.19
1140	40	1.09	0.63	0.42	0.30	0.22	0.18
1130	50	1.21	0.69	0.49	0.32	0.24	0.18
1120	60	1.36	0.78	0.50	0.35	0.26	0.20
1110	70	1.53	0.87	0.56	0.39	0.29	0.22
1100	80	1.70	0.96	0.62	0.43	0.32	0.24
1090	90	1.88	1.06	0.68	0.47	0.35	0.27
1080	100	2.06	1.16	0.75	0.52	0.38	0.29
1070	110	2.25	1.27	0.81	0.56	0.42	0.32
1060	120	2.43	1.37	0.88	0.61	0.45	0.34
1050	130	2.62	1.48	0.95	0.66	0.48	0.37

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TABLE C-7 SETTLEMENT IN INCHES/KIP $\times 10^{-4}$ FOR CHANNEL PIERS BASED ON END BEARING ONLY.

PIER TIP ELEV. FT	DEPTH OF EMBED. FT	SETTLEMENT INCHES/KIP $\times 10^{-4}$					
		B = 3'	4'	5'	6'	7'	8'
1150	10	1.91	1.07	0.69	0.48	0.35	0.27
1140	20	1.85	1.04	0.67	0.46	0.34	0.26
1130	30	1.84	1.04	0.66	0.46	0.34	0.26
1120	40	2.31	1.30	0.83	0.58	0.43	0.33
1110	50	2.51	1.41	0.90	0.63	0.46	0.35
1100	60	2.80	1.58	1.01	0.70	0.51	0.39
1090	70	3.15	1.77	1.13	0.79	0.58	0.44
1080	80	3.49	1.96	1.26	0.87	0.64	0.49
1070	90	3.87	2.17	1.39	0.97	0.71	0.54
1060	100	4.25	2.39	1.53	1.06	0.78	0.60
1050	110	4.63	2.60	1.67	1.16	0.85	0.65

TABLE C-8 SETTLEMENT IN INCHES/KIP x10⁻⁴ FOR CHANNEL PIERS BASED ON SIDE SHEAR ONLY.

PIER TIP ELEV FT	DEPTH OF EMBED FT	SETTLEMENT, INCHES/KIP x10⁻⁴					
		B=3'	4'	5'	6'	7'	8'
1150	10	2.15	1.44	1.07	0.85	0.70	0.60
1140	20	1.11	0.69	0.48	0.36	0.29	0.23
1130	30	0.99	0.58	0.39	0.28	0.22	0.17
1120	40	1.07	0.62	0.41	0.29	0.22	0.17
1110	50	1.20	0.68	0.45	0.31	0.23	0.18
1100	60	1.35	0.77	0.50	0.35	0.26	0.20
1090	70	1.52	0.86	0.55	0.39	0.29	0.22
1080	80	1.69	0.95	0.61	0.43	0.32	0.24
1070	90	1.87	1.05	0.68	0.47	0.35	0.27
1060	100	2.05	1.16	0.74	0.52	0.38	0.29
1050	110	2.24	1.26	0.81	0.56	0.41	0.32

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330

15

300

$n = .035$
$S = .0019$
$b = 300$
1:1

$Y_n = 12.08$ ft
$V_n = 9.29$ f/s
$Y_c = 7.29$
$F = .48$
$R = 11.28$

SCOUR ANALYSIS

GIVEN DATA

DESIGN DISCHARGE $Q = 35{,}000$ CFS

DISCHARGE/UNIT WIDTH $q = 35{,}000/375 = 93.33$ CFS/FT

MEAN VELOCITY $V_m = 11$ FT/SEC

MEAN DEPTH $Y_0 = 11$ FT

MEAN PARTICLE SIZE $\boxed{D_m = 1.5 \text{ mm}}$

USING ANALYSIS BY NEILL (1973) FOR GENERAL SCOUR

GENERAL SCOUR

$$d_m = 0.47\left(\frac{Q}{f}\right)^{1/3}$$

WHERE d_m = MEAN DEPTH

$$= 0.47\left(\frac{35{,}000}{2.16}\right)^{1/3}$$

$$= 11.9$$

$$f = 1.76\,(D_m)^{1/2}$$

$$= 1.76\,(1.5)^{1/2}$$

$$= 2.16$$

$$d_s = d_m\left(V_m/V_c - 1\right)$$

WHERE $V_c = 4.0$ FT/SEC

$$= 11.9\left(\frac{11}{4} - 1\right)$$

$$= 20.83'$$

USE 20 FEET FOR DESIGN

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LOCAL SCOUR

LOCAL SCOUR HAS BEEN ESTIMATED USING 8 METHODS. EQUATIONS FOR THESE METHODS FOLLOW:

BLENCH (1969)

$$d_s = D\left[\left(\frac{0.83}{D_M^{1/6}}\right)^{3/4} g^{1/4} \left(\frac{y_0}{D}\right)^{3/4} F_0^{1/2} - \frac{y_0}{D}\right]$$

INGLIS - POONA (1962)

$$d_s = D\left[1.70\, g^{1/4} F_0^{1/2} \left(\frac{y_0}{D}\right)^{3/4} - \frac{y_0}{D}\right]$$

LAURSEN II (1962)

$$D/y_0 = 5.5 \frac{d_s}{y_0}\left[\left(\frac{1}{11.5}\frac{d_s}{y_0} + 1\right)^{1.7} - 1\right]$$

SOLUTION BY TRIAL AND ERROR

NEILL (1964)

$$d_s = D\left[1.5\left(\frac{y_0}{D}\right)^{0.3}\right]$$

SHEN I (1969)

$$d_s = 1.39\, g^{1/3} F_0^{2/3} D^{2/3} y_0^{1/3}$$

SHEN II (1969)

$$d_s = D\left[3.4\, F_0^{2/3}\left(\frac{y_0}{D}\right)^{1/3}\right]$$

FWHA DESIGN MANUAL (1975)

$$d_s = D\left[2.0\, F_0^{0.43}\left(\frac{y_0}{D}\right)^{0.35}\right]$$

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62.4 2.65 - 1
$\gamma_w (g_s - g_w)$

5 + 4 = 9

3 x 5 = 18

5 + 6 = 11

11

(2.5x)(9) = 22.5

2.5x15 = 37

LAURSEN (1980)

τ_c = CRITICAL TRACTIVE FORCE

$= 0.047 (\gamma_s - \gamma_w) D_M$

$= 0.047 (110 - 62.4)(1.5mm/25.4/12)$

165

$= 1.5876 \ lb/FT^2$

τ_o' = SEDIMENT-TRANSPORT TRACTIVE FORCE

$$= \frac{V_m^2 D_m^{1/3}}{30 y_o^{1/3}} = \frac{11^2 (1.5/25.4/12)^{1/3}}{30 \ (11)^{1/3}} = 0.3085 \ lb/FT^2$$

$$\tau_c/\tau_o' = \frac{1.5876}{0.3085} = 5.1462 > 1$$

$F_0 = 0.5845 \longrightarrow$ SUBCRITICAL FLOW

FOR $l = 1/2 D$

$$\frac{D}{2y_o} = 2.75 \frac{d_s}{y_o} \left[\left(\frac{1}{11.5} \frac{d_s}{y_o} + 1 \right)^{1.69} - 1 \right]$$

SOLUTION BY TRIAL AND ERROR

IMPLEMENTING ESTIMATED GENERAL AND LOCAL SCOUR, TOTAL DESIGN SCOUR IS PRESTED IN TABLE C-9

TABLE C-9

SCOUR (FT)	B = 3'	4'	5'	6'	7'	8'
GENERAL	20	20	20	20	20	20
LOCAL	9.53	11.25	12.78	14.18	15.48	16.70
TOTAL	29.53	31.25	32.78	34.18	35.48	36.70

NOTE: THIS TABLE PRESENTS THE AVERAGED VALUES TO BE USED FOR DESIGN. ANY METHOD PREFERENCE WILL SUBSTANTIALLY EFFECT THESE VALUES.

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

1. Scope of Work

These specifications cover the requirements for slurry-assisted construction of drilled, cast-in-place concrete piers. The requirements include slurry material quality, preparation, handling and placement of slurry drilled pier excavation, steel placement and concrete placement, and quality assurance and nondestructive testing procedures.

2. Submittals

The following submittals shall be made by the Contractor:

A. A list of personnel who will be committed to the construction project, and their experience.

B. A list of drilling equipment and tools, bentonite mixing and cleaning equipment, and pumping equipment to be utilized during construction which includes equipment name and specifications.

C. A summary of the procedures to be employed in drilled pier construction and any special techniques to be incorporated.

D. Certificate of compliance with the quality requirements and standards and testing methods specified herein for commercial grade bentonite.

E. Mix design for Portland Cement concrete.

F. Shop drawings for reinforcing cages, and geophysical access tubes and a description for procedures for placement an securing of reinforcing cages and access tubes to maintain their alignment during construction.

3. Quality Assurance

3.1 Quality Control

The Engineer may perform quality assurance tests to provide an independent assessment of the quality of the work. However, the quality assurance observations and testing will not relieve the Contractor of his requirement to perform quality control testing during the course of work or to complete the work in accordance with these specifications.

3.2 Construction Quality Assurance

3.2.1 Independent observations and testing shall be performed by the Engineer under separate contract, and shall be done at no expense to the Contractor.

3.2.2 Prior to construction, the Engineer will review the submittals required under Section 2.

3.2.3 Observation of the construction of the drilled piers, including excavation, slurry placement, steel and concrete placement, will be made by the Engineer.

3.2.4 Should the Engineer have reason to believe that the construction techniques, sequence of operations, or workmanship has been deficient for a given pier, so that the integrity of the pier under operating conditions, is questionable, the Contractor shall be so notified.

3.2.5 The following tasks will be performed by the Engineer during and after drilling and concreting operations as the basis for evaluation of the drilled pier installations:

A. Providing a written chronology of events during the drilling of piers and the placement of steel and concrete.

B. Verifying design depths are reached in drilling as indicated on the plans, that proper cleaning of the excavations is done and that the slurry in maintained within specifications throughout excavation, interruptions and concreting.

C. Verifying concrete delivered to the site meets specifications for consistency and pumpability.

D. Verifying that a positive head of concrete above the bottom of the tremie pipe is maintained at all times during concrete pumping operations.

E. Monitoring volume of concrete placed in the excavations in relationship to depth.

F. Nondestructive testing of the completed piers by the gamma ray backscattering method.

3.3 Measurement for Plumbness

Prior to placement of concrete, the plumbness of the reinforcing cage will be measured by inclinometer survey. To provide access tubes for the inclinometer, there shall be securely attached to the rebar cage, a 3- or 4-inch diameter PVC tubing which is slotted along the bottom 10 feet.

3.4 Nondestructive Testing

Nondestructive testing of finished concrete piers will be accomplished by means of geophysical techniques involving gamma ray scattering to investigate continuity. This will be accomplished by the installation of 2-inch I.D. PVC pipe (Schedule 40) or steel pipe axially downward the shaft of the piers. Such pipe must be tied to the reinforcing cage and capped at the bottom. A minimum of four pipes shall be used for shafts up to 5.0 feet in diameter and a minimum of six pipes for shafts greater than 5.0 feet in diameter.

3.5 Rejection of Piers & Requirements for Remedial Action

Judgement of the acceptability of the drilled piers will be made by the Engineer, based upon visual observations of the construction sequence and gamma ray inspection. If, in the judgement of the Engineer, evidence indicates that the pier is not structurally adequate, the pier shall be rejected and, where appropriate, construction of additional piers shall be suspended. Such rejection shall prevail until the Contractor, at his expense, repairs, replaces or supplements the defective pier and the Engineer approves the remedial work. Suspension of pier construction shall remain in effect until corrections in the methods of construction are made to the satisfaction of the Engineer.

4. Materials

4.1 Bentonite Slurry

4.1.1 The slurry shall consist of a stable suspension of commercial grade bentonite with physical and chemical properties in accordance with the requirements of American Petroleum Institute (API) Specification 13A, latest edition.

4.1.2 Water for mixing shall be the quality of drinking water with respect to soluble salts content. Bacterial contamination will be acceptable only upon written approval of the Engineer.

4.1.3 Water in which the chemical quality will permit flocculation of the bentonite shall not be used.

4.1.4 The slurry shall be stirred or agitated prior to use, so as to maintain a uniform consistency and viscosity.

4.1.5 The slurry properties shall be in conformance with the requirements of Table 1.

4.2 <u>Concrete</u>

The concrete shall be in conformance with the Standard Specifications for Concrete Materials and Concrete for this project with the following additional requirements:

4.2.1 Portland Cement concrete shall be of an exceptionally rich, pumpable mixture, which will settle under its own weight and will not have a tendency to mix with the slurry.

4.2.2 Concrete shall not contain less than 650 pounds of cement per cubic yard.

4.2.3 The concrete shall have a slump in the range of 7 to 9 inches when tested by ASTM C143.

4.2.4 The maximum size of coarse aggregate shall be 3/4 inch.

TABLE 1

Requirement for Bentonite Slurry

Slurry Property	Test Method	Acceptable Range: During Excavation	During Interruptions	During Concreting
Bentonite Concentration i.e. (wt. bent/ wt. water)	n/a	4 1/2%	4 1/2%	4 1/2 - 15%
Density	ASTM D4380-84 Mud Balance Density	64-69 pcf	64-70 pcf	64-75 pcf
Sand Content (% by volume)	ASTM D4381-84	4-5	5-10	15 max.
API Fluid Loss (ml in 30 min.)	API 13A	20	20	40
Viscosity:				
(sec)	API Marsh Cone	30-90	3 min.	90 max.
(cp)	Fann Viscometer	10 max.	4-10	20 max.
pH	pH meter or indicator paper	9-12	9-12	n/a
Shear Strength: (10 min. gel strength)	psf, Fann Viscometer	0.06 min. 0.20 max.	0.15 min.	0.40 max.

4.2.5 Admixtures as needed, to prevent segregation of the mix, allow free flow through the placing equipment, and to retard settling during hot weather, shall be added by the Contractor only upon written approval of the Engineer.

5. HANDLING & STORAGE OF BENTONITE SLURRY

5.1 Bentonite slurry shall be handled and stored in such a manner as to prevent deterioration or intrusion of foreign matter.

5.2 Bentonite slurry shall be handled and stored so as to produce a minimum amount of segregation.

5.3 Slurry may be stored in earth basins or tanks which allow easy measurement of the slurry mix.

6. MIXES

6.1 Bentonite Slurry Mix

6.1.1 The bentonite slurry shall be stirred or agitated prior to use with a slurry pump, drum mixer or other mechanical mixing device approved by the Engineer.

6.1.2 The slurry shall be thoroughly mixed and be free of lumps.

6.1.3 The practice of "mudding," that is, the dropping of a sack of dry bentonite into an open hole to be mixed by the action of the auger, shall be considered unacceptable a a standard procedure but may be used on occasion with the approval of the Engineer to provide higher density to the slurry.

6.1.4 Slurry shall be first introduced where an unacceptable amount of caving of the side of the excavation is experienced, or where free-flowing or standing water is encountered, whichever is first.

6.1.5 The level of bentonite slurry shall be maintained 4.0 feet or more above the level where unacceptable caving is encountered or of standing groundwater or free flowing water, whichever is first.

6.1.6 In the event that a sudden loss in bentonite is experienced, followed by caving, the boring shall be backfilled immediately and instructions from the Engineer sought.

6.1.7 It shall be verified by observations and measurement that excavations are open to the specified depths.

6.1.8 Where a completed excavation containing slurry is left open overnight prior to placement of concrete, a probe shall be lowered to measure the amount of caving materials or settling of slurry which has taken place. Where more than 3 inches of material has settled, additional passes of the auger or bucket shall be made to clean the excavation, until there is no more than 3 inches of loose material at the base.

6.2 Concrete Mixing

Mixing of concrete shall be in conformance with the specifications outlined in the Specifications for Concrete Materials and Concrete.

7. DRILLED PIER CONSTRUCTION

7.1 Slurry-Assisted Excavation

7.1.1 Straight, drilled pier excavations shall be advanced with approved drilling tools to depths indicated by the plans.

7.1.2 Excavations shall be advanced so the axis does not exceed specified tolerances.

7.1.3 Caving of the hole shall be prevented at all times by use of a bentonite slurry.

7.1.4 Pressure relief holes shall be provided in the sides of cutters to ensure that wall erosion or additional caving is not induced during travel of buckets or bits.

7.1.5 Properties of the slurry shall be maintained within the specified limits given on Table 1.

7.2 Cleaning of Slurry

7.2.1 The slurry shall be cleaned so as to separate the slurry from the soil particles introduced during the excavation process to the extent that the slurry properties are maintained within the specified limits.

7.2.2 Slurry cleaning can be performed by sedimentation, a vibrating screen, a cyclone, or a combination thereof.

7.3 <u>Placement of Reinforcing Steel</u>

7.3.1 The steel reinforcing cage shall be completely formed at the surface and lowered in one continuous operation with a crane of sufficient capacity.

7.3.2 The steel cage shall be constructed so as to provide clearances as shown on the construction drawings.

7.3.3 The rebar cage shall be supported from the top by a ground surface frame, or other positive means to ensure cage plumbness and to prevent downward slumping.

7.3.4 A minimum clearance between the reinforcing steel and the walls of the excavation, as shown on the construction drawings, shall be provided. At least 6 inches of clearance between the reinforcing steel and the walls of the excavation shall be provided. This shall be accomplished by the use of spacer blocks which are firmly tied to the reinforcing cage so as not to become disconnected or disoriented during lowering of the cage into the hole.

7.3.5 The steel cage shall be constructed so as to provide a minimum of 6 inches between vertical bars and a minimum of 12 inches between horizontal ties.

7.4 <u>Placement of Concrete</u>

7.4.1 Concrete shall be placed as soon as possible after completion of the excavation and immediately after placement of the reinforcing steel and verification of plumbness by inclinometer surveys.

7.4.2 Concrete shall be placed by pumping through a steel tremie pipe. No free fall of the concrete will be allowed.

7.4.3 The steel tremie pipe shall be rigid, watertight and not be less than 6 inches in inside diameter.

7.4.4 The tremie pipe shall be equipped with a bottom valve, or other approved device which will prevent mixing of the slurry with the concrete inside the pipe and prevent the intrusion of slurry into the concrete in the event that the tremie pipe is removed and replaced.

7.4.5 The pump utilized shall be capable of pumping 60 cubic yards per hour, a vertical height of 200 feet.

7.4.6 A backup pump shall be provided by the Contractor during concreting operations on-site, unless assurance of delivery of a second pump to the site within 30 minutes is provided.

7.4.7 Reinforcing steel shall be in-place and the tremie pipe inserted to the bottom of the hole prior to concrete placement.

7.4.8 Concrete shall be placed in a continuous operation in such a manner that the concrete always flows upward within the hole.

7.4.9 The delivery pipe shall be slowly withdrawn as the elevation of the concrete in the hole rises, but the discharge end of the pipe shall, at all times, be maintained at least 5.0 feet below the surface of the concrete.

7.4.10 Raising of the tremie pipe shall be done only when the pipe contains a sufficient head of concrete to prevent the formation of a void at the top. A predetermined plan shall be formulated between the Contractor's foreman and the pump operator concerning how and when an order will be given to lift the tremie pipe.

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