NEW RIVER BRIDGE

660 West of 99th Avenue
Peoria/Glendale, Arizona

Registered Professional Engineer
CERTIFICATE NO.
10971
KENNETH L.
RICKER
Date Signed 12-8-89
ARIZONA, U.S.A.

Prepared for

BRW, Inc.
2700 N. Central Avenue
Suite 1000
Phoenix, Arizona 85004

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

sla

SIMONS, LI & ASSOCIATES, INC.

4600 S. MILL AVENUE, SUITE 280
TEMPE, ARIZONA 85282
TELEPHONE (602) 491-1393

Dennis L. Richards, P.E.
Assistant Vice President

DEC 13 1989

December 11, 1989

Mr. Lee Arnold
BRW, Inc.
2700 North Central Avenue
Suite 1000
Phoenix, AZ 85004

RE: FLOOD CONTROL DISTRICT REVIEW COMMENTS ON NORTHERN AVENUE BRIDGE OVER NEW RIVER

Dear Lee:

This is in response to our telephone conversation regarding the November 16, 1989, Interoffice Memorandum you received from the Flood Control District of Maricopa County regarding the Northern Avenue Bridge over New River. Comment No. 1 requests that a more accurate material sampling be obtained in order to confirm that the scour depth does not exceed the limits estimated.

Local scour at bridge piers was estimated using the CSU equation. This equation, along with many of the local-scour equations referenced in the literature, does not require sediment sizes for determining local scour. The CSU equation has been widely accepted, and is the equation recommended by the Federal Highway Administration (FHWA) in Technical Advisory T 5140.20, dated September 16, 1988, and titled "Scour at Bridges." It is also the equation for estimating local scour included in the publication "Highways in the River Environment," dated May 1975, and updated in 1988. While some local-scour equations do include sediment size as a parameter, it is our professional judgement (and that of others in the field) that the derivations of such equations are based upon insufficient data to justify their use in an alluvial channel such as the New River.

Of course, sediment sizes would be required for analyzing general scour/deposition and long-term channel response. However, analyses of these scour components was not part of our Scope of Work. Rather, our Scope of Work was merely to estimate local scour, as well as to review/verify the sediment-transport analyses for general scour/deposition, and other scour components, as determined by Coe & Van Loo in a previous study effort.

If you have any questions or need additional information regarding scour estimates, please call.

Sincerely,

SIMONS, LI & ASSOCIATES, INC.

Dennis L. Richards

Dennis L. Richards, P.E.
Vice President

THOMAS-HARTIG & ASSOCIATES, INC.
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Geotechnical, Materials Testing, and Environmental Consultants
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Frank M. Guerra, P.E.
Steven A. Haire, P.E.
Kenneth L. Ricker, P.E.

BRW, Inc.
2700 N. Central Avenue
Suite 1000
Phoenix, Arizona 85004

8 December 1989

Attention: Ralph Blum

Project: New River Bridge
660 West of 99th Avenue
Peoria/Glendale, Arizona

Project No. 89-0747

This report presents the results of the geotechnical engineering services authorized for the New River Bridge located approximately 660 West of 99th Avenue, in Peoria/Glendale, Arizona. The purpose of these services is to determine the soil conditions at the locations indicated which thereby provide a basis for the design discussions and recommendations presented herein. This firm should be notified for evaluation if conditions other than described herein are encountered during construction.

The services performed provide an evaluation at selected locations of the soils throughout the zone of significant foundation influence. Our field services have not included exploration for underlying geologic conditions or evaluation of potential geologic hazards such as seismic activity, faulting, and ground subsidence/cracking potential due to groundwater withdrawal, or the presence of contamination.

The recommendations presented in this report are based upon the project information received and described in "Scope" Part I. This firm should be contacted for review if the design conditions are changed substantially.

If requested, we will be available to review project plans and specifications relative to compliance to the intent of this report.

Respectfully submitted,

THOMAS-HARTIG & ASSOCIATES, INC.

Registered Professional Engineer
CERTIFICATE NO. 10971
KENNETH L. RICKER
Date Signed 12-8-89
ARIZONA, U.S.A.

By: Kenneth L. Ricker, P.E.

Reviewed by: Harry E. Hartig, P.E.

Registered Professional Engineer
CERTIFICATE NO. 5641
HARRY E. HARTIG
Date Signed 12-8-89
ARIZONA, U.S.A.

/go

Copies to: Addressee (5)

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TABLE OF CONTENTS

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10971
KENNETH L.
RICKER
Date Signed 12-8-89
ARIZONA, U.S.A.

PART 1
REPORT

SCOPE

The proposed New River Bridge on Northern Avenue will replace a low water crossing located approximately 660 west of 99th Avenue, in Peoria/Glendale, Arizona. We understand the bridge will be 388 feet long, 81 feet wide, and have 4 spans. The bridge will be constructed of reinforced concrete and supported on 6 or 6 1/2 foot diameter drilled shafts with 3 or 4 shafts per pier bent. The bridge will cross a channelized portion of the New River. The river channel will have soil cement sides slopes and a bottom elevation of 1058.25 feet. Maximum water levels will be at Elevation 1069.37 feet for the 100 year flood. Maximum loads at the bottom of the scour depth have been estimated by BRW, Inc. at 1206 to 1608 kips per drilled shaft for the bridge piers and 892 kips per drilled shaft for abutments. The maximum local scour depth has been estimated by Simons-Li & Associates at approximately 31.1 feet below the proposed channel bottom. Roadway approaches to the bridge will require fill depths of up to 12 feet.

SITE DESCRIPTION

The bridge will be located on Northern Avenue at the New River approximately 660 feet west of 99th Avenue in Peoria/Glendale, Arizona (see site plan). The existing low water crossing of the New River consisted of a two lane asphalt concrete roadway with a dirt shoulder on the north side, and a concrete shoulder on the south side. The channel surface north of the roadway was at to slightly above roadway grade while the channel surface south of the roadway was approximately 3 to 4 feet below roadway grade. The proposed east abutment of the bridge was in the bottom of the existing channel. The proposed west abutment of the bridge is at the top of existing channel bank. The existing roadway was cut into the channel side slopes. Numerous cobble and boulder size materials exist on the surface.

INVESTIGATION

Test borings were drilled at five (5) locations along the bridge alignment. The test borings were drilled with 7 inch diameter hollow stem augers using a CME 55 drill rig. However, shallow refusal occurred in all test borings advanced with augers. Two of the test borings were then extended to depths of 75 feet using 9 inch outside diameter double wall drill pipe advanced with a truck-mounted Drill Systems AP1000 percussion hammer drill equiped with a diesel pile hammer. During test drilling encountered soils were visually classified, and representative soil samples were obtained at selected depths. Sampling was accomplished with the augers full of water. The results of the test drilling are presented in Appendix

A, "Field Results".

Representative samples obtained during the test drilling were subjected to the following laboratory analyses:

Test	Sample(s)	Purpose
Direct Shear	Remolded (6)	Bearing capacity analysis
Sieve Analysis & Plasticity Index	Representative Soil (7)	Soil classification to correlate engineering properties
Moisture Content *	Split Spoon (20)	In-site moisture determination to correlate engineering properties

* Reported on boring logs

The results of testing are presented in Appendix B, "Laboratory Results".

SOIL CONDITIONS

As disclosed by the test borings and illustrated on the attached boring logs, the soil profile encountered at the test boring locations is somewhat variable. Granular soil deposits were encountered for the full depth of exploration (11 to 75 feet), except at Test Boring 5 where a 7 foot thick sandy clay layer caps the granular soils. The upper 9 to 12 feet of the granular deposits consisted of relatively clean sands and gravels containing some cobble size materials. These surficial granular soils were underlain by dense to very dense interbedded and stratified deposits of clayey sand and gravel, silty sandy gravels, and sands and gravels. These deposits contained various amounts of cobble size materials and occasional layers of dense to very dense gravely sands with some clay and sands with some silt and gravel. The boring logs presented in Appendix A provide a more detailed description of soil profiles.

Soil moisture contents were described as slightly damp to damp. No groundwater was encountered in any of the test borings at the time of exploration. Refusal to auger penetration occurred in all test borings drilled with hollow stem augers (1 to 5) at depths ranging from 11 to 21 feet.

DISCUSSION AND RECOMMENDATIONS

General: Geotechnical engineering recommendations are presented in the following sections. These recommendations are based upon the design procedures presented in

NAVFAC DM 7.2 and the results of the field and laboratory testing which are presented in Appendices A and B of this report. Alternative recommendations may be possible and will be considered upon request. The soil parameter used in design are based on the standard penetration test blow counts, remolded direct shear tests results performed on minus No. 4 sieve size material, and correlation of field data with soil parameters as presented in NAVFAC DM 7.1. The engineering analysis assumed that no material or lateral soil support occurs above the scour level and the soils below sour level are submerged. An idealized soil profile based on the boring logs was used. The design loads, soil parameters and engineering analysis are presented in Appendix C.

Drilled Shafts: Circular drilled straight shaft cast-in-place piers were analyzed for foundation support. At the request of BRW, Inc. allowable drilled shaft capacities for 6 and 6 1/2 foot diameter shafts were computed and are presented below. These capacities are based on end bearing only. Drilled shaft capacities for other diameters can be analyzed, if desired. No reduction for group action is indicated if shafts are spaced on centers exceeding 3.0 times the shaft diameter.

Although groundwater was not observed during field exploration, a perched groundwater condition may be encountered during construction. Underwater concrete placement techniques may be required.

Allowable Drilled Shafts Capacity

*Depth (Feet)	6 Foot Diameter	6 1/2 Foot Diameter
20	688	807
25	860	1009
30	1032	1211
35	1204	1413
40	1377	1615
45	1549	-
47	1617	-

* As measured below maximum scour level

It is our opinion that the soil parameters and groundwater conditions assumed for the very dense very coarse grained granular soil encountered in the test borings are conservative. Therefore, a factor of safety of 2.5 was applied to the ultimate shaft capacities.

Total settlements for the allowable drilled shaft capacities applied to the design dead loads were computed to be 0.72 to 0.89 inches. Based on the soil and groundwater conditions encountered and our past experience with similar projects, it is our opinion that actual settlements will occur during the application of the dead load. Differential settlement between drilled shafts are estimated to be less than one half the total settlement.

Approach Fill: The following procedure is presented for development of the approach fills:

1. Strip and remove all dumped or spread fill zones, any organic or debris materials, and any obviously loose surface soils.
2. Clean and widen any depressions.
3. Scarify to a minimum depth of 10 inches, moisture condition, and compact all cleaned subgrade.
4. Place all required fills in lifts of a thickness compatible with the compaction equipment used.

Compaction of all soil should be accomplished to a minimum 95 percent of the ASTM D698 at a moisture range of optimum ± 3 percent.

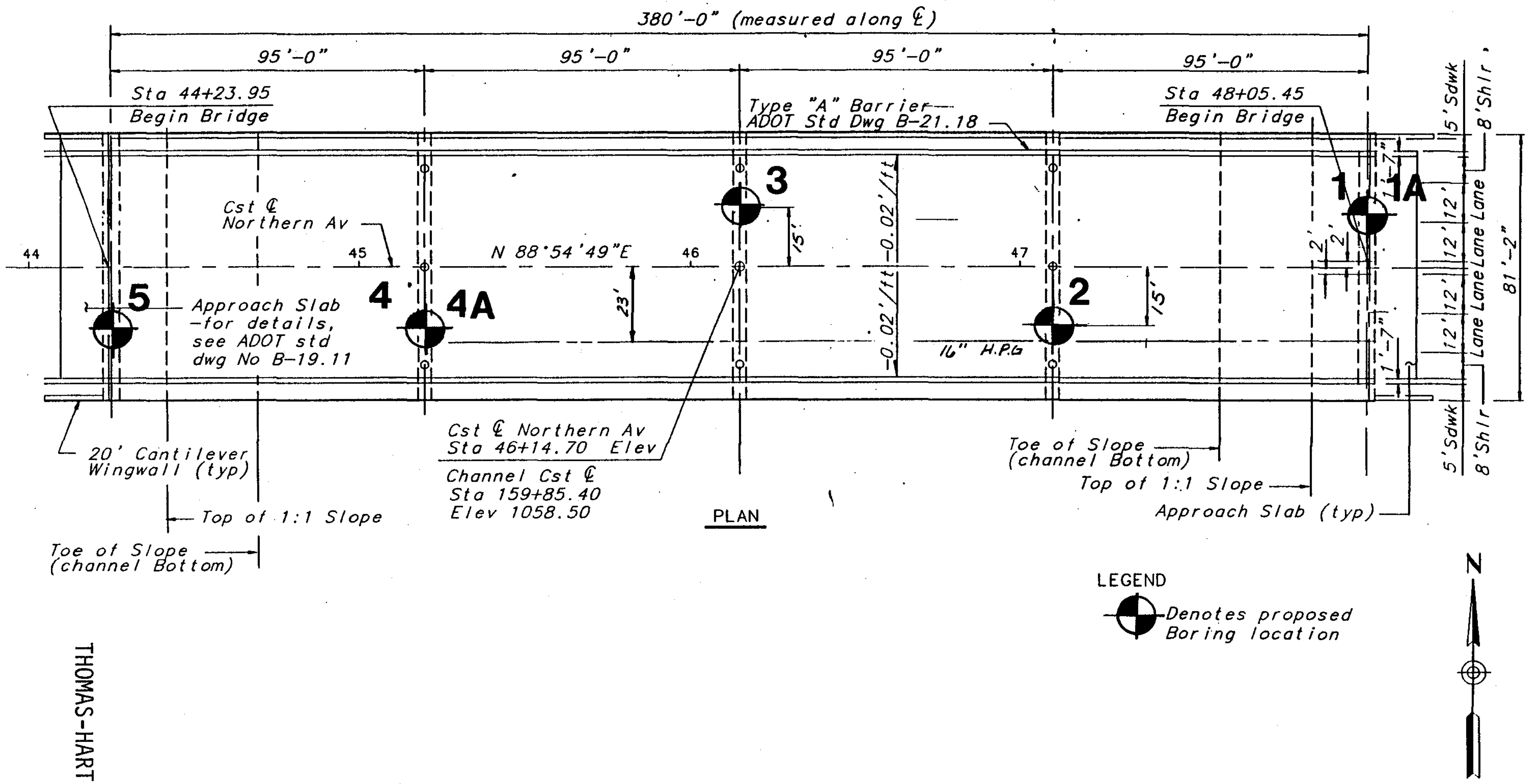
Settlements are estimated at one inch or less, and the majority of settlement will occur during initial construction.

Structure Backfilling: Backfill required against abutment walls and other retaining structures should be granular soils meeting the Maricopa Association of Government (MAG) specifications for backfill materials. The backfill soils should be free of any silty or clayey fines so that the backfill will be free-draining and not susceptible to increased loadings due to hydrostatic forces. Compaction should be accomplished to a minimum 95 percent of the ASTM D698 maximum density. Retaining structures should be braced to resist equipment loadings during compaction of the backfill.

The following tabulation presents recommended soil pressures for estimation of lateral forces against retaining walls.

Equivalent "Active" Soil Pressure (Yielding Structure)	30 psf/ft.
Equivalent "at-Rest" Soil Pressure (Rigid Structure)	45 psf/ft.

APPENDIX A
FIELD RESULTS


380'-0" (measured along ℄)
95'-0"
95'-0"
95'-0"
95'-0"
Sta 44+23.95
Begin Bridge
Type "A" Barrier
ADOT Std Dwg B-21.18
Sta 48+05.45
Begin Bridge
5'Sdwk
8'Shlr
Cst ℄
Northern Av
44
45
N 88°54'49"E
46
47
1
1A
2
3
4
4A
5
15'
15'
23'
-0.02'/ft
-0.02'/ft
16" H.P.G
2'
2'
1'-7"
1'-7"
12'
12'
12'
12'
Lane
Lane
Lane
Lane
81'-2"
Approach Slab
-for details,
see ADOT std
dwg No B-19.11
20' Cantilever
Wingwall (typ)
Cst ℄ Northern Av
Sta 46+14.70 Elev
Channel Cst ℄
Sta 159+85.40
Elev 1058.50
Top of 1:1 Slope
Toe of Slope
(channel Bottom)
PLAN
Toe of Slope
(channel Bottom)
Top of 1:1 Slope
Approach Slab (typ)
5'Sdwk
8'Shlr
LEGEND
Denotes proposed
Boring location
N

LEGEND

SOIL CLASSIFICATION

COARSE-GRAINED SOIL

More than 50% larger than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
	GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	
	GM	SILTY GRAVELS. GRAVEL-SAND-SILT MIXTURES. MORE THAN 12% - #200 FINES	
	GC	CLAYEY GRAVELS. GRAVEL-SAND-CLAY MIXTURES. MORE THAN 12% - #200 FINES	
	SW	WELL-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
	SP	POORLY-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	
	SM	SILTY SANDS. SAND-SILT MIXTURES MORE THAN 12% - #200 FINES	
	SC	CLAYEY SANDS. SAND-CLAY MIXTURES MORE THAN 12% - #200 FINES	

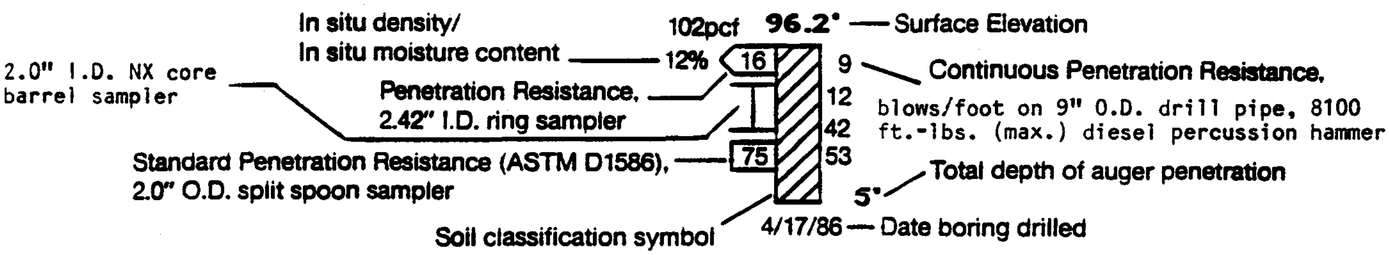
FINE-GRAINED SOIL

More than 50% smaller than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	ML	INORGANIC SILTS. ROCK FLOUR. AND FINE SANDY OR CLAYEY SILTS OF LOW TO MEDIUM PLASTICITY	SILTS AND CLAYS Liquid limit less than 50
	CL	INORGANIC CLAYS. GRAVELLY CLAYS. SANDY CLAYS. SILTY CLAYS. AND LEAN CLAYS OF LOW TO MEDIUM PLASTICITY	
	OL	ORGANIC SILTS AND ORGANIC SILT-CLAY MIXTURES OF LOW TO MEDIUM PLASTICITY	
	MH	INORGANIC SILTS. MICACEOUS OR DIATOMACEOUS. AND FINE SANDY OR CLAYEY SILTS OF HIGH PLASTICITY	SILTS AND CLAYS Liquid limit greater than 50
	CH	INORGANIC CLAYS. FAT CLAYS. AND SILTY CLAYS OF HIGH PLASTICITY	
	OH	ORGANIC CLAYS AND ORGANIC SILTS OF MEDIUM TO HIGH PLASTICITY	
	PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	

LEGEND FOR GRAPHICAL BORING LOGS:

Log denotes visual approximation unless accompanied by mechanical analysis and Atterberg limits.



GRAIN SIZES							
	U.S. STANDARD SERIES SIEVE: 200	40	10	4	CLEAR SQUARE SIEVE OPENINGS: 3/4"	3"	12"
SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE ⟶)

DRY	SLIGHTLY DAMP	DAMP (Plastic Limit)	MOIST	VERY MOIST	WET (SATURATED) (Liquid Limit)

CONSISTENCY CORRELATION		RELATIVE DENSITY CORRELATION	
CLAYS & SILTS	BLOWS/FOOT*	SANDS & GRAVELS	BLOWS/FOOT*
VERY SOFT	0-2	VERY LOOSE	0-4
SOFT	2-4	LOOSE	4-10
FIRM	4-8	MEDIUM DENSE	10-30
STIFF	8-16	DENSE	30-50
VERY STIFF	16-32	VERY DENSE	OVER 50
HARD	OVER 32		

*Number of blows of 140 lb. hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

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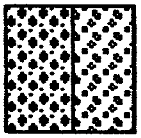
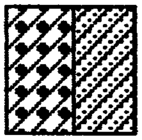
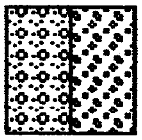
LEGEND OF SOIL TYPES



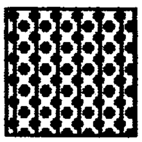
SAND AND GRAVEL, SOME COBBLES, TRACE SILT (GP-SW); brown; medium dense to very dense; slightly damp.



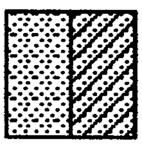
CLAYEY SAND AND GRAVEL WITH SOME COBBLES (GC-SC); brown; dense to very dense; slightly damp.



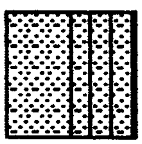
SAND AND GRAVEL WITH COBBLES (GW-SW); brown; dense to very dense; slightly damp.



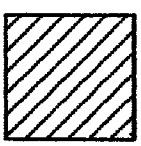
SILTY SANDY GRAVEL WITH COBBLES (GM); brown; very dense; slightly damp.



GRAVELLY SAND; SOME CLAY (SP-SC); brown; dense to very dense; slightly damp.



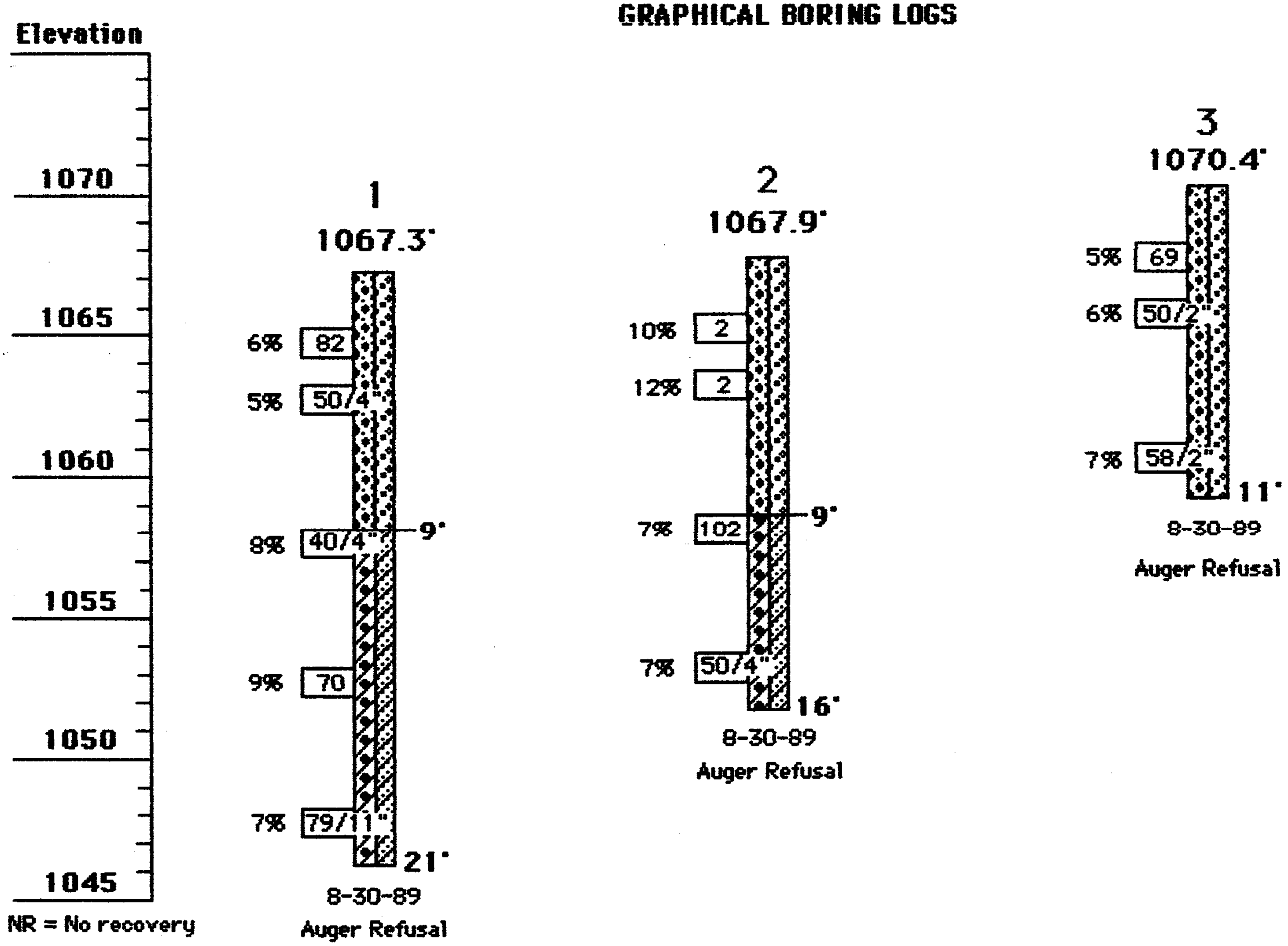
SAND WITH SOME SILT, AND GRAVEL (SP-SM); brown; dense; slightly damp.



SANDY CLAY (CL); brown; soft to firm; damp; low to medium plasticity.

Project No. 89-0747
Thomas-Hartig & Associates, Inc.

GRAPHICAL BORING LOGS



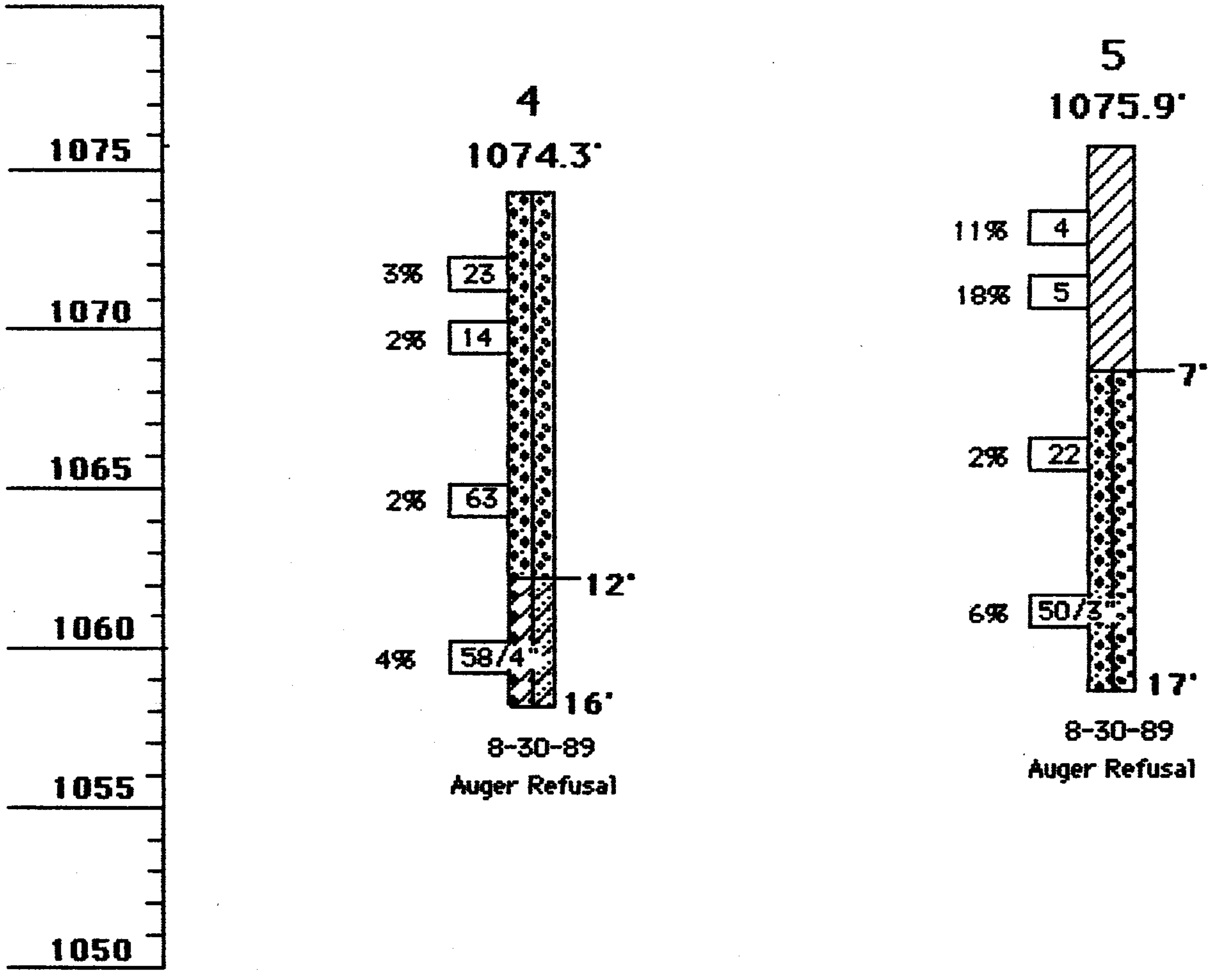
No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

Project No. 89-0747
Thomas - Hartig & Associates

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

GRAPHICAL BORING LOGS



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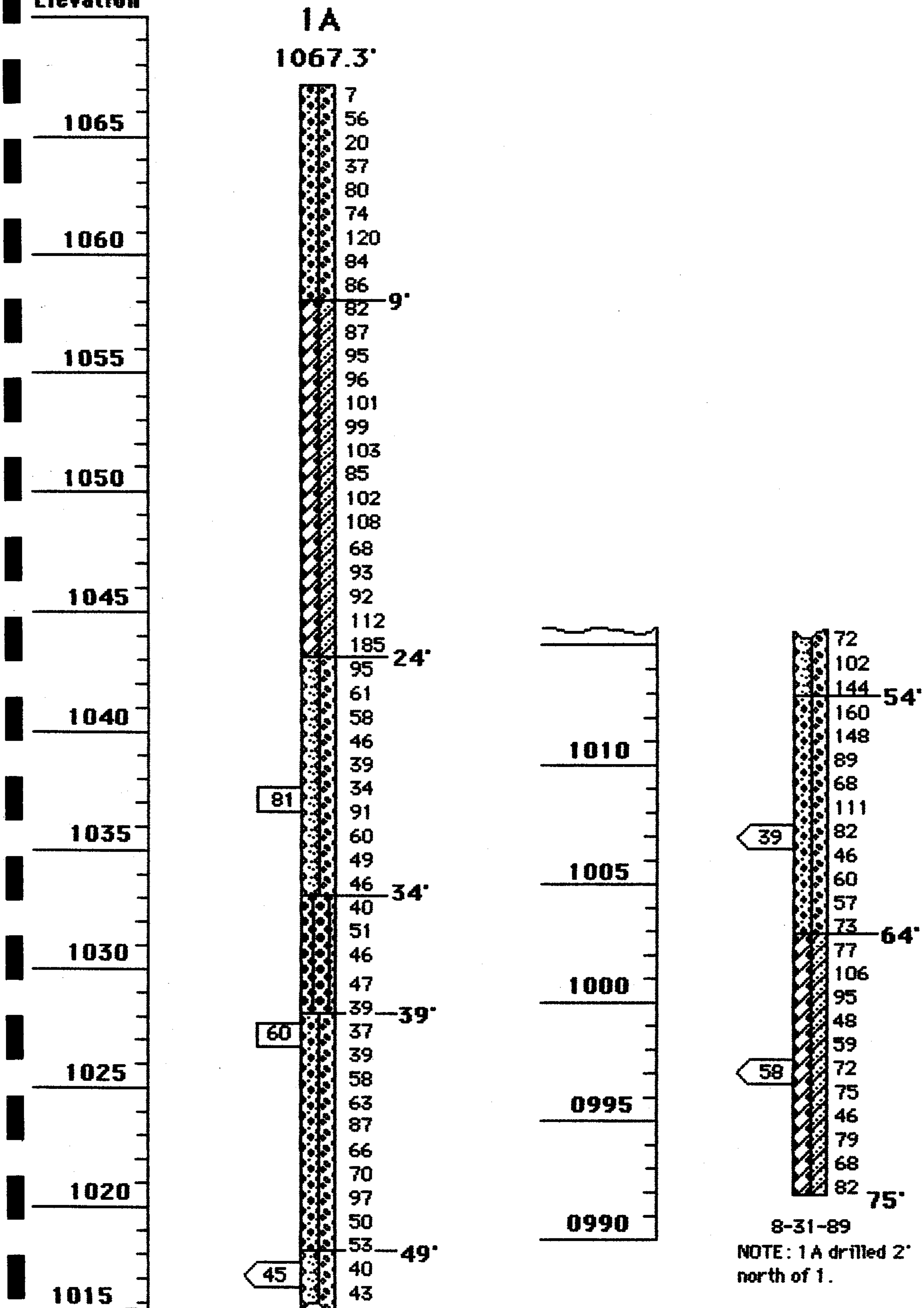
Project No. 89- 0747

Thomas - Hartig & Associates

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

GRAPHICAL BORING LOGS



NOTE: Test Boring drilled with 9" OD Double Wall Drill Pipe by Diesel Percussion Hammer.

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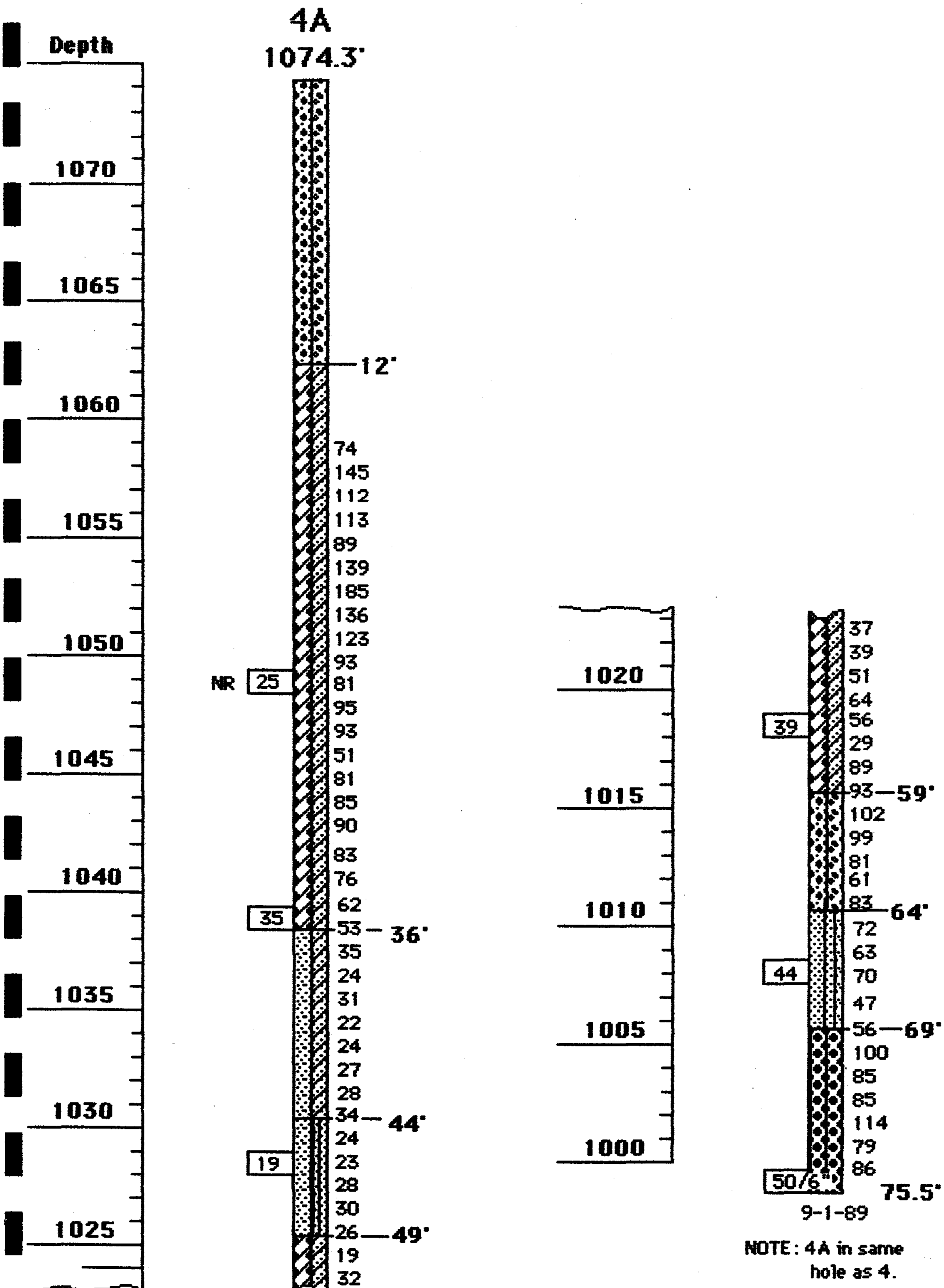


NOTE: 4A in same hole as 4.

NOTE: Test Boring drilled with 9" OD Double Wall Drill Pipe by Diesel Percussion Hammer.

Project No. 89-0747
Thomas-Hartig & Associates, Inc.

APPENDIX B
LABORATORY RESULTS

REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 1A; 29 - 34'

Type Grab Sample

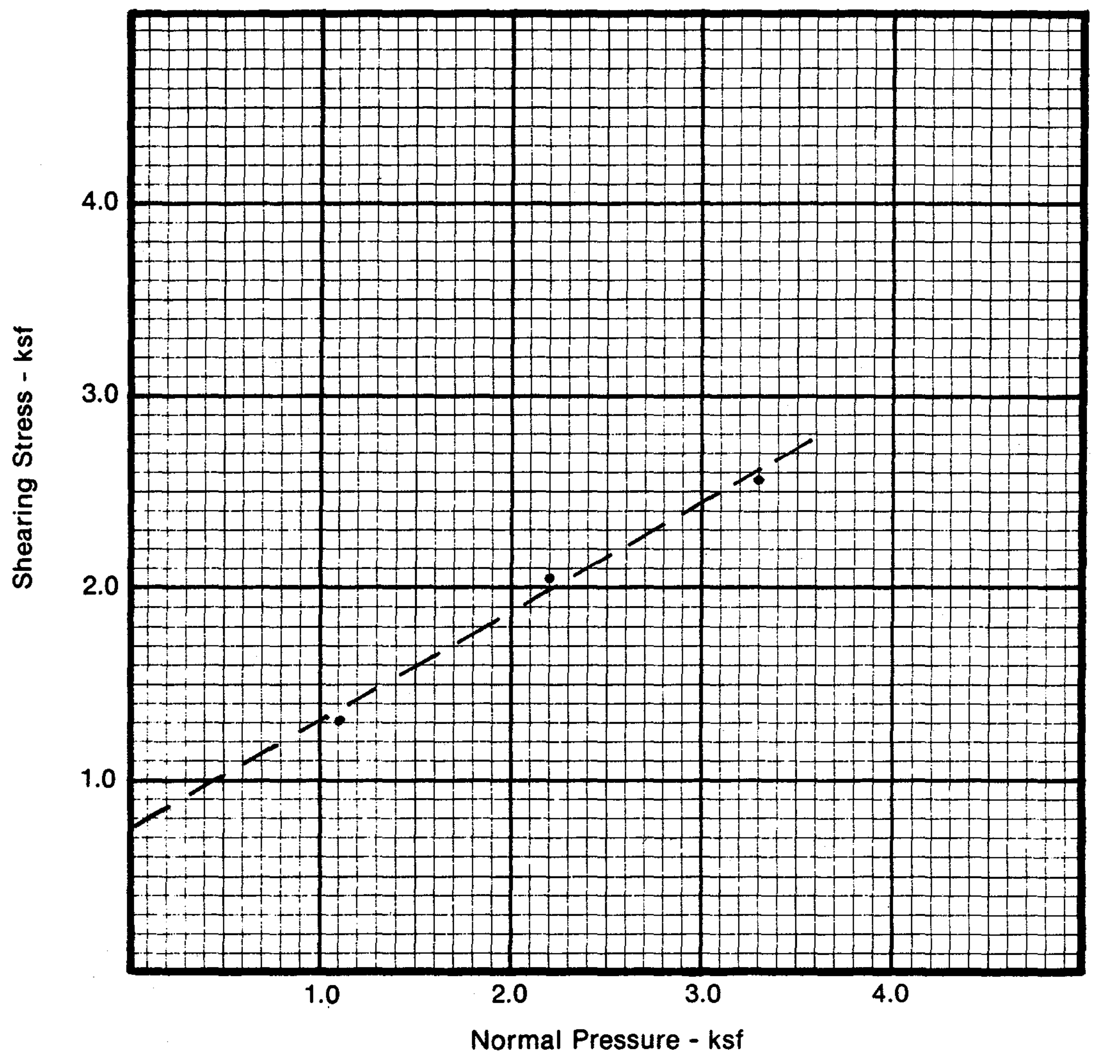
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 30° Cohesion (c) = 750 psf



Note: Direct Shear run on minus #4 material.

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REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 1A; 49 - 54'

Type Grab Sample

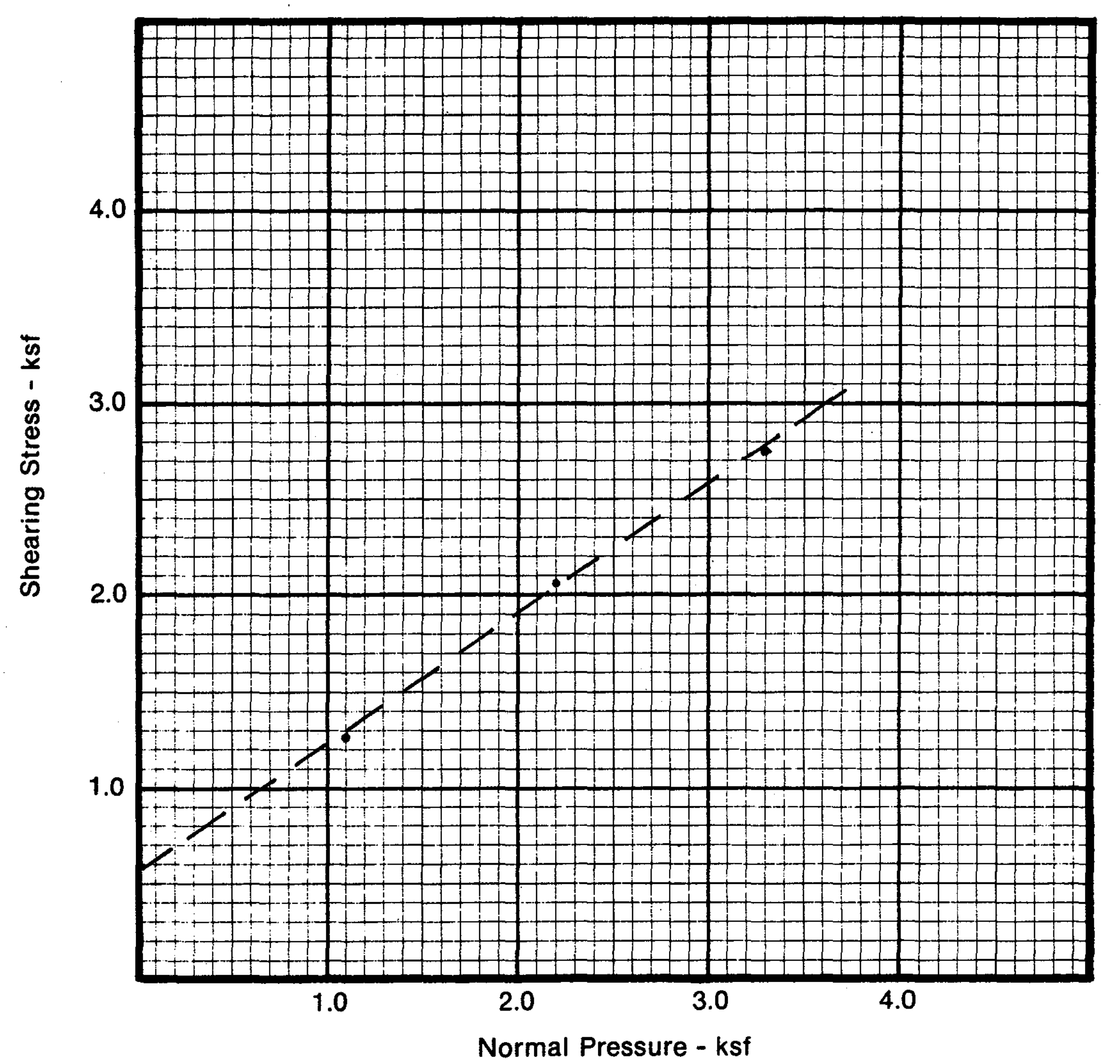
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 34° Cohesion (c) = 563 psf



Note: Direct shear run on minus #4 material.

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TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 1A; 69 - 70'

Type Grab Sample

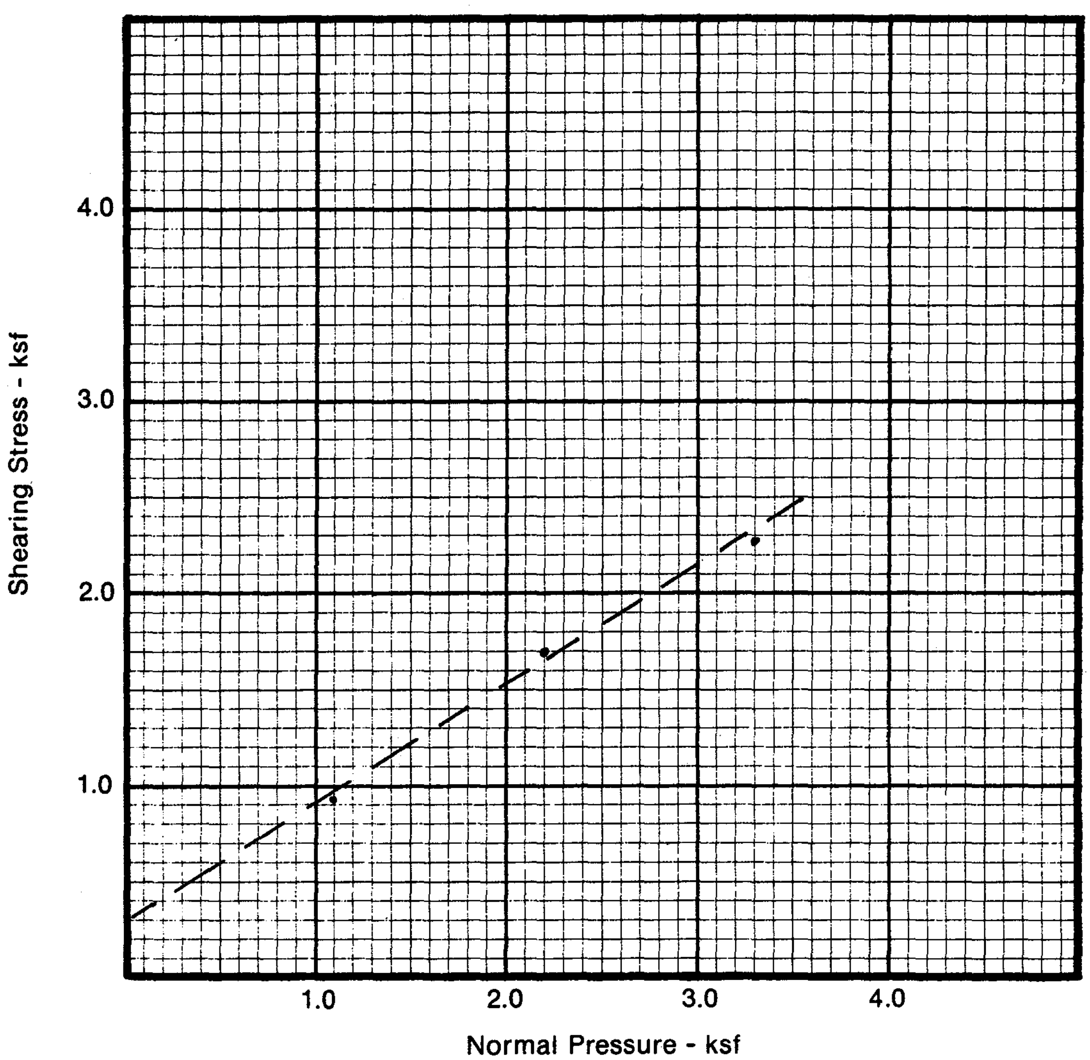
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 32° Cohesion (c) = 300 psf



Note: Direct shear run on minus #4 material.

Project No. 89-0747

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 1A; 44 - 49'

Type Grab Sample

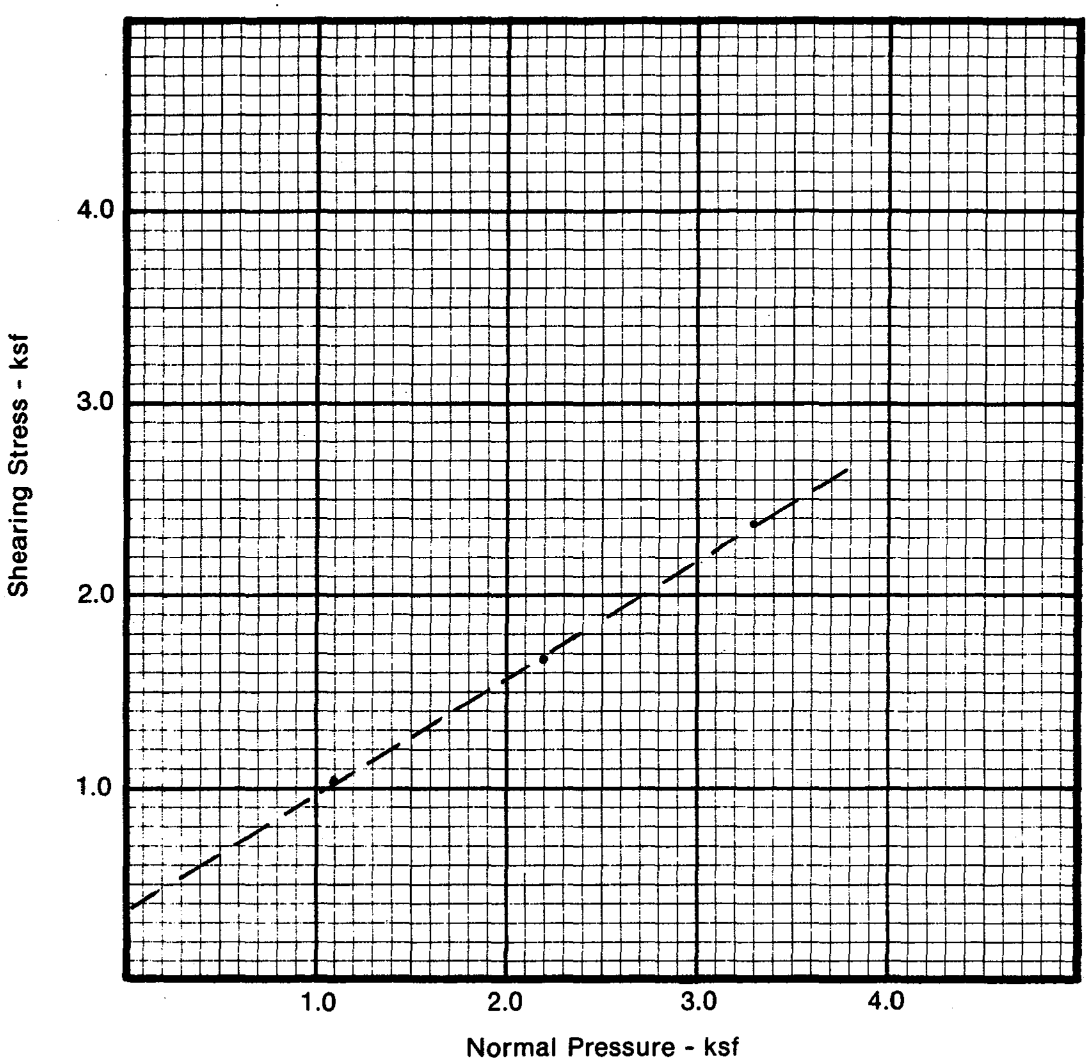
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 32° Cohesion (c) = 370 psf



Note: Direct shear run on minus #4 material.

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REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 4A; 54 - 55'

Type Grab Sample

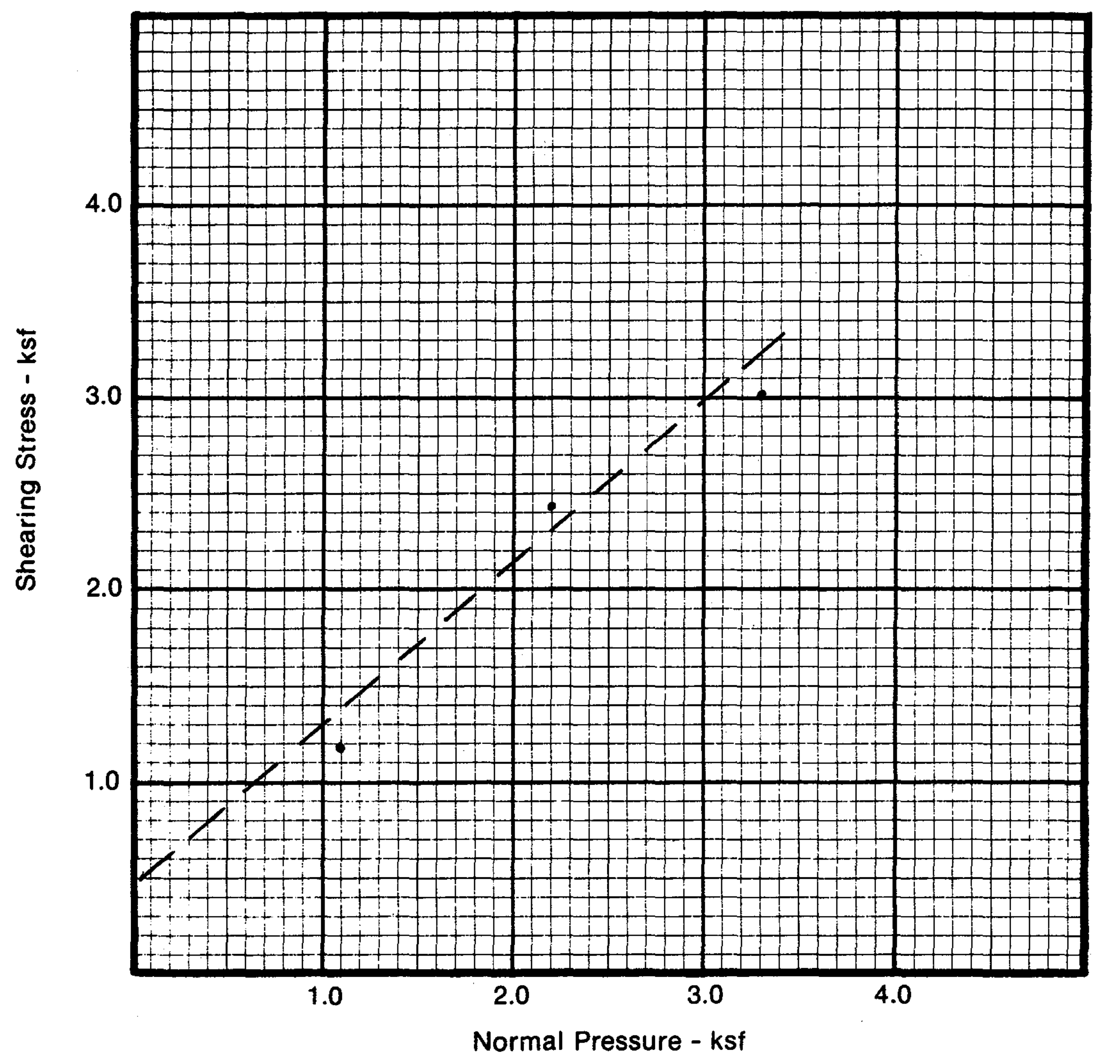
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 40° Cohesion (c) = 480 psf



Note: Direct shear run on minus #4 material.

Project No. 89-0747

REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source Test Boring 4A; 64 - 69'

Type Grab Sample

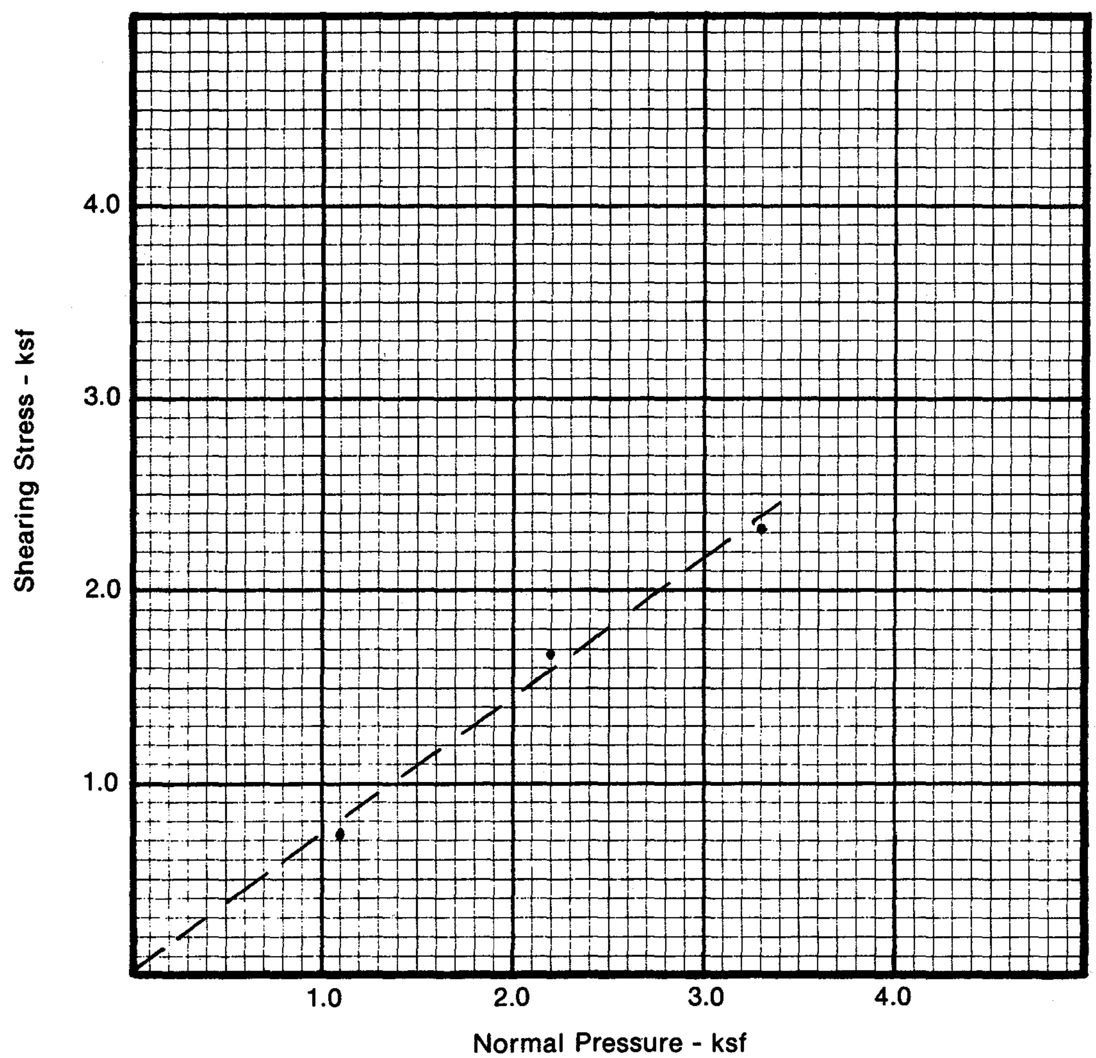
Material Soil

Sampled By TH/Perry

TESTED: Direct shear on compacted sample, sample submerged prior to shearing.

RESULTS:

Friction Angle (Ø) = 36° Cohesion (c) = 0



Note: Direct shear run on minus #4 material.

Project No. 89-0747

REPORT ON LABORATORY TESTS

SAMPLE: Date 9-29-89

Source As Noted Below

Type Bulk

Material Soil

Sampled By TH/Perry

TESTED: Sieve Analysis and Plasticity Index

RESULTS:
Note: Cutting samples from percussion hammer drill; Accum. % Passing for + #4 material may not be representative of actual size encountered.

Sample	LL	PI	Sieve Size - Accum. % Passing											* Class
			200	100	50	30	16	8	4	3/4"	1"	2"	3"	
1A; 29-34'	26	6	7	9	14	29	41	47	53	68	70	94	100	SP-GC
1A; 49-54'	39	11	40	46	52	61	68	73	77	90	100			SC
1A; 69-75'	36	14	31	38	46	58	69	75	79	93	95	100		SC
4A; 34-39'	36	13	7	9	13	23	42	59	69	80	86	95	100	SP-SC
4A; 44-49'	--	NP	6	10	21	47	73	87	95	100				SP-SM
4A; 54-59'	38	14	27	31	38	48	58	64	69	84	89	94	100	SC-GC
4A; 64-69'	22	3	7	10	19	43	78	93	97	100				SP-SM

NP = Non-Plastic

* Unified Soil Classification

Project No. 89-0747

THOMAS-HARTIG & ASSOCIATES, INC.

APPENDIX C
ENGINEERING COMPUTATIONS

Thomas-Hartig & Assoc, Inc.
2720 S. Hardy Drive
Tempe, Arizona 85282

PROJECT NO. 89-0747 COMPUTED BY KLM DATE 9/26/89
SUBJECT NORTHERN AVE BRIDGE CHECKED BY JWO DATE 10/6/89
DESIGN CRITERIA PER BRW, INC.

CHANNEL INVERT ELEV — 1058.25' w/ SOIL CEMENT SIDE SLOPES

ESTIMATED SCOUR = 15 TO 20' OR EL. 1043.25 TO 1038.25'

WATER LEVEL (100 YR) = ELEV. 1069.37
SHAFT DIA — 6 FOOT BELOW CONCAVE
TOTAL LENGTH OF BRIDGE 388'
TOTAL WIDTH OF BRIDGE 81'

LOAD INFORM.

ALTERNATE 1 — 3 SPAN STRUCTURE; 4 SHAFTS / PIER
3 SHAFTS / ABUT.

AT PIER -
D.L. 1015 KIPS / SHAFT
L.L. 204 KIPS / SHAFT MAX
T.L. 1219 KIPS / SHAFT

AT ABUT —
D.L. 766 KIPS / SHAFT
L.L. 126 KIPS / SHAFT (MAX)
T.L. 892 KIPS / SHAFT.

ALTERNATE 2 — 4 SPAN STRUCTURE; 3 SHAFTS / PIER & ABUT

AT PIER
D.L. 965 KIPS / SHAFT
LL 214 KIPS / SHAFT
T.L. 1179 KIPS / SHAFT
AT ABUT
DL 575 KIPS / SHAFT
L.L. 126 KIPS / SHAFT
T.L 701 KIPS / SHAFT

LATERAL LOADS (WORST CASE)

AT 10 FOOT BELOW CHANNEL
SHEAR 49 KIPS
MOMENT 575 FT-KIPS

AT 15 FOOT BELOW CHANNEL
SHEAR 49 KIPS
MOMENT 1578 FT-KIPS

AT 20 FOOT BELOW CHANNEL
SHEAR 49 KIPS
MOMENT 2031 FT-KIP

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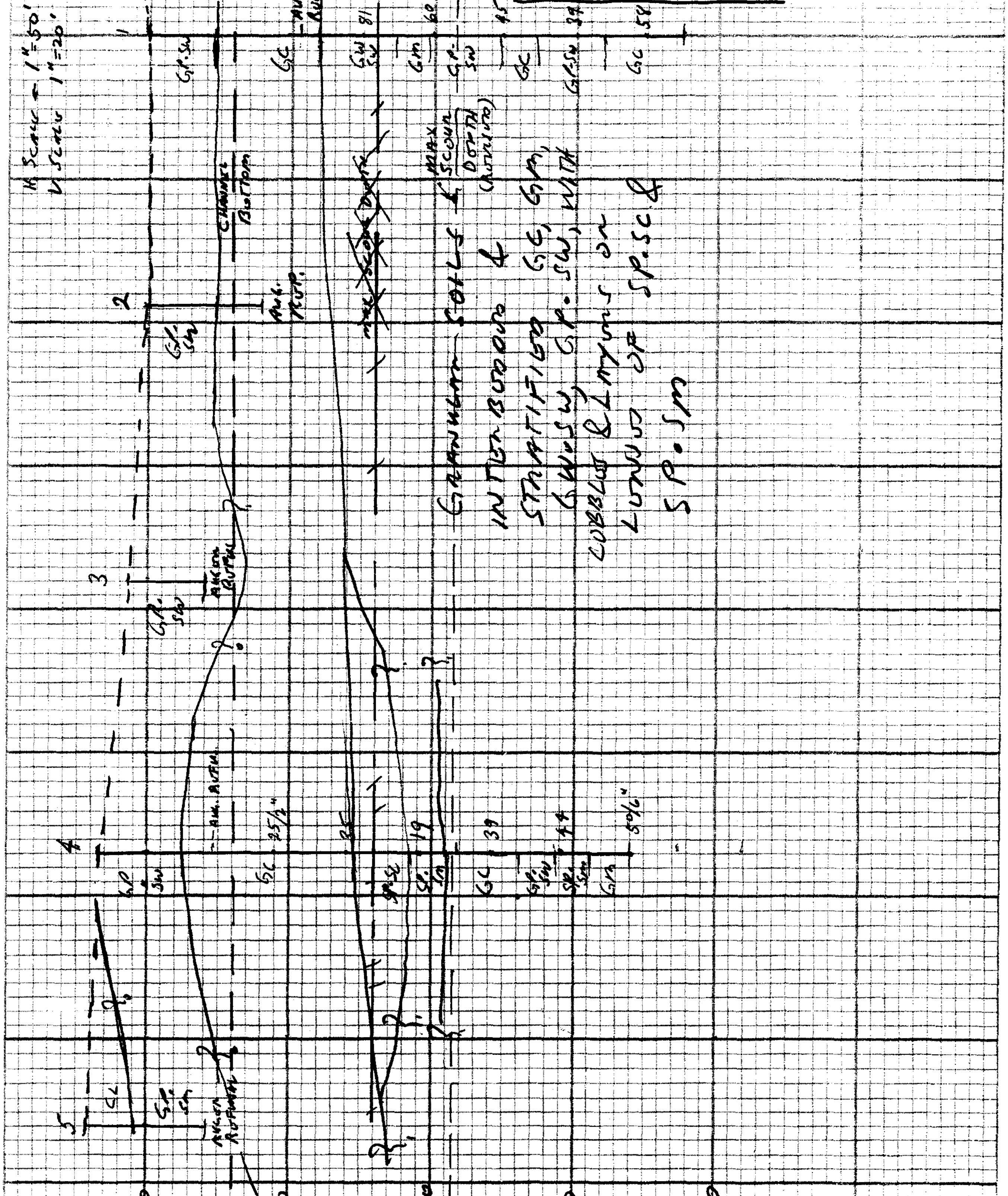
PROJECT NO. 89-0747 COMPUTED BY KLR DATE 9/26/89

SUBJECT Northern Ave. Bridge CHECKED BY JWD DATE 10/6/89

SOIL PROFILE



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PROJECT NO. 89-0747 COMPUTED BY KLM DATE 9/26/89
SUBJECT NORTHERN AVE. BRIDGE CHECKED BY JWO DATE 10/6/89

SOIL PARAMETERS

SOILS BELOW SCOUR ELEVATION A GRANULAR MATERIALS GC, GM, GW-SW, GP-SW WITH COBBLES AND LENSES OR LAYERS OF SP-SC & SP-SM SOILS.

IDEALIZED PROFILE BELOW SCOUR.

		From Table 6 Page 7.1-22 NAVFAC DM 7.1 γ_{sub}		Based on (blow counts) use	
		Loose	Dense		
Layer 1 0-10	SP-SC or SP-SM	53	73	63	(19 AA) Relative density 52 to 75
Layer 2 Below 10'	GC, GM, GP-SW, GW-SW	62	86	80	(39 TO 100) Relative density 72 TO 95

ϕ AND C SHOWN ON MINUS #4 MATERIAL (SUBMERGED)

FOR SP-SC, SP-SM $\phi = 36°$ $c = 0$
$\phi = 32°$ $c = 370$

FOR GC, GM, GP-SW, GW-SW $\phi = 30°$ $c = 750$
$\phi = 34°$ $c = 560$
$\phi = 32°$ $c = 300$
$\phi = 40°$ $c = 480$

FROM FIGURE 7 PAGE 7.1-149 OF NAVFAC DM-7.1

LAYER 1 SP-SC & SP-SM @ 52% RELATIVE DENSITY $\Rightarrow \phi = 34°$ $c = 0$
LAYER 2 GC, GM, GP-SW, GW-SW @ 72% RELATIVE DENSITY $\Rightarrow \phi = 37°$ $c = 0$

THEREFORE USE

LAYER 1 $\gamma_{sub} = 63$ PCF; $\phi = 34°$; $c = 0$

LAYER 2 $\gamma_{sub} = 80$ PCF; $\phi = 37°$; $c = 0$

DUE TO INCREASE IN SCOUR DEPTH NO LAYER 1 WILL EXIST.

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PROJECT NO. 89-0747 COMPUTED BY KL DATE 9/26/89
SUBJECT NORTHERN AVE BRIDGE CHECKED BY JWO DATE 10/9/89
PIER CAPACITY

CHECK BOTH END BEARING & SKIN FRICTION
CAPACITY USE ONE OR OTHER.

USE NAVFAC DM 7.2 PAGES 7.2-192 TO
LIMITING DEPTH OF INCREASE = 20D = 20 (6) = 120 FEET

END BEARING ONLY

$Q_{ULT} = P_T N_q A_T$

LAYER 1 $\gamma_{SUB} = 63$ PCF (0-10')
$N_q = 21$

LAYER 2 $\gamma_{SUB} = 80$ PCF (BELOW 10')
$N_q = 38$

$A_T = \frac{\pi D^2}{4} = \frac{\pi (6)^2}{4} = 28.3\ FT^2$

Q_{ULT} @ 15, 20, 25, 30.

$P_{T@15'} = (10)(63) + 5(80) = 1030$ PSF
$P_{T@20} = (10)(63) + 10(80) = 1430$ PSF
$P_{T@25} = (10)(63) + 15(80) = 1830$ PSF
$P_{T@30} = 10(63) + 20(80) = 2230$ PSF
$P_{T@35} = 10(63) + 25(80) = 2630$ PSF
$P_{T@40} = 10(63) + 30(80) = 3030$ PSF

$Q_{ULT} = P_T (38)(28.3) = 1075.4\ P_T / 1000\#/KIP = 1.0754\ P_T$ (IN KIPS)

			FS=2.5
$Q_{ULT@15'}$	= 1.0754 (1030)	= 1107.7 KIPS	443.1 KIPS
@20	= 1.0754 (1430)	= 1537.8 KIPS	615.1
@25	= 1.0754 (1830)	= 1968.0 KIPS	787.2
@30	= 1.0754 (2230)	= 2398.1 KIPS	959.3
@35	= 1.0754 (2630)	= 2828.3 KIPS	1131.3
@40	= 1.0754 (3030)	= 3258.4 KIPS	1303.4

USED FS = 2.5 BECAUSE TOTAL LOAD INCLUDES SHORT TERM LIVE LOAD
AND STRENGTH ASSUMPTIONS ARE CONSERVATIVE

FRICTION ONLY

$Q_{ULT} = K_{HC} P_O \tan\delta\ S$ $S = \pi D = \pi (6) = 18.8\ FT^2/FT$

LAYER 1 $\gamma_{SUB} = 63$ PCF (0-10')
$K_{HC} = 0.7$
$\tan\delta = \tan 3/4\phi = 0.48$

LAYER 2 $\gamma_{SUB} = 80$ PCF (BELOW 10')
$K_{HC} = 0.7$
$\tan\delta = \tan 3/4\phi = 0.53$

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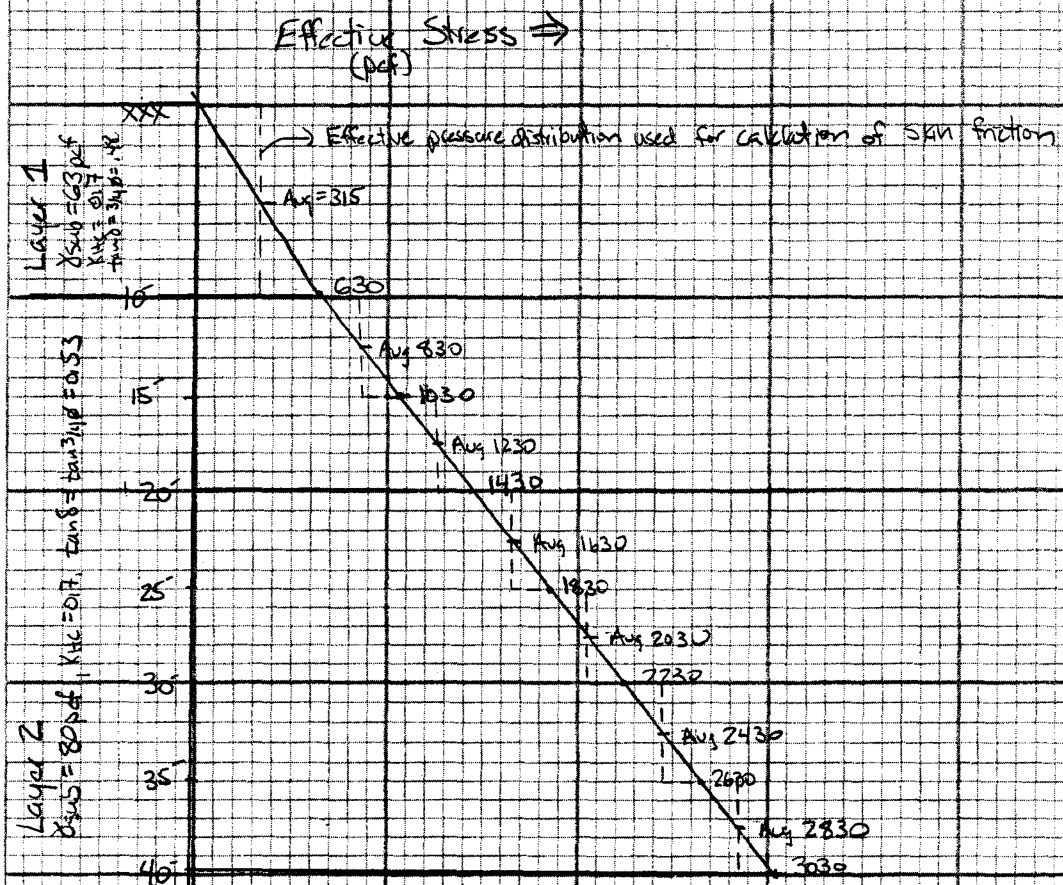
PROJECT NO. 89-0747 COMPUTED BY JWD DATE 10/9/89
SUBJECT Northern Ave. Bridge CHECKED BY [illegible] DATE 10/10/89



	K_{HC}	$\tan\delta$	P_o (ksf)	S (ft²/ft)	H (ft)	Q skin friction (kips)	Cumulative Q skin friction (kips)
0-10'	0.7	0.48	0.315	18.8	10'	19.9	19.9
10'-15'	0.7	0.53	0.830	18.8	5'	28.9	57.8
15'-20'	0.7	0.53	1.230	18.8	5'	42.9	100.7
20-25'	0.7	0.53	1.630	18.8	5'	56.8	157.5
25'-30'	0.7	0.53	2.030	18.8	5'	70.8	228.3
30'-35'	0.7	0.53	2.430	18.8	5'	84.7	313.0
35-40	0.7	0.53	2.830	18.8	5'	98.7	411.7

↑ F.S = 1

— Not enough capacity use end bearing only

24874

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PROJECT NO. 89-0747 COMPUTED BY KLN DATE 9/27/89
SUBJECT NORTHERN AVE. CHECKED BY JWO DATE 10/19/89

LATERAL LOAD CAPACITY

BASED ON NAVFAC DM 7.2 PAGES 7.2-235 TO 241

BASED ON A MIN. RELATIVE DENSITY OF 52%

$K_H = \frac{fz}{D}$ $f = 21 \text{ TON}/FT^3$ (FIGURE 9) $= 21 \times \frac{2000 \text{ lb}}{\text{TON}} \frac{FT^3}{(144)(12) IN^3}$

$D = 6 FT.$ $= 24.3 \text{ lb}/IN^3$

$T = \left(\frac{EI}{f}\right)^{1/5}$ $E = 3 \times 10^6$ PSI

$I = 0.049087 D^4 = 0.049087 (72)^4$

$= 1.319 \times 10^6 \; IN^4$

$T = \left[\frac{(3 \times 10^6 \text{ psi})(1.319 \times 10^6 \; IN^4)}{24.3 \text{ lb}/IN^3}\right]^{1/5} = \left[1.628 \times 10^{11}\right]^{1/5} = 1.75 \times 10^2 \; IN$

SAY $L = 35'$

$L/T = \frac{35' \; 12''/'}{175 \; IN} = 2.4$

AT $Z = 0$ FOR MAX DEFLECTION

FOR MOMENT $F_\delta = 2.1$ $F_m = 1.0$ $F_V = 0$

SHEAR $F_\delta = 3.2$ $F_m = 0$ $F_V = 0.85$

$\delta_m = F_\delta \left(\frac{MT^2}{EI}\right) = (2.1)\left(\frac{2031 \text{ FT-KIP} \; (12 \; IN/FT)(1000 \; \text{lb}/\text{KIP})(175 \; IN)^2}{(3 \times 10^6 \text{ PSI}) \; 1.32 \times 10^6 \; IN^4}\right)$

$= 0.452$ INCHES

$M_m = F_m M = (1)(2031 \text{ FT-KIPS})$

$V_m = F_V \left(\frac{M}{T}\right) = 0$

$\delta_P = F_\delta \frac{PT^3}{EI} = 3.2 \frac{(49 \times 10^3 \text{ lbs})(175 \; IN)^3}{(3 \times 10^6 \text{ PSI})(1.32 \times 10^6 \; IN^4)} = \frac{2.12 \times 10^8}{\times 10^9}$

$= 0.212$ INCH

$\delta_{max} = 0.452 + 0.212 = 0.664$ INCHES

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PROJECT NO. 89-0747 COMPUTED BY KLZ DATE 9/27/89
SUBJECT NORTHERN LATERAL (CONT) CHECKED BY JWO DATE 10/9/89

FROM CHART M_{MAX} OCCURS AT $Z = 0.8T$

$F^m_m = 0.85$ $F^s_m = 0.55$ $Z = 0.8(175) = 140$ inches
11.7 FT.

$M_m = F_m(M) = 0.85(2031 \text{ FT-KIPS})$
$= 1726.35$ FT-KIPS

$M_p = F_m(PT) = 0.55(49 \text{ KIPS})(175 \text{ in}) \frac{FT}{IN} = 393.0$ FT-KIPS

$M_{MAX} = 1726.4 + 393.0 = 2119$ FT-KIPS

FROM CHART V_{MAX} OCCURS AT $Z = 1.3T$ ON MOMENT & $Z = 1.6T$ ON SHEAR

$Z = 1.3$ $F^m_V = 0.7$ $F^s_V = 0.4$

$Z = 1.6$ $F^m_V = 0.65$ $F^s_V = 0.65$

@ $Z = 1.3$

$V_m = 0.7\left(\frac{(2031 \text{ FT-KIPS})(12 \text{ IN/FT})}{175 \text{ IN.}}\right) = -97.5$ KIPS

$V_p = 0.4(49) = 19.6$ KIPS

$V = 97.5 + 19.6 = 117.1$ KIPS

@ $Z = 1.6T = 23.3$ FT

$V_m = 0.65\left(\frac{(2031 \text{ FT-KIP})(12 \text{ IN/FT})}{175 \text{ IN}}\right) = 90.5$ KIPS

$V_p = 0.65(49) = 31.9$

$\boxed{V_{MAX} = 90.5 + 31.9 = 122.4 \text{ KIPS}}$

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PROJECT NO. 89-0747 COMPUTED BY JWO DATE 10/9/89
SUBJECT Northern Ave Bridge CHECKED BY KCR DATE 10/11/89

Settlement of Drilled Piers - Per NAVFAC DM-7.2 pp 7-2-207 to 7-2-209

1) Settlement due to axial deformation of pile shaft

$$Ws = (Q_p + \alpha_s Q_s)\ L/_{AE_p}$$

Q_p = Pile dead load

Q_s = Assumed zero for completely end bearing piles

L = pile length

A = Cross sectional Area of pile

E_p = modulus of elasticity of pile

Alternative 1

At Pier

$$Ws = (1015\text{ kips})\left(35^{ft}/(28.3ft^2)(4.32\times10^5 ksf)\right)\left(\frac{12in}{ft}\right) = 0.035\text{ in}$$

At Abutment

$$Ws = (766\text{ kips})\left(25^{ft}/(28.3ft^2)(4.32\times10^5 ksf)\right)\left(^{12in}/_{ft}\right) = 0.012\text{ in}$$

Alternative 2

At Pier

$$Ws = (965)\left(35'/(28.3)(4.32\times10^5)\right)\left(^{12in}/_{ft}\right) = 0.033\text{ in}$$

At Abutment

$$Ws = (575)\left(20'/(28.3)(4.32\times10^5)\right)\left(^{12in}/_{ft}\right) = 0.011\text{ in}$$

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PROJECT NO. 89-0747 COMPUTED BY JWO DATE 10/9/89
SUBJECT Northern Ave Bridge CHECKED BY [illegible] DATE 10/10/89

Settlement of drilled piers continued con't

2) Settlement of pile point caused by load Transmitted at the point

$$W_{pp} = \frac{C_p\, Q_p}{B\, q_o}$$

$C_p = (\text{table } 5) = \cancel{0.14}\ 0.09$

Q_p = Pile dead load

q_o = Ultimate end bearing capacity before applying factor of safety

B = Pile diameter

Alternative 1

At Pier

$$W_{pp} = \frac{\cancel{0.14}\ 0.09\,(1015\text{ kips})}{(6\text{ft})(2828.8\text{ kips}/28.3\text{ft}^2)}\left(\frac{12\text{in}}{\text{ft}}\right) = \cancel{2.84}\ 1.83\text{ in}$$

At Abutment

$$W_{pp} = \frac{\cancel{0.14}\ 0.09\,(766\text{ kip})}{(6\text{ft})(1968/28.3)}\ \frac{12\text{in}}{\text{ft}} = \cancel{3.08}\ 1.98\text{ in}$$

Alternative 2

at Pier

$$W_{pp} = \frac{\cancel{0.14}\ 0.09\,(965)}{(6\text{ft})(2828.3/28.3)}\left(\frac{12\text{in}}{1\text{ft}}\right) = \cancel{2.7}\ 1.74\text{ in}$$

at Abutment

$$W_{pp} = \frac{\cancel{0.14}\ 0.09\,(575)}{(6\text{ft})(1537.8/28.3)}\left(\frac{12\text{in}}{\text{ft}}\right) = \cancel{2.96}\ 1.90\text{ in}$$

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PROJECT NO. 89-0747 COMPUTED BY JWO DATE 10-9-89
SUBJECT CHECKED BY [illegible] DATE 10-10-89

Summary of Settlements

Alternative 1

At Pier

Settlement = 0.035 + ~~2.84~~ 1.83 = ~~2.875~~ 1.87 in

At Abutment

Settlement = 0.012 + ~~3.08~~ 1.98 = ~~3.092~~ 1.99 in

Alternative 2

At Pier

Settlement = 0.033 + ~~2.7~~ 1.74 = ~~2.733~~ 1.77 in

At Abutment = 0.011 + ~~2.96~~ 1.90 = ~~2.971~~ 1.91 in

SINCE BEARING STRATA IS TOO DENSE TO DRIVE PILE INTO AND FORMATION CONTAINS A HIGH PERCENTAGE OF GRAVEL

Cp OF 0.09 NOT VALID SHOULD USE LOWER VALUE FROM TABLE 5

DRIVEN PILE IN SAND RANGE FROM DENSE (0.02) TO LOOSE (0.04)

THE BORED PIER MUST BE AT LEAST EQUIVALENT TO LOOSE DRIVEN ∴ USE 0.04

SETTLEMENTS BECOME AS FOLLOWS

ALTERNATE 1

AT PIER

SETTLEMENT = 0.035 + 0.813 = 0.848 IN.

AT ABUTMENT

SETTLEMENT = 0.012 + 0.88 = 0.892 IN.

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PROJECT NO. 89-0747 COMPUTED BY KLA DATE 11/28/89
SUBJECT NORTHERN AVE BRIDGE CHECKED BY ______ DATE ______

LOAD CONDITIONS FOR LOU ARNOLD - BRW

THE BRIDGE WILL BE 4 SPANS WITH EITHER
3 OR 4 DRILLED PIERS PER BRIDGE PIER BENT

FOR 3 PIERS PER BENT
MAX DEAD LOAD = 1382 KIPS
MAX LIVE LOAD = 226 KIPS
MAX TOTAL LOAD = 1608 KIPS

FOR 4 PIERS PER BENT
MAX DEAD LOAD = 1037 KIPS
MAX LIVE LOAD = 169 KIPS
MAX TOTAL LOAD = 1206 KIPS

PIER MOMENT CALCUL. TO BE DONE BY BRW, INC.

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PROJECT NO. 89-0747 COMPUTED BY KUA DATE 11/22/89
SUBJECT NORTHERN AVE BRIDGE CHECKED BY JWP DATE 12/4/89
PIER CAPACITY

END BEARING ONLY

NO LAYER 1 MATERIAL

LAYER 2 $\gamma_{SUB} = 80 PCF$; $N_q = 38$

$A_T = \frac{\pi D^2}{4}$ FOR 6' Ø = 28.3 FT^2

FOR 6.5' Ø = 33.2 FT^2

FROM NAVFAC DM 7.2

$Q_{ULT} = P_T N_q A_T$

					END BEARING F.S. 2.5
P_T @ 15 = 15 (80)	= 1200 PSF	X N_q (38)	= 45.6 KSF		18.2 KSF (9.1 TSF)
P_T @ 20 = 20 (80)	= 1600 PSF	X N_q	= 60.8		24.3 KSF (12.2 TSF)
P_T @ 25 = 25 (80)	= 2000 PSF	X N_q	= 76.0		30.4 KSF (15.2 TSF)
P_T @ 30 = 30 (80)	= 2400 PSF	X N_q	= 91.2		36.5 KSF (19.2 TSF)
P_T @ 35 = 35 (80)	= 2800 PSF	X N_q	= 106.4		42.6 KSF (21.3 TSF)
P_T @ 40 = 40 (80)	= 3200 PSF	X N_q	= 121.6		48.6 KSF (24.3 TSF)
P_T @ 45 = 45 (80)	= 3600 PSF	X N_q	= 136.8 KSF		54.7 KSF (27.4 TSF)
					57.4 KSF (28.7 TSF)

FOR 6' Ø $Q = P_T N_q (28.3 FT^2)$ F.S = 2.5

DEPTH		
15'	1290.5 KIPS	516.2 KIPS
20'	1720.6	688.3
25'	2150.8	860.3
30'	2581.0	1032.4
35'	3011.1	1204.4
40'	3441.3	1376.5
45'	3871.4	1548.6

AT 47' = 1617.4

FOR 6.5' Ø $Q = P_T N_q (33.2 FT^2)$ FS = 2.5

DEPTH		
15	1513.9	605.6
20	2018.6	807.4
25	2523.2	1009.3
30	3027.8	1211.1
35	3532.5	1413.0
40	4037.1	1614.8

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PROJECT NO. 89-0747 COMPUTED BY KLR DATE 11/28/89
SUBJECT NORTHERN AVE BRIDGE CHECKED BY JWD DATE 12/4/89

FOR FOUR PIERS PER BENT
FROM NAVFAC DM 7.2 pp 7.2-207 TO 7.2-209

1. SETTLEMENT DUE TO AXIAL DEFORMATION OF PILE SHAFT BELOW JCONN.

$W_s = (Q_P + \alpha_s Q_S)\, L/AE_p$

$Q_p = 1036.5$

$Q_S = 0$ (END BEARING ONLY)

$L_1 = 30'$; $L_2 = 35'$; $B_1 = 6.5$; $B_2 = 6.0$

$A_1 = \frac{(6.5)^2(\pi)}{4} = 33.2\ FT^2$ $A_2 = 28.3\ FT^2$

$E_p = 4.32 \times 10^5$ KSF

FOR 6' Ø ; 35' DEEP

$W_s = (1036.5 + 0) \frac{(35\ FT)}{(28.3\ FT^2)(4.32 \times 10^5\ KSF)} \frac{12\ IN}{FT}$

$W_{s_2} = 0.036$ INCHES

FOR 6.6' Ø ; 30' DEEP

$W_{s_1} = (1036.5 + 0) \frac{30\ FT}{(33.2\ FT^2)(4.32 \times 10^5\ KSF)} \frac{12}{FT}$

$W_{s_1} = 0.026$ INCHES

2. SETTLEMENT OF PILE TIP BY END BEARING ONLY

$W_{pp} = \frac{C_p Q_p}{B\ q_o}$

USE $C_p = 0.04$ FROM TABLE 5 SEE 10 OF 10 FOR JUSTIF.

$q_{o_1} = (21.3\ TSF)(2\ \frac{K}{T})(2.5)^{F.S.} = 106.5$ KSF

$q_{o_2} = (24.3\ TSF)(2\ K/T)(2.5) = 121.5$ KSF

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PROJECT NO. 89-0747 COMPUTED BY KC DATE 11/28/89

SUBJECT Northern Ave. CHECKED BY JWO DATE 12/4/89

$$w_{pp_1} = \frac{(0.04)(1036.5)}{(6.5\,FT)(106.5\,KSF)}\left(\frac{12\,IN}{FT}\right) = 0.719\ \text{INCHES}$$

$$w_{pp_2} = \frac{(0.04)(1036.5)}{(6.0)(121.5\,KSF)}\frac{12}{FT} = 0.682\ \text{INCHES}$$

3. SETTLEMENT FOR FRICTION LOAD

END BEARING ONLY

$w_{ps_1} = 0$

$w_{ps_2} = 0$

4. TOTAL SETTLEMENT

FOR 6.5' Ø 30' LONG

$w_{T_1} = w_{S_1} + w_{pp_1} + w_{ps_1} = 0.026 + 0.719 = 0.745$ INCHES

FOR 6.0' Ø 35' LONG

$w_{T_2} = w_{S_2} + w_{pp_2} + w_{ps_2} = 0.026 + 0.682 = 0.718$ INCHES

FOR 3 PLUS BENT

$Q_p = 1382$, $Q_s = 0$ $L_3 = 47'$ $B_3 = 6.0'$ $A_3 = 28.3\ FT^2$

$E_p = 4.32 \times 10^5$ KSF $L_4 = 40'$ $B_4 = 6.5'$ $A_4 = 33.2\ FT^2$

$$w_{S_3} = \frac{(1382)(47)(12)}{(28.3)(4.32 \times 10^5)} = 0.064\ \text{INCHES}$$

$$w_{S_4} = \frac{(1382)(40)(12)}{(33.2)(4.32 \times 10^5)} = 0.046\ \text{INCHES}$$

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PROJECT NO. 89-0743 COMPUTED BY [illegible] DATE 11/28/89
SUBJECT Northern Ave. Bridge CHECKED BY JWO DATE 12/4/89

$C_p = 0.04$

$q_{o_3} = (28.7\ TSF)(2)(2.5) = 143.5\ KSF$

$q_{o_4} = 121.6\ KSF$

$$W_{pp_3} = \frac{(0.04)(1382)(12)}{6.0(143.5)} = 0.770$$

$$W_{pp_4} = \frac{0.04(1382)(12)}{(6.5)(121.6)} = 0.839$$

$W_{ps_3} = 0$; $W_{ps_4} = 0$

6' Ø, 47' LONG

$WT_3 = 0.064 + 0.770 = 0.834$ INCHES

6.5' Ø, 40' LONG

$WT_4 = 0.046 + 0.839 = 0.885$ INCHES