Property of
Flood Control District of MC Library
Please Return to
2801 W. Durango
Phoenix, AZ 85009

OLD CROSS CUT CANAL

Indian School Road to McDowell Road
Phoenix, Arizona

Prepared for

Greiner, Incorporated
7310 North 16th Street, Suite 160
Phoenix, Arizona

FLOOD CONTROL DISTRICT
RECEIVED
FEB 13 '91

	CH ENG		P & PM
	DEP		HYDRO
	ADMIN		DIRECT
	FINANCE		FILE
	C & O		
	ENGR		

REMARKS

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

A103.903

THOMAS-HARTIG & ASSOCIATES, INC.

TOM W. THOMAS, P.E. • HARRY E. HARTIG, P.E.
Geotechnical, Materials Testing, and Environmental Consultants
7031 West Oakland Street • Chandler, Arizona 85226

James R. Morrow
John P. Boyd, P.E.
Charles H. Atkinson, P.E.
James M. Willson, P.E.

Frank M. Guerra, P.E.
Steven A. Haire, P.E.
Kenneth L. Ricker, P.E.
Judith A. McBee

Dale V. Bedenkop, P.E.
John C. Patton
Kenneth D. Walsh, P.E.
Lynn Olson

Greiner, Incorporated
7310 North 16th Street, Suite 160
Phoenix, Arizona 85020-2401

12 February 1991

Attention: Shi-En Shiau, P.E.

Project: Old Cross Cut Canal
Indian School Road to McDowell Road
Phoenix, Arizona

Project No. 90-0863

This report presents the results of the geotechnical engineering services authorized on the site for the proposed Old Cross Cut Canal from Indian School Road to McDowell Road in Phoenix, 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.

By: Kenneth D. Walsh, P.E.

Reviewed by: Kenneth L. Ricker, P.E.

/dkl-s
Copies to: Addressee (2)
Entranco/William Kantor, P.E. (1)

TABLE OF CONTENTS

PART I - REPORT

Scope 1
Site Description 1
Investigation 1
Laboratory Investigation 2
Soil Conditions 3
Discussion and Recommendations:
- General 4
- Expansion Potential 4
- Culvert Support 4
- Lateral Design Parameters 5
- Cross Street Support 5
- Excavation Conditions 5
- Site Soil Workability 6

PART II - MATERIALS

Fill Materials 8
Pavement 8

PART III - EXECUTION

Site Grading 9
Paving 10

APPENDIX A - FIELD RESULTS

Site Plan 11
Legend 12
Legend of Soil Types 13
Boring Logs 14
Schematic Well Construction Plan 25
Field Resistivity Tests 26

APPENDIX B- LABORATORY RESULTS

Gradation & Plasticity Index 27
Direct Shear Tests 29
Compression Tests 34
pH, Soluable Salts, Sulfates & Chlorides 39
Moisture-Density Relationship Curves 40
R-Value 45
Expansion Tests 46

Registered Professional Engineer
CERTIFICATE NO.
KENNETH
Date Signed 2-12-91
ARIZONA, U.S.A.

PROJECT NO. 90-0863

PART I
REPORT

SCOPE

The proposed old Cross Cut Canal Flood Control Improvements will eventually consist of a new culvert to carry the canal flow and improved roadways for the Hohokam Parkway and 48th Street. The project extends from McDowell north to the Arizona Canal along the Old Cross Cut Canal. This phase of the project includes only the installation of the culvert. This report contains a description of our field operations and laboratory results and design recommendations concerning constructability, excavations and slope stability, bearing capacity and lateral earth pressures, bedding and backfilling materials, and pavement thicknesses for City of Phoenix cross streets affected by this project.

SITE DESCRIPTION

The Old Cross Cut Canal in the project area from McDowell Road to the Arizona Canal consists of an open channel with undercrossings at Thomas Road, Osborn Road, and Indian School Road. South of Osborn Road, the canal is bounded by an unpaved service road with a pedestrian walkway and bicycle path on the west bank and by 48th Street on the east bank. North of Osborn Road, the canal is bounded on the west by 48th Street northbound and on the east by residential areas. The canal banks are typically unlined and steeply sloping. As of our field operations, the canal carried only low water flows south of Osborn Road.

INVESTIGATION

The field investigation included a site reconnaissance, subsurface exploration, and field resistivity testing. The subsurface exploration consisted of drilling 22 test borings at the locations shown on the site plan in Appendix A. The test borings were drilled with a CME 55 drill rig using 7-inch hollow stem augers. The test borings were drilled to a depth of 25 feet. Standard Penetration Test (SPT) sampling and driven ring sampling was performed in all borings, alternating at 5-foot intervals, to obtain an indication of the relative density and/or consistency of the formation being penetrated and to obtain samples for laboratory testing. Where possible, bulk samples were obtained from the cuttings. Groundwater levels were noted during drilling, and in some test borings stabilized groundwater levels were measured in holes left temporarily open.

Piezometers for observing groundwater levels were constructed at Test Borings 3, 8, 13, 16, and 21. These piezometers will be monitored monthly until such time as

the design plans are approved and accepted by the Flood Control District. The wells will then become the property and responsibility of the Flood Control District for subsequent monitoring and abandonment. We emphasize that the abandonment must be conducted by the Flood Control District in accordance with the policies and regulations of the Arizona Department of Water Resources.

During the field investigation, the soils encountered were visually classified by our field engineer. The results of the test drilling conducted for this project are presented on the boring logs in Appendix A, "Field Results."

The soil resistivity was measured using a 4-terminal "Megger Earth Tester" resistivity meter. The resistivity tests were conducted using two different electrode spacings to indicate the variation of soil resistance with depth. The resistivity values ranged from about 1910 to 9580 ohm-cm. The results of the field resistivity testing conducted for this project are presented in Appendix A, "Field Results".

LABORATORY INVESTIGATION

Laboratory testing was conducted on representative soil samples obtained during the test drilling. The testing was conducted to obtain the data necessary to develop design recommendations for this project. The following tests were conducted:

Test	Sample(s)	Purpose
Sieve Analysis & Atterberg Limits	Representative (22)	Classification and correlation of engineering properties
Dry density and Moisture Content	Undisturbed (51) Disturbed (55)*	In-situ density and moisture determination to correlate engineering properties
Direct Shear	Undisturbed (5)	Bearing capacity and slope stability analysis
Compression	Undisturbed (5)	Settlement analyses
Soluble Salts, Sulfates and Chlorides	Representative (5)	Corrosion potential

Test	Sample(s)	Purpose
ASTM D698	Representative Grab Sample (5)	Compaction characteristics
R-value	Representative Grab Sample (4)	Pavement design
Expansion	Compacted (2) Undisturbed (1)	Expansion potential

*Disturbed samples from SPT sampling tested for moisture content only.

The results of the moisture and density testing are presented on the graphical boring logs in Appendix A. The results of the remainder of the testing are presented in Appendix B.

SOIL CONDITIONS

The soil profile at the boring locations is presented on the graphical boring logs in Appendix A. The soil profile along the site consists of a medium dense to dense clayey sand/sandy clay deposit. The deposit is light brown to reddish brown, and contains varying amounts of gravel particles and gravelly lenses. The gravels consist predominantly of subangular to angular granite fragments. The material exhibits moderate to high plasticity. The degree of calcareous cementation varies from light to heavy, and generally increases with depth. A review of nearby projects in our files indicated that similar materials have been encountered along the alignment and for some distance on either side. Expansion potentials from nearby projects ranged from 0 to 4.6% on remolded samples from projects in the area, and from 0.3 to 1.4% from this project.

Soil moisture contents at the time of test drilling were generally described as damp to moist above the groundwater level. Groundwater was detected in most of the test borings, as shown on the test boring logs in Appendix A, at depths ranging from 12 to 25 feet below existing ground surface. These groundwater levels represent only the conditions encountered at the time of our field drilling operations. Groundwater levels may vary with time, seasonal conditions, and/or water flow in the old Cross Cut Canal.

DISCUSSION AND RECOMMENDATIONS

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

Expansion Potential: Existing soils are sandy clays and clayey sands, predominantly of medium plasticity. At existing moisture conditions, the undisturbed soils will demonstrate moderately low potentials for expansion. However, compaction of these soils will further increase expansive potentials, especially if these soils are compacted to relatively high densities at moisture contents below optimum. Expansive potentials of new fills constructed in these soils are estimated on the order of 1/4 to 1/2-inch per foot of compacted fill. Additionally, significant swelling pressures could develop against culvert walls adjacent to compacted backfills. For this reason, imported granular soils exhibiting low expansive potentials are recommended for any backfills above the base of the excavation for the culvert installation.

Culvert Support: The culvert to be installed to convey the canal flow will be placed from 14 to 20 feet below ground. The soil along the canal is fairly strong, and the culvert will be lighter than the soil it replaces. Therefore, we anticipate low settlements of less than 1/2 inch with an allowable bearing capacity of 5000 psf. Two feet of granular fill should be provided below the bottom of the culvert, as described in Site Grading later in this report.

Because of the shallow groundwater level along the alignment, allowance must be made to prevent buoyant uplift under the condition of high groundwater when the culvert is empty or near empty. A minimum 4-foot soil cover over the top of the culvert will be sufficient to prevent such uplift.

Lateral Design Parameters: The following tabulation presents recommendations for lateral earth pressures expected against buried culvert structures:

Lateral Backfill Pressures:

Above groundwater table ..... 60 psf/ft.

Below groundwater table ..... 95 psf/ft.

These pressures are equivalent fluid pressures for vertical walls and horizontal backfill surfaces (maximum 12 foot height). Pressures do not include temporary forces imposed during compaction of the backfill, swelling pressures developed by over-compacted clayey backfill, or surcharge loads. Walls should be suitably braced during backfilling to prevent damage and excessive deflection. We recommend that only manual compaction equipment be used within 5 feet of culvert walls.

Cross Street Pavements: Pavement reconstruction will be required over the culvert installation at Thomas Road, Osborn Road, and Indian School Road. Based on discussions with the City of Phoenix Materials staff, we recommend that an 8-inch thick full depth asphalt concrete section be used, unless the existing pavements are thicker. Thickness of existing pavement was checked on as-built drawings for the cross streets. The final recommended pavement thicknesses are tabulated below:

Cross Street	Full Depth Asphalt Concrete (inches)
Thomas Road	8 1/2
Osborn Road	8
Indian School Road	11

Pavement materials should not be placed when the subgrade is wet. The surface should be sealed after weathering is apparent to minimize water infiltration directly through the pavement section and retard oxidation.

Excavation Conditions: The test drilling and field sampling at the site were performed for design purposes. It is not possible to accurately correlate auger drilling results with the ease or difficulty of digging for various types and sizes of

excavation equipment. We present the following general comments regarding excavatability for the designers' information with the understanding that they are approximations based only on test boring data. More accurate information regarding excavatability should be evaluated by contractors or other interested parties from test excavations using the intended equipment.

The near surface soils are non-cemented to weakly cemented natural soil deposits which can probably be removed with conventional excavating equipment. However, variable carbonate cementation (caliche) was encountered in some locations, typically below about 4 feet, and excavations into these deeper soils could be more difficult. All excavations should be braced or sloped to provide personnel safety and satisfy local safety code regulations. We recommend temporary cut slopes at 1:1 (horizontal:vertical) for the upper 8 feet and 1/2:1 (H:V) for lower portions of the excavation. The excavation will probably encounter groundwater for much of its length.

Site Soil Workability: Below the culvert bottom the moisture content of existing site soils should be maintained between optimum and optimum +3 percent (ASTM D698) during and subsequent to site grading to reduce expansive potentials. At these conditions, some pumping may be experienced under dynamic loading if the compaction is done by very heavy equipment (i.e., loaded scrapers, water-pulls, etc.). We would not consider some pumping detrimental in areas below the culvert bottom (i.e., static loading conditions) provided specified densities are obtained. Lighter compaction equipment and/or drying of wet soils may be used to reduce pumping if this condition becomes severe.

In bituminous paved areas, the moisture content of the subgrade and backfill should be maintained at 2 percent below optimum or lower during site grading to reduce the potential for pumping. If in-situ moisture contents are higher than this at the time of construction, pumping may occur, and special precautions should be taken to prevent disturbance, equipment mobility problems, and loss of shear strength in the subgrade. These precautions may include spreading and drying of wet soils, removal and replacement of wet soils, construction of temporary gravel roads at channelized traffic areas, and/or use of lighter compaction equipment.

Because of the shallow groundwater conditions encountered in many of the test borings, the use of a dewatering system will be required during construction. A dewatering scheme we believe to be acceptable would consist of two feet of wash gravel below culvert grade and the employment of pumped drainage sumps to remove accumulating water. The recommendations of material and grading requirements to follow are based upon this system. The geotechnical engineer should be contacted for review of alternative drainage and/or bedding concepts.

PART II
MATERIALS

FILL MATERIALS

All fill materials should be soils free of vegetation, debris, organic contaminants, and fragments larger than *6 inches in size. The existing site surface soils become moderately expansive when compacted. Therefore, these soils should not be used for backfill against the sides of the culverts, but may be used as backfill above the top of the culverts. All backfills against the side of the culverts should be of imported soils with low expansive potentials.

Backfill materials against culvert sides should be imported granular soils conforming to the following specification requirements:

Maximum particle size	6 inches*
Maximum percent expansion	1.5**
Maximum percent passing No. 200 sieve	25***
Maximum plasticity index	5***

* Maximum size may be reduced at engineer's direction to satisfy trenching and landscaping requirements, etc.

** Performed on sample remolded to 95 percent of the maximum ASTM D698 density and 2 percent below optimum moisture under a 100 psf surcharge pressure.

*** Required for deep fills or culvert backfills where the fill thickness is greater than 4 feet.

Two feet of granular material should be provided beneath the bottom of the culverts. This material will provide a working surface for placing precast culverts or forming cast-in-place culverts and a drainage layer for controlling shallow groundwater along the alignment. We recommend a clean wash gravel with 100% passing the 1-inch sieve.

PAVEMENT

Pavement materials should be in accordance with the requirements of the Maricopa Association of Governments Standard Specifications for Asphalt Concrete (Section 710, Type C-3/4).

PART III
EXECUTION

SITE GRADING

The following recommendations are presented for grading and excavation along the culvert alignment. All phases of earthwork should be performed under observation and testing directed by the geotechnical engineer.

1. Excavation should be performed with as little disturbance to the base of the excavation as possible. We recommend the use of a bucket from above the sides of the excavation instead of a front end loader from within the excavation. The base of excavation should be at least two feet below the bottom of the culverts.

2. The shallow groundwater conditions along most of the alignment will require the employment of a dewatering system. We recommend the use of a sump system within the excavation.

3. With the water level at or below the base of the excavation, the base should be cleaned of all organic contaminants, debris, utilities or subsurface facility remnants and any loose or disturbed soils encountered. The cleaned surface should be observed for evidences of debris laden soils, disturbance, concealed facility remnants, or loose zones requiring additional removal. The sides should be braced or sloped in accordance with the recommendations under "Excavation Conditions".

4. Place the granular backfill previously described under "Fill Materials" at the base of the excavation to bring the excavation back to the level of the bottom of the culverts.

5. With the water level at or below the top of the granular backfill, construct cast-in-place culverts or place precast culverts.

6. Backfill against the sides of the culverts with imported fill materials as previously described under "Fill Materials". We recommend that only manual compaction equipment be used within 5 feet of the culvert walls. Backfill should be placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment used.

7. Natural site soils may be placed above the top of the culverts in non-pavement areas. Where paved surfaces are to be placed over the excavation, imported fill soils should be used. All backfill should be placed and compacted in horizontal lifts of thickness compatible with the compaction equipment used.

8. Compaction of cleaned exposed soil, backfill, and granular bedding materials should be accomplished to the following density criteria:

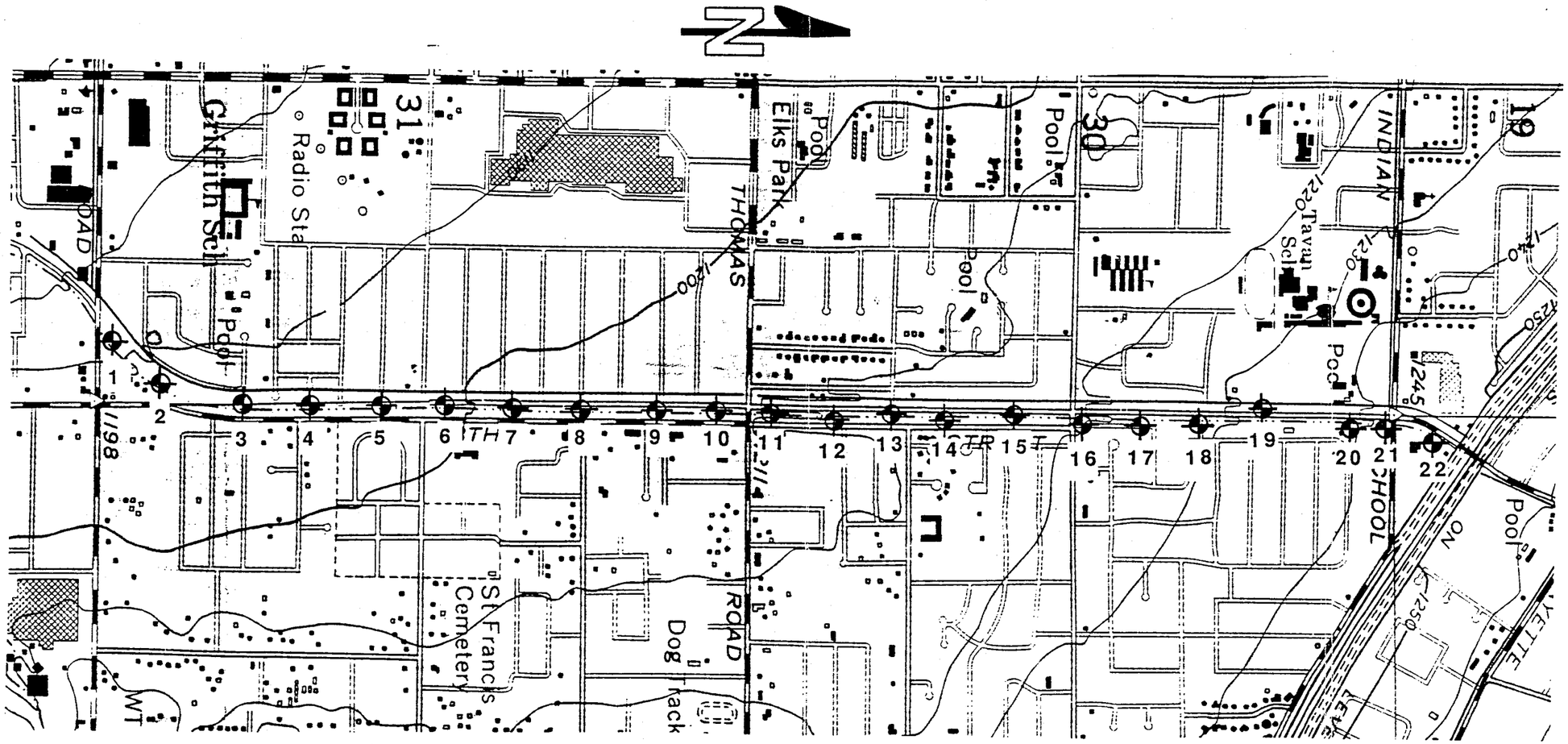
Material	Percent Compaction (ASTM D698)
Exposed Soil below base of culvert	95 min.
Granular Bedding below culvert	100 min.
Imported backfill against culvert walls	100 min.
Backfill above culvert top	
Non-paved areas	90 min.
Paved areas	95 min.

Compaction of exposed site soil beneath the base of the excavation should be performed with soils uniformly mixed at a moisture content between optimum and optimum +3 percent. Compaction of imported fill soils with low expansive potentials should be accomplished within the range of optimum moisture content -1 to +3 percent. Compaction of exposed soil and fill material below asphaltic pavement should be accomplished at a moisture content 2 percent below optimum, or lower.

Natural undisturbed soils or compacted soils subsequently disturbed or removed by construction operations should be replaced with materials compacted as specified above.

PAVING

Placement requirements for paving should be in accordance with the Maricopa Association of Governments' Specifications for Asphalt Concrete Pavement (Section 321). Observation and testing should be performed as necessary to verify conformance with these recommended specifications, especially compaction requirements for asphaltic concrete surfacing.



LEGEND:

Test Boring Location

Elevations approximated from Plan and Profile provided by Greiner Engineering

NOTE: Test borings 3, 8, 13, 16, & 21 completed as piezometer wells

Site Plan

Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

APPENDIX A
FIELD RESULTS

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	

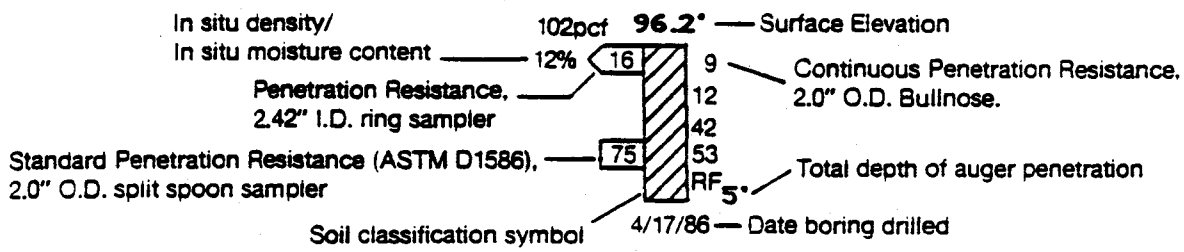
FINE-GRAINED SOIL

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.



PENETRATION RESISTANCE: Blows per foot using 140 lb. hammer with 30" free-fall unless otherwise noted.

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).

Project No. 90-0863

LEGEND OF SOIL TYPES

ASPHALT CONCRETE, AGGREGATE BASE

CLAYEY SAND/SANDY CLAY (SC, SC/CL, CH, CL); light brown to reddish brown; sand fraction primarily fine to medium; occasional to frequent zones with traces to some small gravel, predominantly consisting of subangular to angular granite fragments; scattered gravelly lenses or layers; medium dense to dense; moderate to high plasticity; variable light to heavy calcareous cementation; damp to wet.

1-11-91

Approximate groundwater level encountered during drilling.

1-11-91

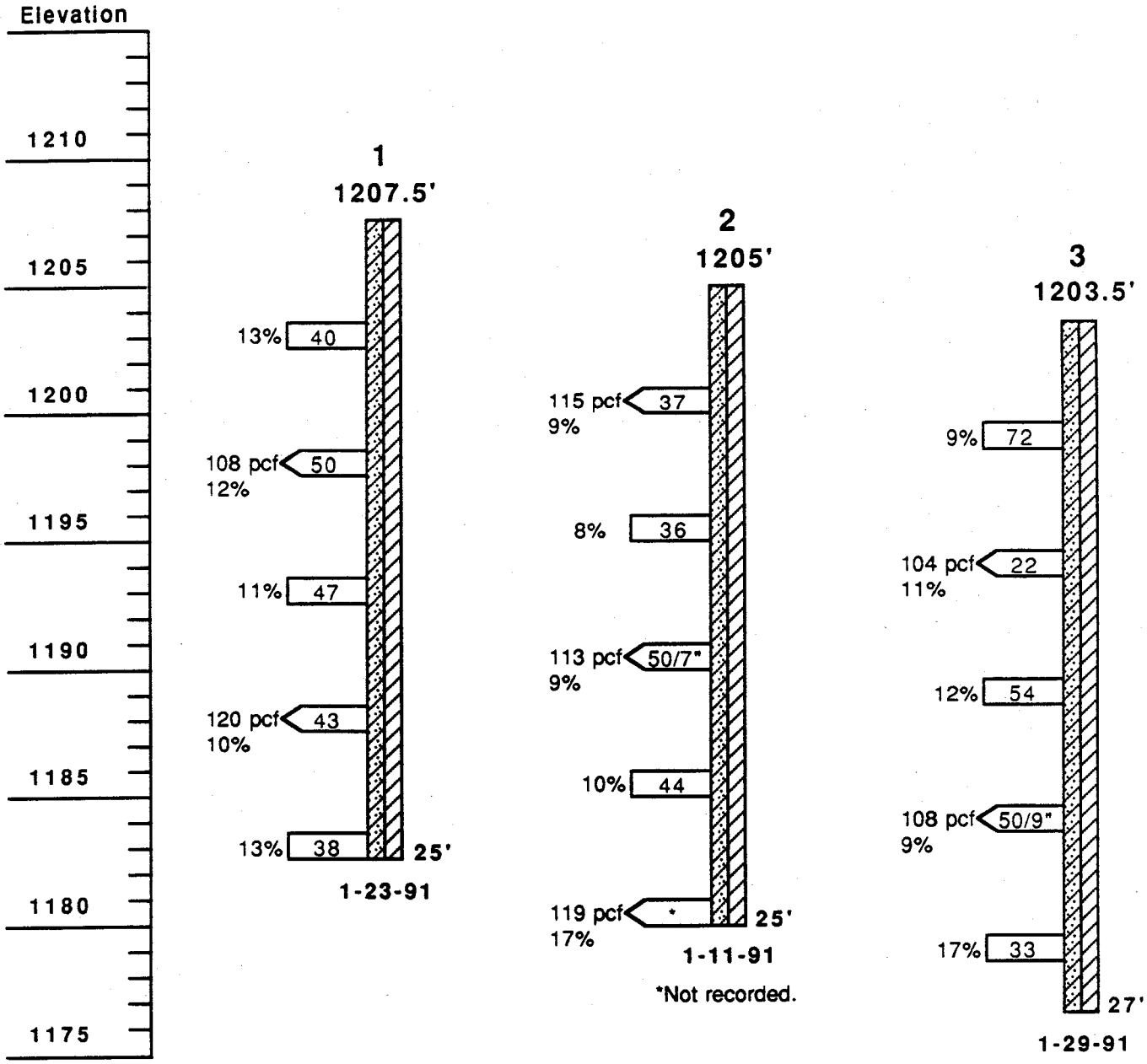
Stabilized groundwater level observed in temporarily open hole or piezometer.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



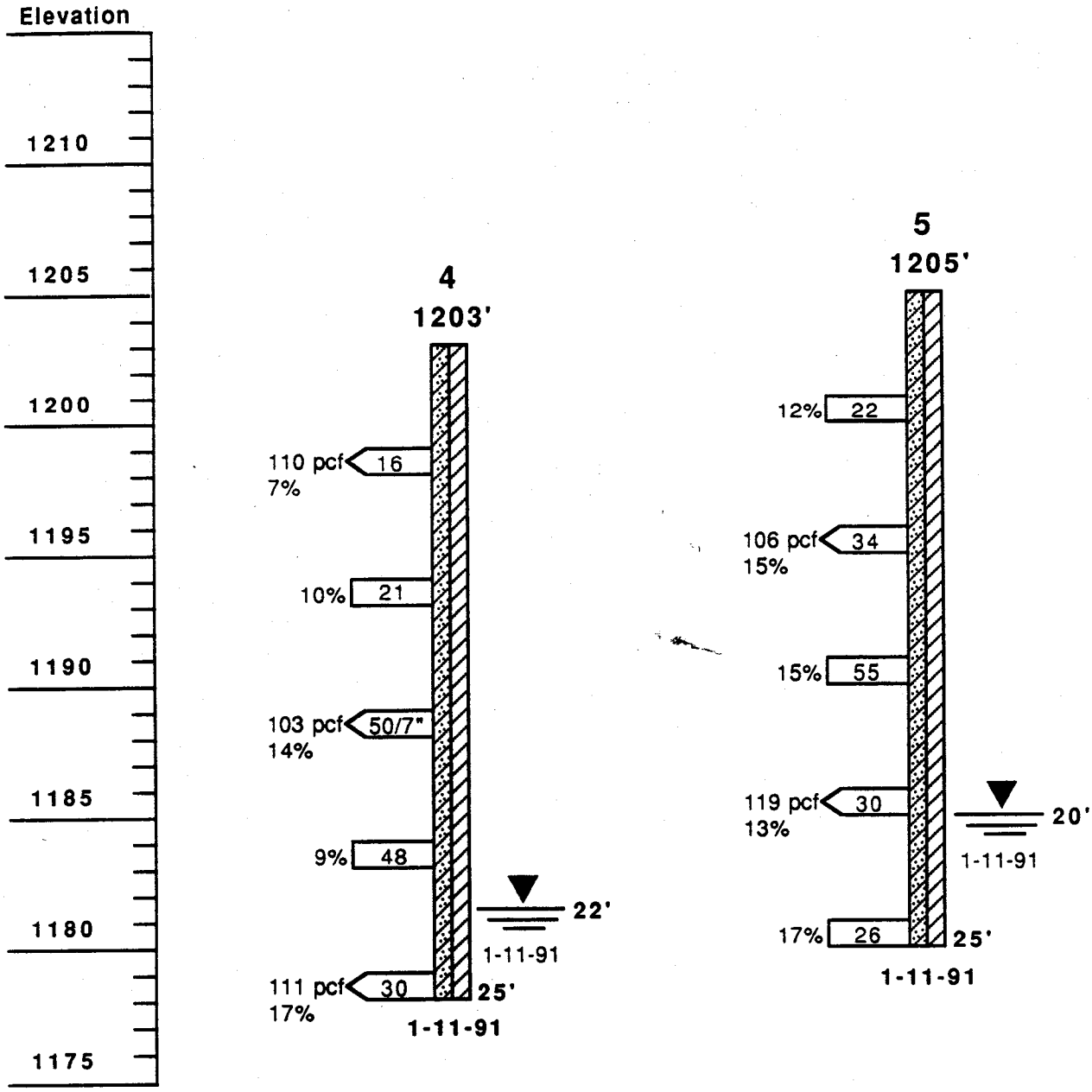
No free groundwater was encountered in the test borings during drilling unless otherwise noted.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



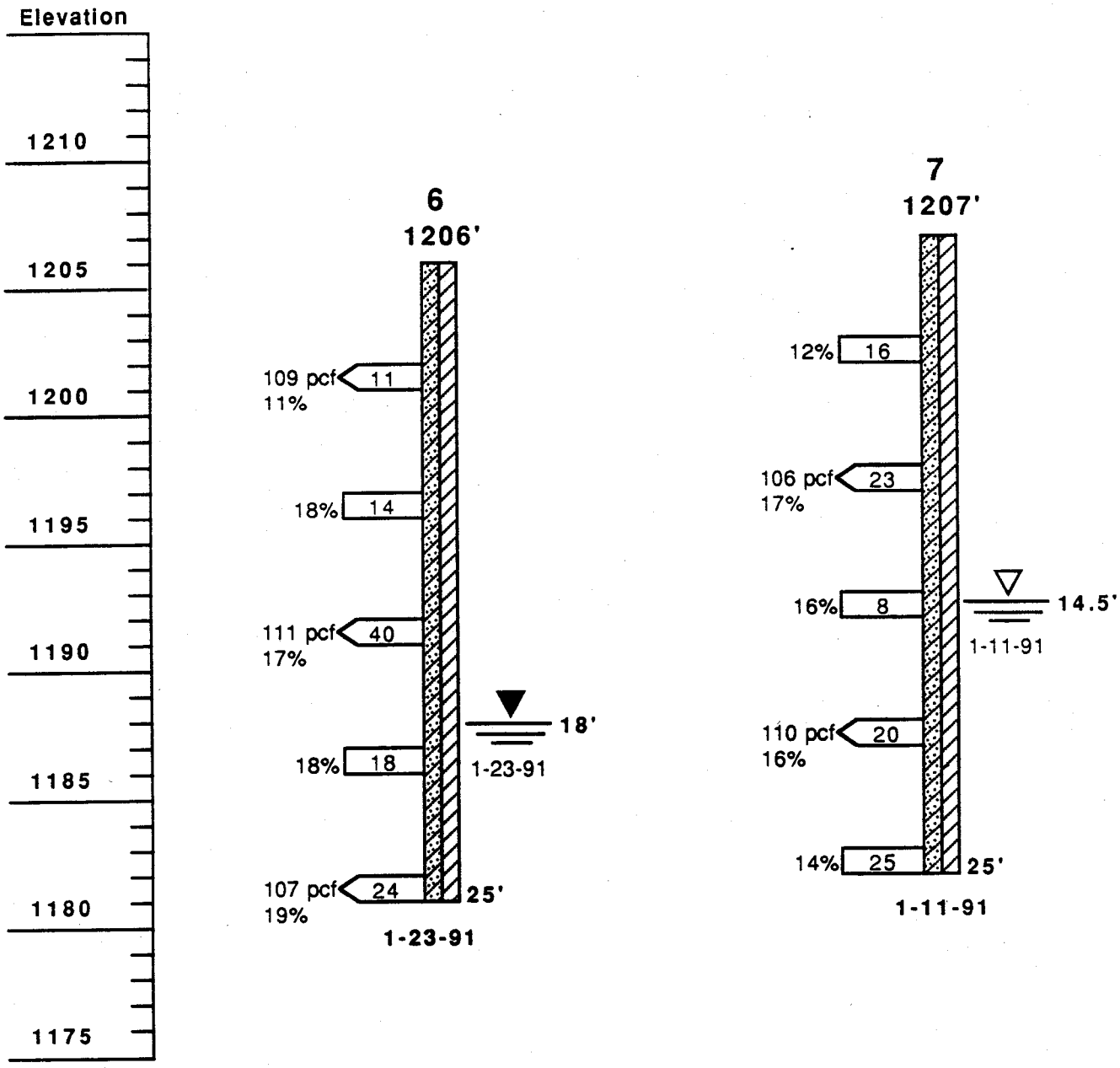
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



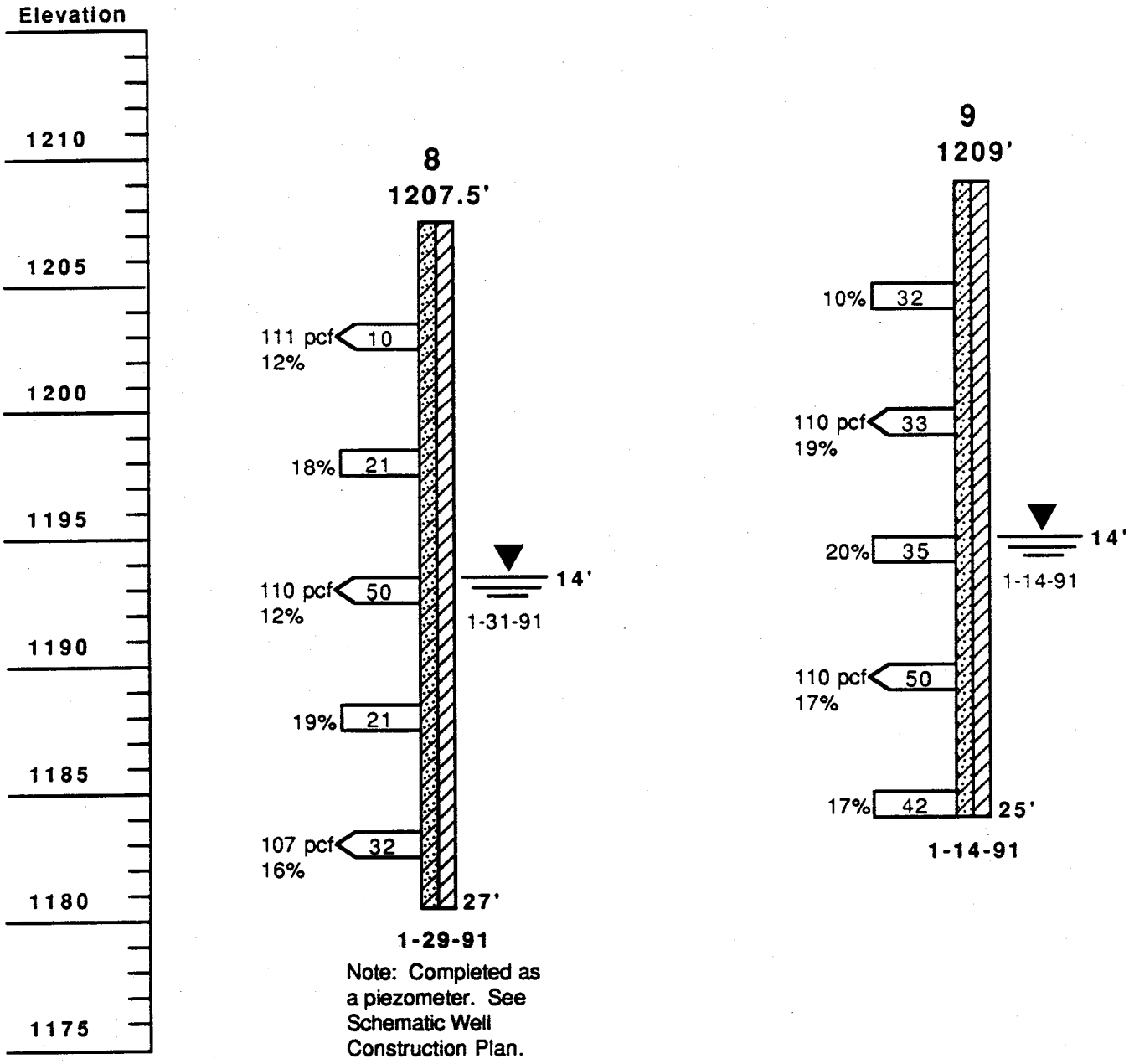
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



Note: Completed as a piezometer. See Schematic Well Construction Plan.

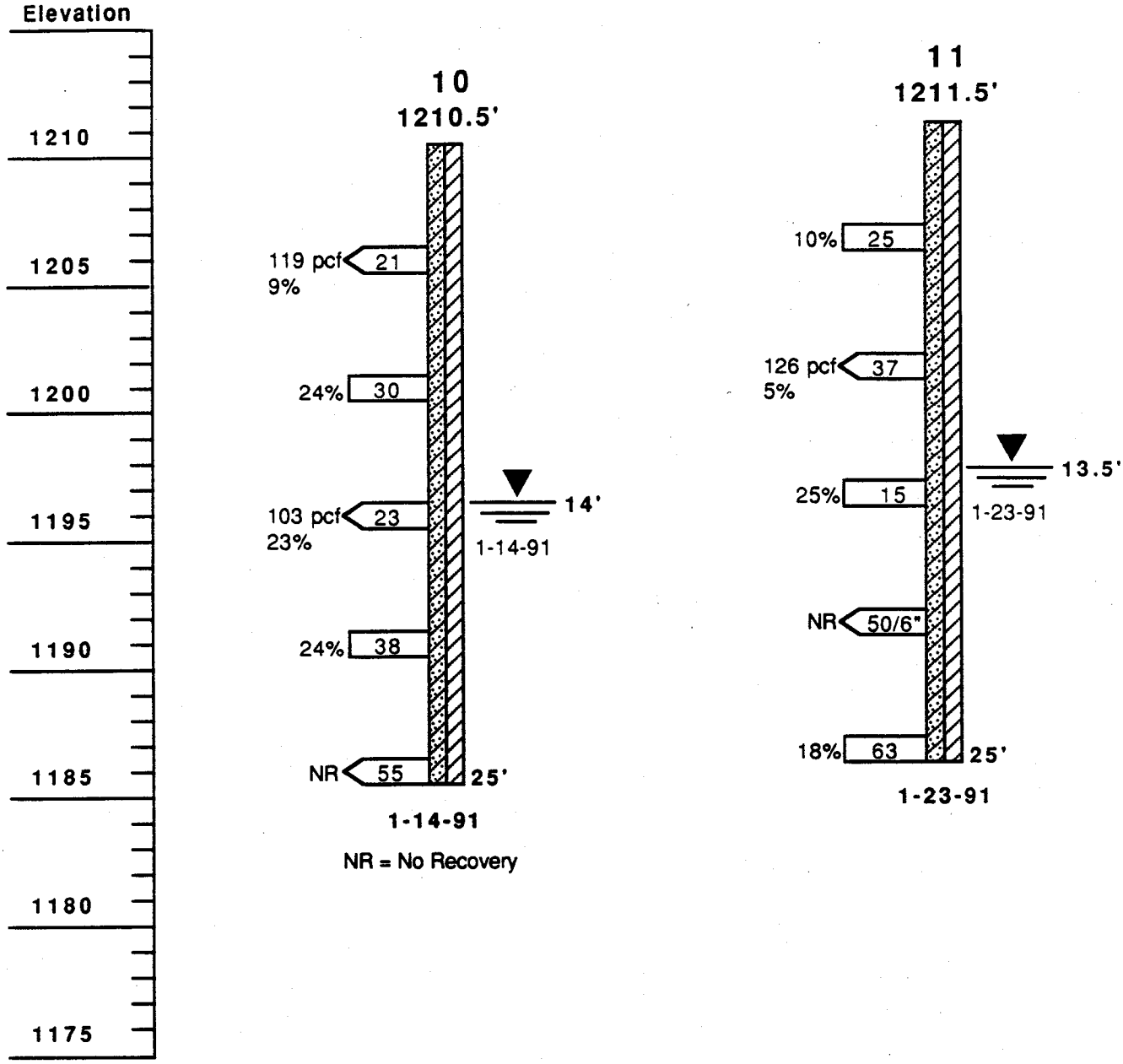
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



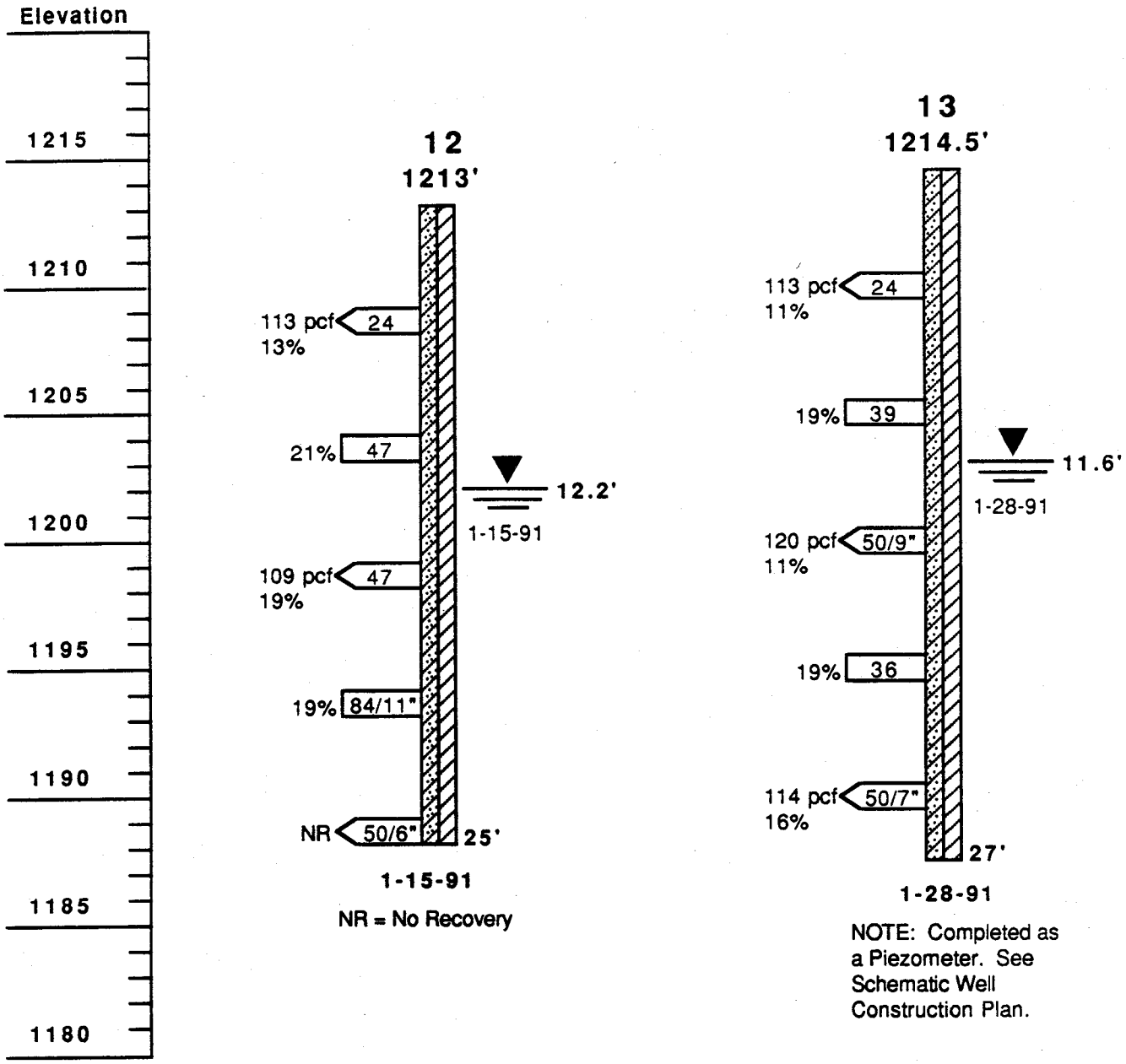
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



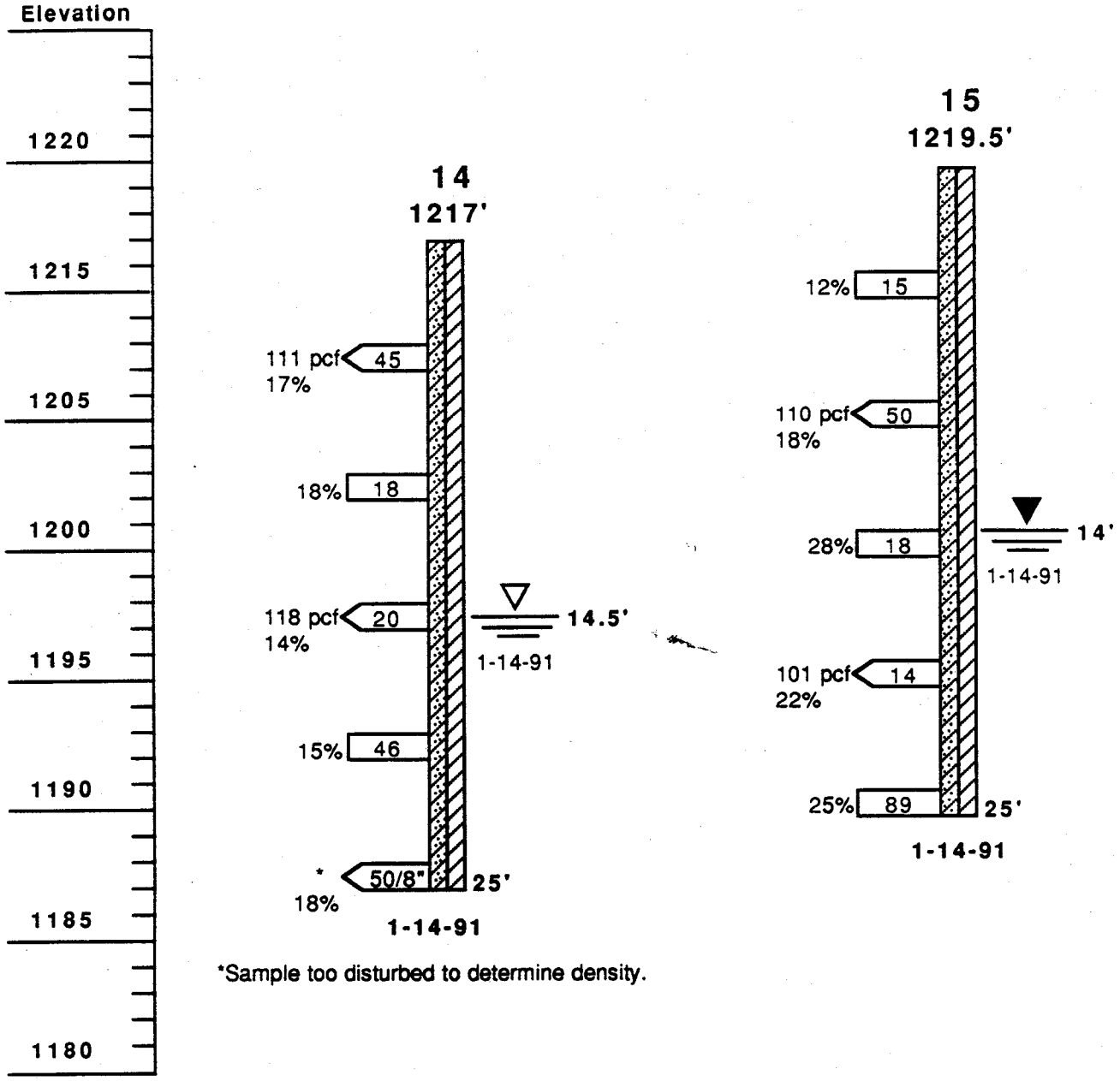
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



*Sample too disturbed to determine density.

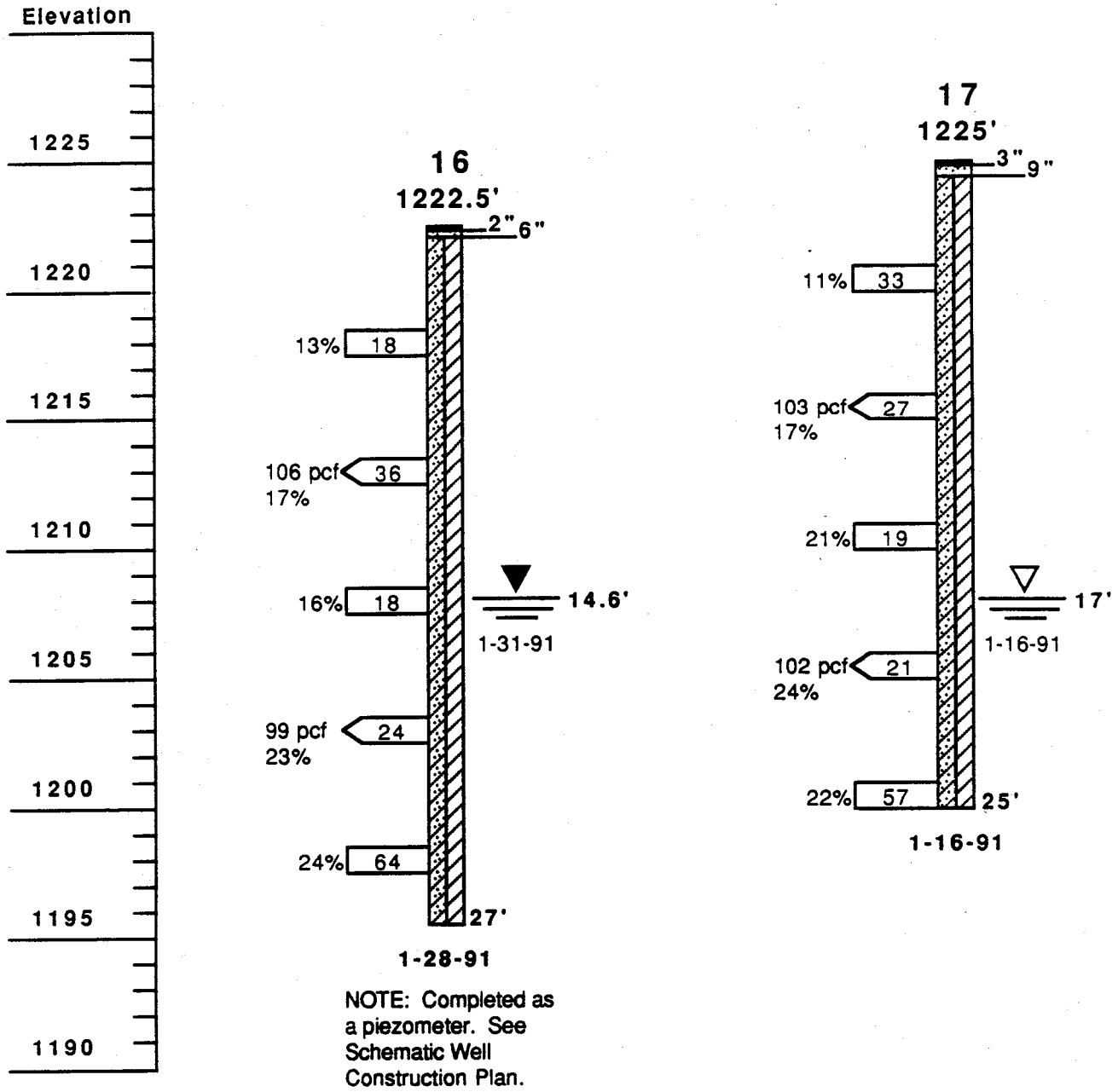
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



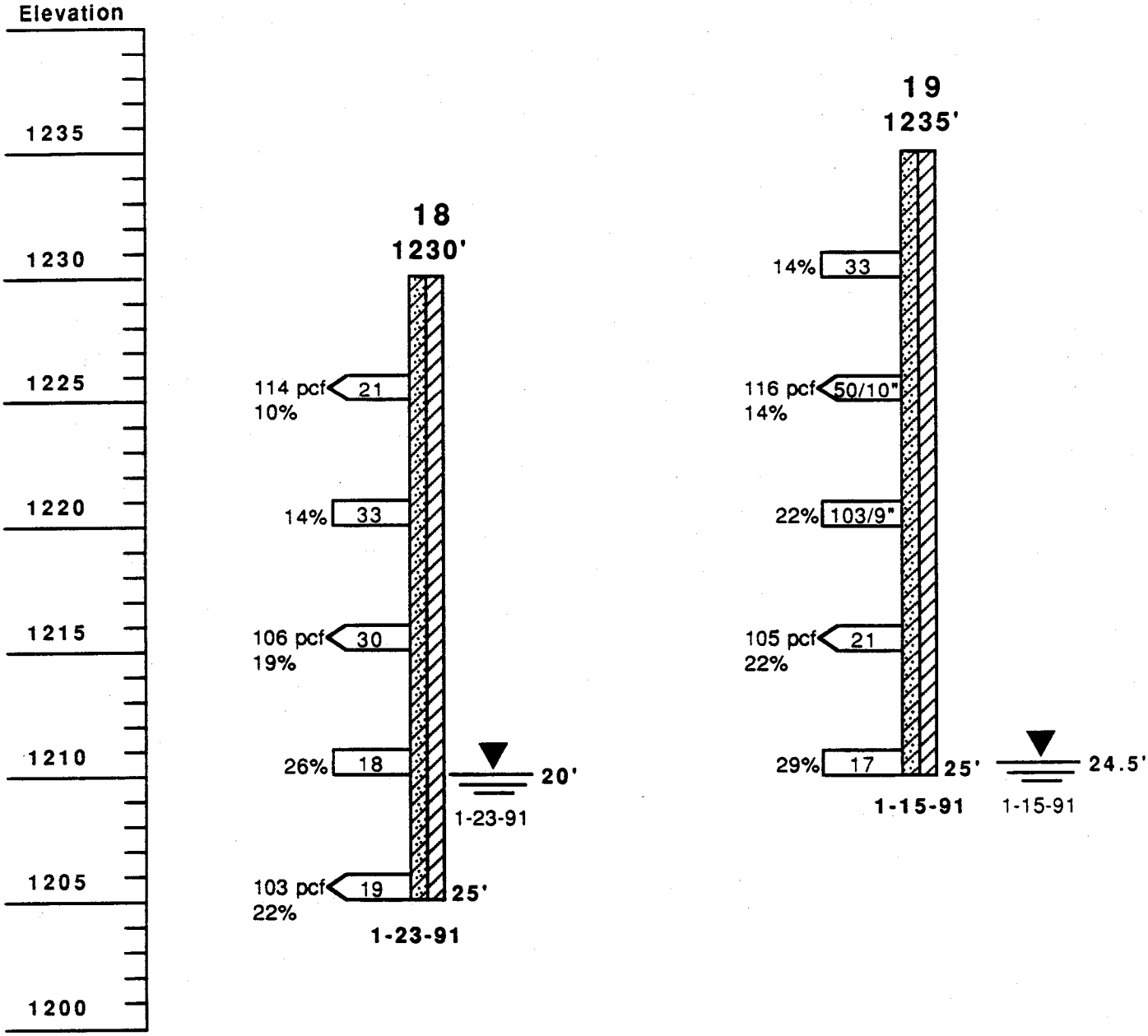
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



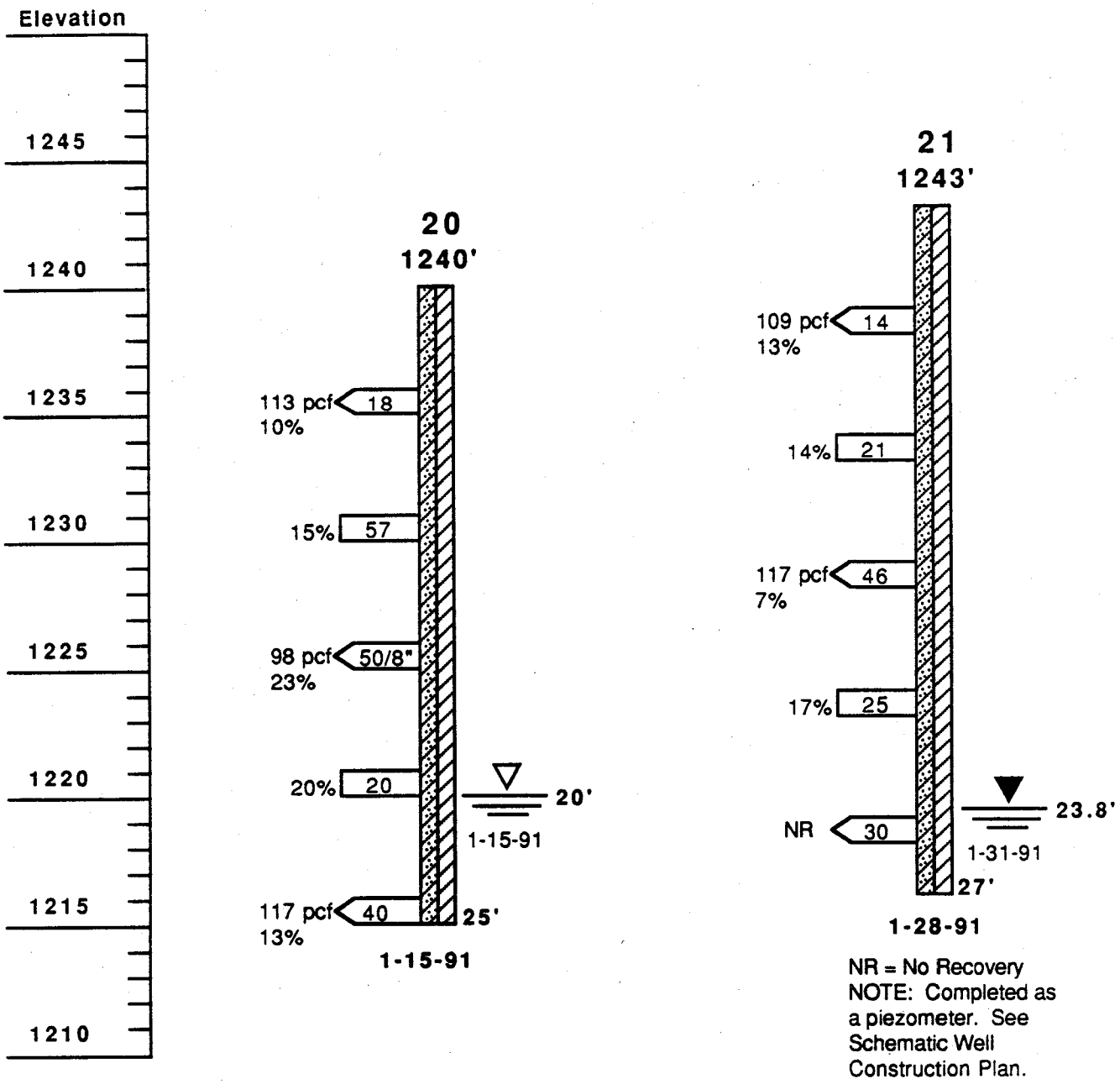
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



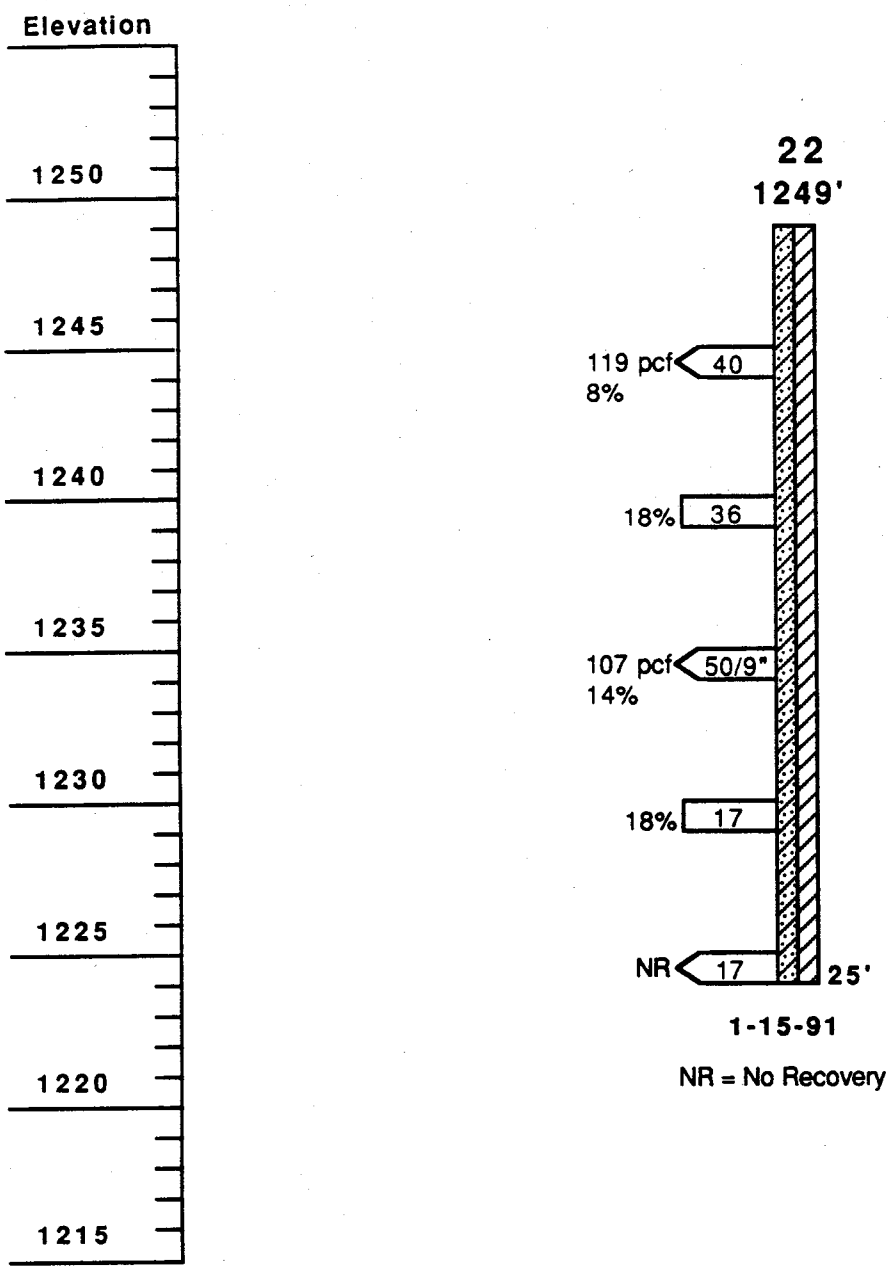
Free groundwater was encountered in the test borings during drilling as noted above.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



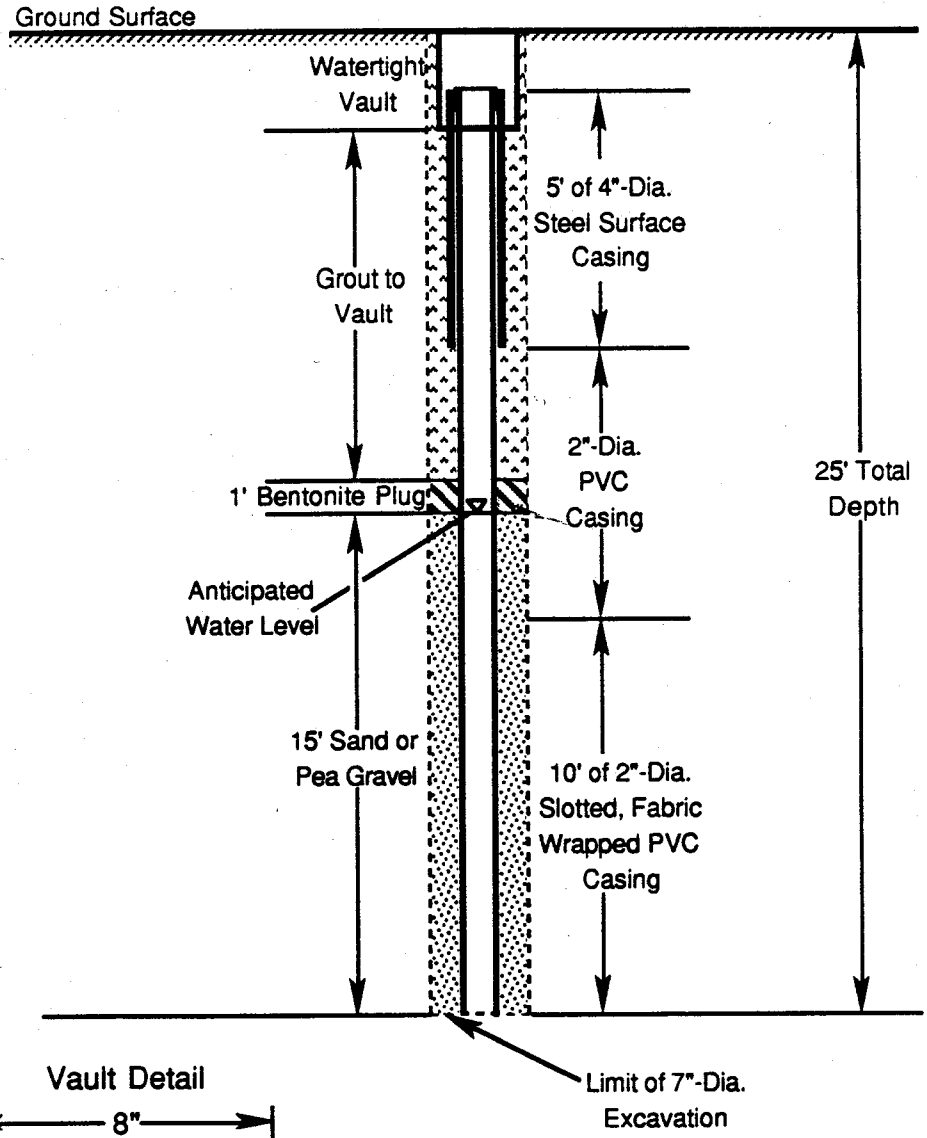
NR = No Recovery

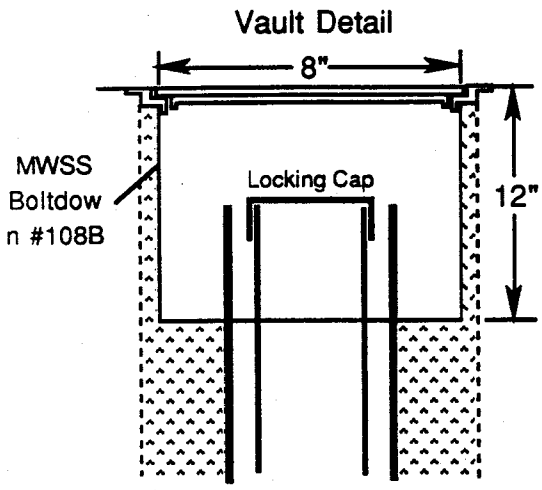
No free groundwater was encountered in the test borings during drilling unless otherwise noted.

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

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.

Project No. 90-0863
Thomas-Hartig & Associates





Not to Scale

Schematic Well Construction Plan

PROJECT NO. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON FIELD RESISTIVITY TESTS

DESCRIPTION: Date: 12/30/90 and 1/2/91

Location: Noted Below

Material: Subsurface Soil

Performed By: TH/R. Thompson

TESTED: Field electrical resistivity using the 4-probe method.

RESULTS:

Test Boring	Resistivity (ohm-cm) 0 - 12 ft.	0 - 25 ft.
1	2940	2390
2	2320	3350
3	3060	3020
4	3290	2300
5	3080	2300
6	3700	2680
7	5790	3500
8	4760	3690
9	5520	5170
10	7010	4400
11	5330	3540
12	2550	2820
13	2870	2730
14	2670	2300
15	1910	2630
16	2990	2780
17	2830	9580
18	2480	2110
19	3260	2630
20	3680	3160
21	3150	3590
22	3150	3250

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

APPENDIX B
LABORATORY RESULTS

REPORT ON LABORATORY TESTS

SAMPLE: Date: 1/28/91

Source: Noted Below

Type: Bulk Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Gradation and Plasticity Index

RESULTS

			Sieve Size -					Accumulative % Passing						*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
1; 0' - 8'	38	14	36	44	49	53	59	68	79	95	95	100		SC
2; 4' - 9'	46	21	49	57	62	66	71	80	90	100				SC/CL
3; 14' - 15'	56	21	42	52	58	64	71	81	90	100				SM
4; 10' - 17'	59	31	38	45	49	53	58	68	80	100				SC
5; 17' - 24'	58	32	33	40	45	49	61	64	78	100				SC
6; 16' - 24'	54	29	47	53	57	61	66	75	87	100				SC/CH
7; 8' - 16'	61	33	48	55	60	64	70	78	90	100				SC/CH
8; 17' - 24'	56	29	48	56	62	68	75	86	95	100				SC/CH
9; 0' - 8'	39	19	33	41	46	51	59	73	87	100				SC
10; 7' - 14'	61	35	38	44	48	53	59	69	84	100				SC
11; 7' - 17'	51	28	28	33	37	42	49	63	82	100				SC
12; 8' - 14'	67	40	49	56	61	65	71	81	92	100				SC/CH
13; 9' - 16'	54	28	52	60	66	71	78	87	96	100				CH/SC
14; 0' - 9'	39	20	34	41	46	51	59	71	87	100				SC
15; 18' - 24'	56	31	58	67	72	76	80	87	95	100				CH
16; 0' - 8'	33	12	38	47	51	55	61	71	84	97	100			SC
17; 7' - 14'	58	28	34	39	44	49	59	75	92	100				SC

*Unified Soil Classification

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON LABORATORY TESTS

SAMPLE: Date: 1/28/91

Source: Noted Below

Type: Bulk Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Gradation and Plasticity Index

RESULTS

			Sieve Size -					Accumulative % Passing						*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
18; 8' - 16'	51	25	57	65	69	72	78	86	96	100				CH
19; 16' - 24'	57	28	52	59	64	68	75	84	92	100				CH/SC
20; 0' - 8'	42	19	41	49	53	57	65	77	92	100				SC
21; 8' - 16'	62	33	46	52	56	60	67	77	89	100				CH
22; 0' - 8'	52	25	31	36	39	42	49	64	83	100				SC

*Unified Soil Classification

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source Test Boring 5; 19 - 20'

Type Driven ring sample; 119 pcf dry density; 13% field moisture

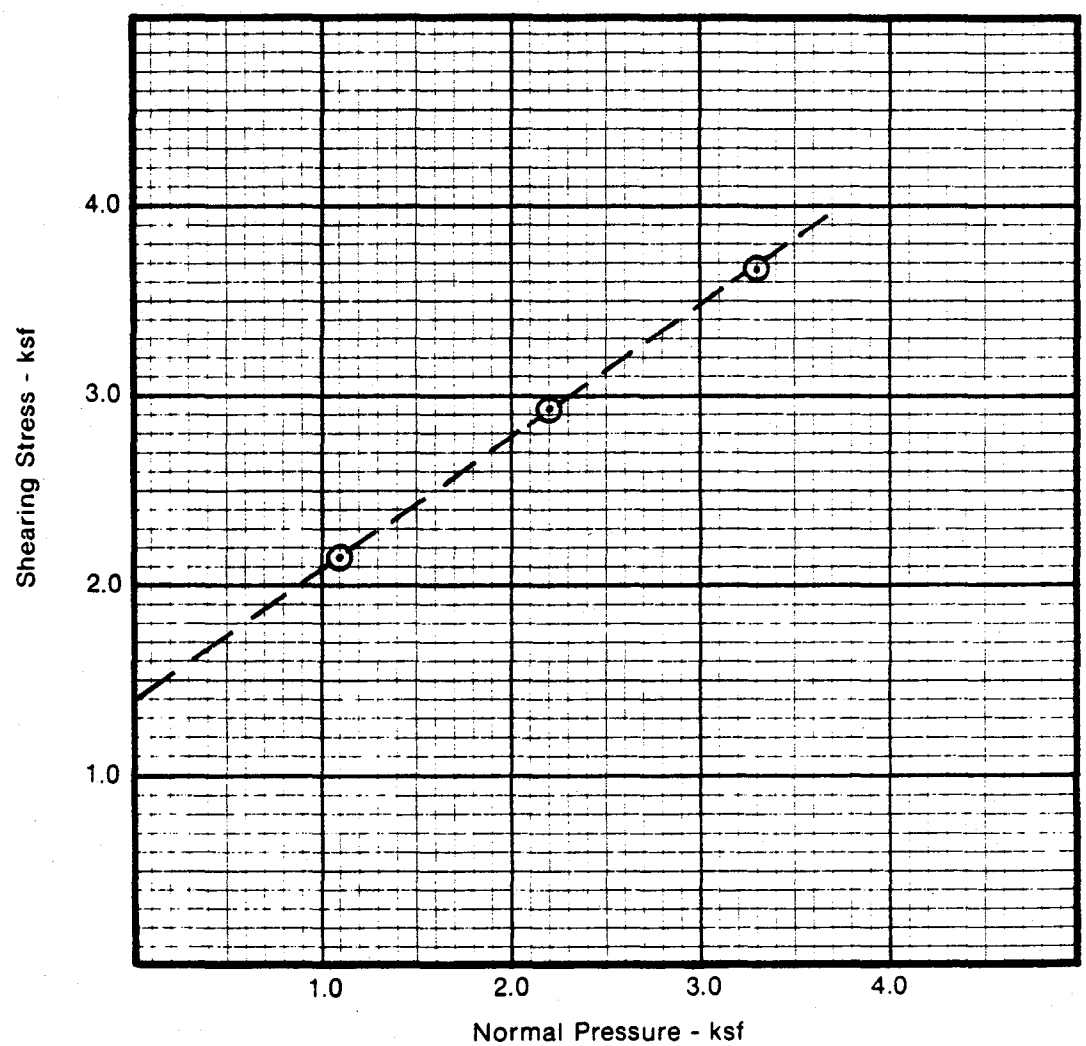
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 35° Cohesion (c) = 1.4 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source Test Boring 7; 9 - 10'

Type Driven ring sample; 106 pcf dry density; 17% field moisture

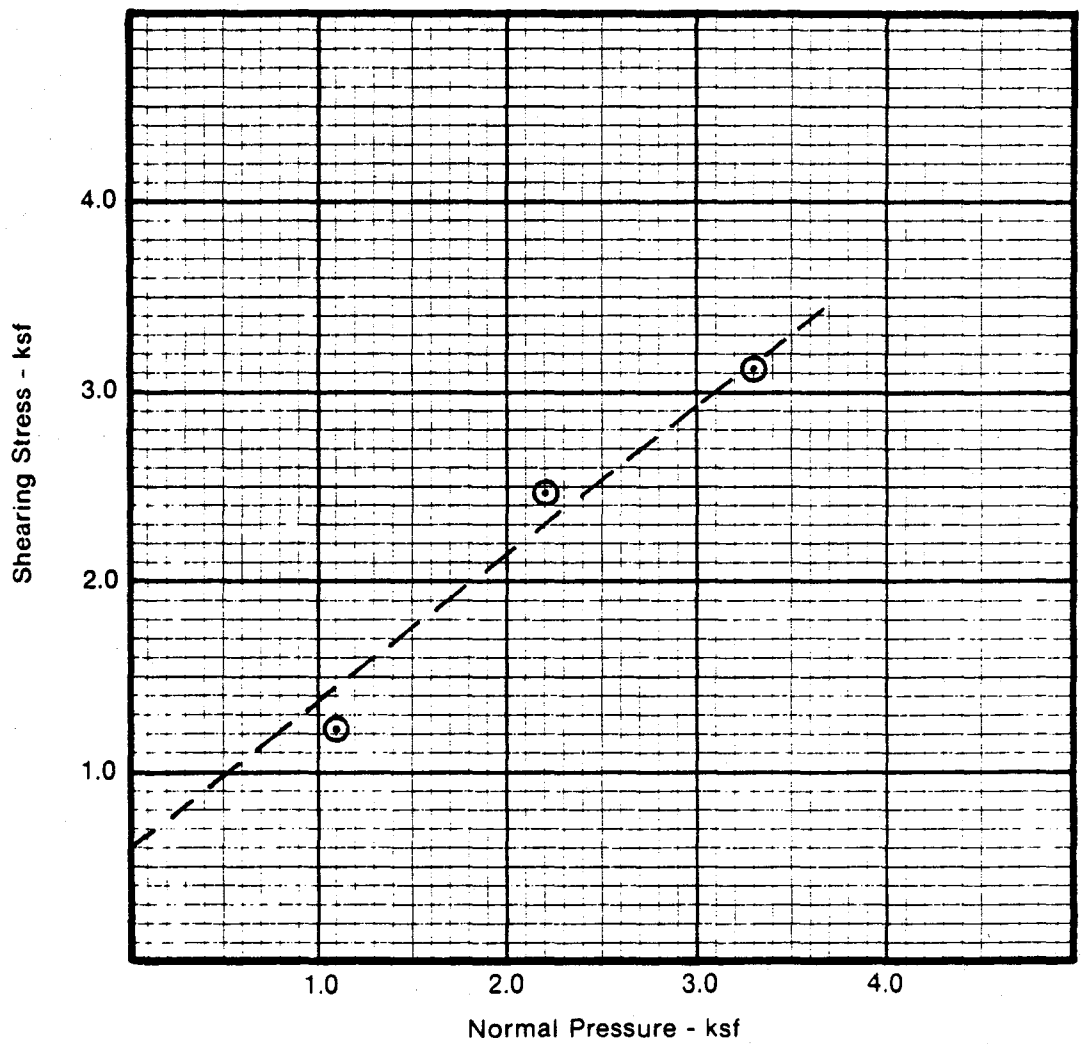
Material Clayey Sand/ Sandy Clay (SC/CH)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 38° Cohesion (c) = 0.6 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 12; 4 - 5'

Type Driven ring sample; 113 pcf dry density; 13% field moisture

Material Clayey Sand (SC)

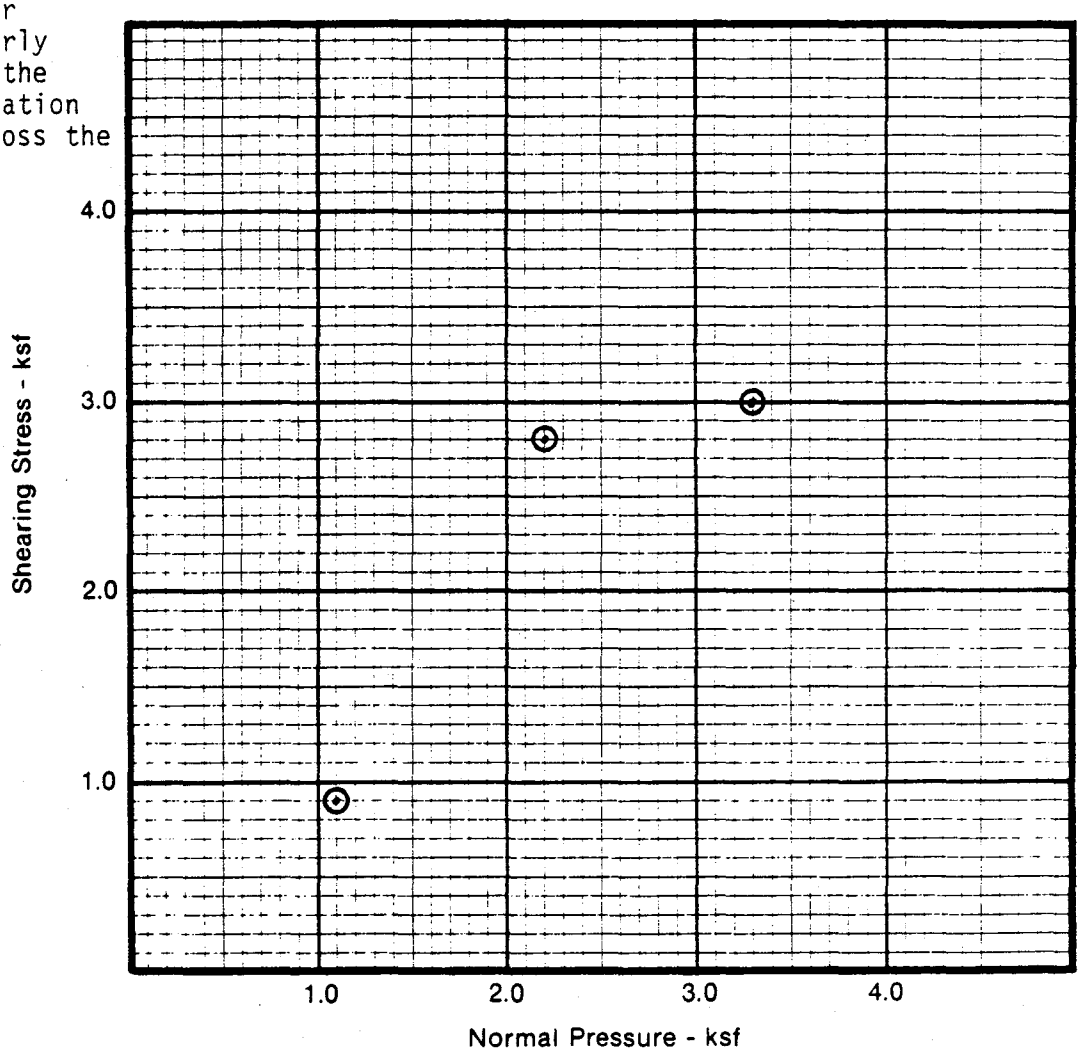
Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = Cohesion (c) =

NOTE: The shear envelope is poorly defined due to the variable cementation and nodules across the failure plane.



Project No. 90-0863

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 15; 9 - 10'

Type Driven ring sample; 110 pcf dry density; 18% field moisture

Material Clayey Sand/ Sandy Clay (SC/CH)

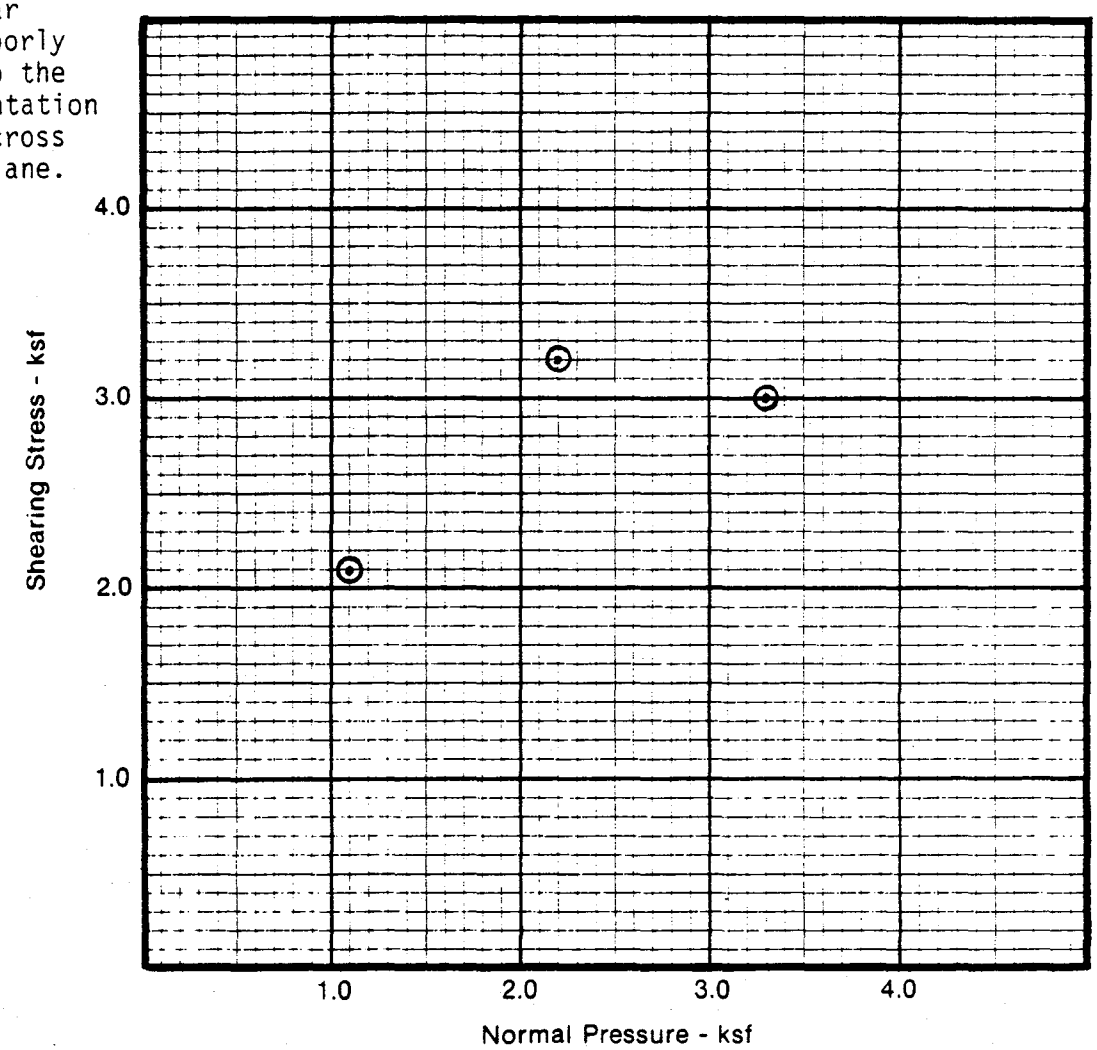
Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = Cohesion (c) =

NOTE: The shear envelope is poorly defined due to the variable cementation and nodules across the failure plane.



Project No. 90-0863

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

Source Test Boring 20; 24 - 25'

Type Driven ring sample; 117 pcf dry density; 13% field moisture

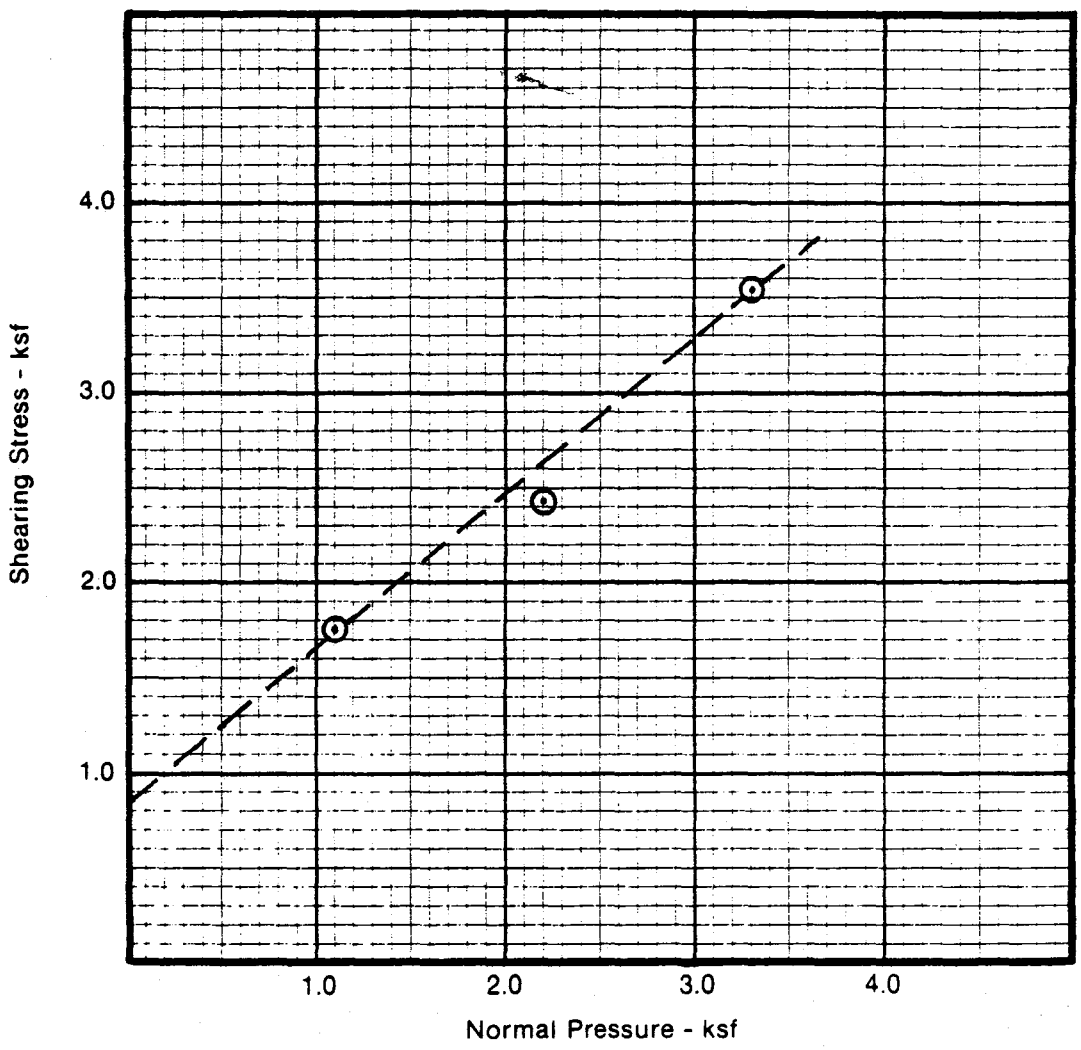
Material Sandy Clay (SC)

Sampled By TH/Thompson

TESTED: Direct Shear with sample immersed.

RESULTS:

Friction Angle (Ø) = 39° Cohesion (c) = 0.9 ksf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

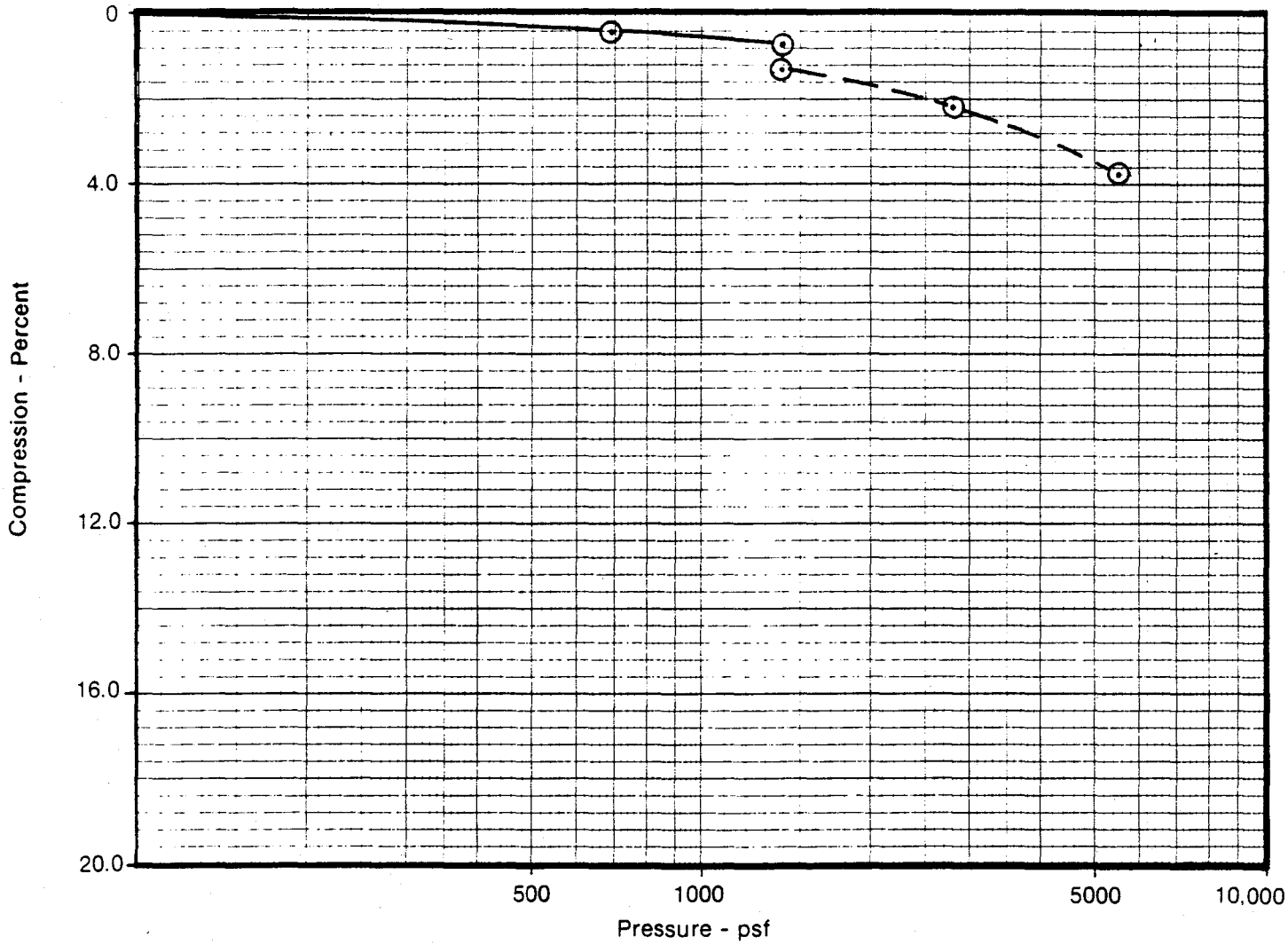
Source Test Boring 2; 4 - 5'

Type Driven ring sample; 115 pcf dry density; 9% field moisture

Material Clayey Sand/ Sandy Clay (SC/CL)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 1385 psf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/91

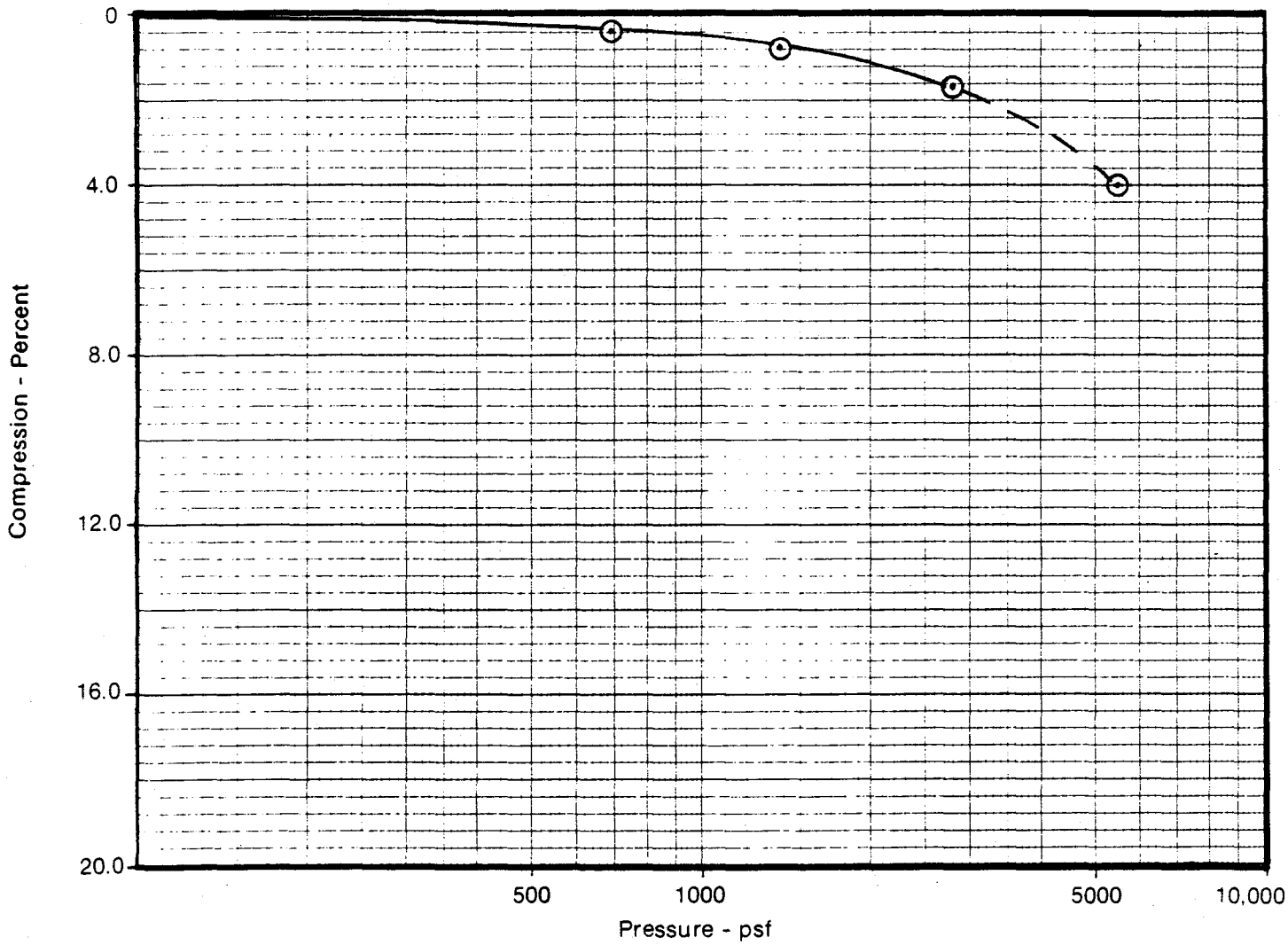
Source: Test Boring 7; 9 - 10'

Type: Driven ring sample; 106 pcf dry density; 17% field moisture

Material: Clayey Sand/Sandy Clay (SC/CH)

Sampled By: TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

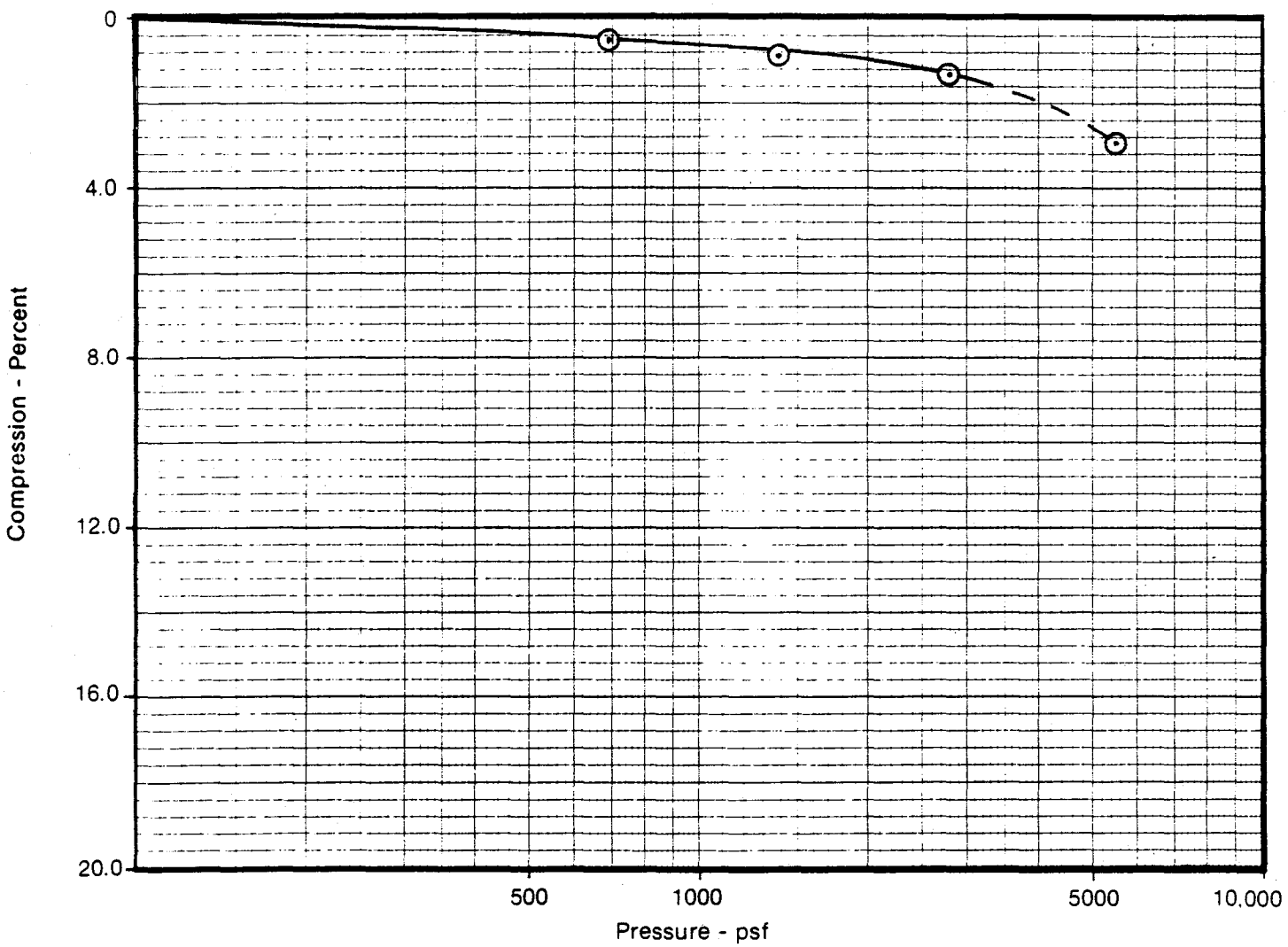
Source Test Boring 10; 14 - 15'

Type Driven ring sample; 103 pcf dry density; 23% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

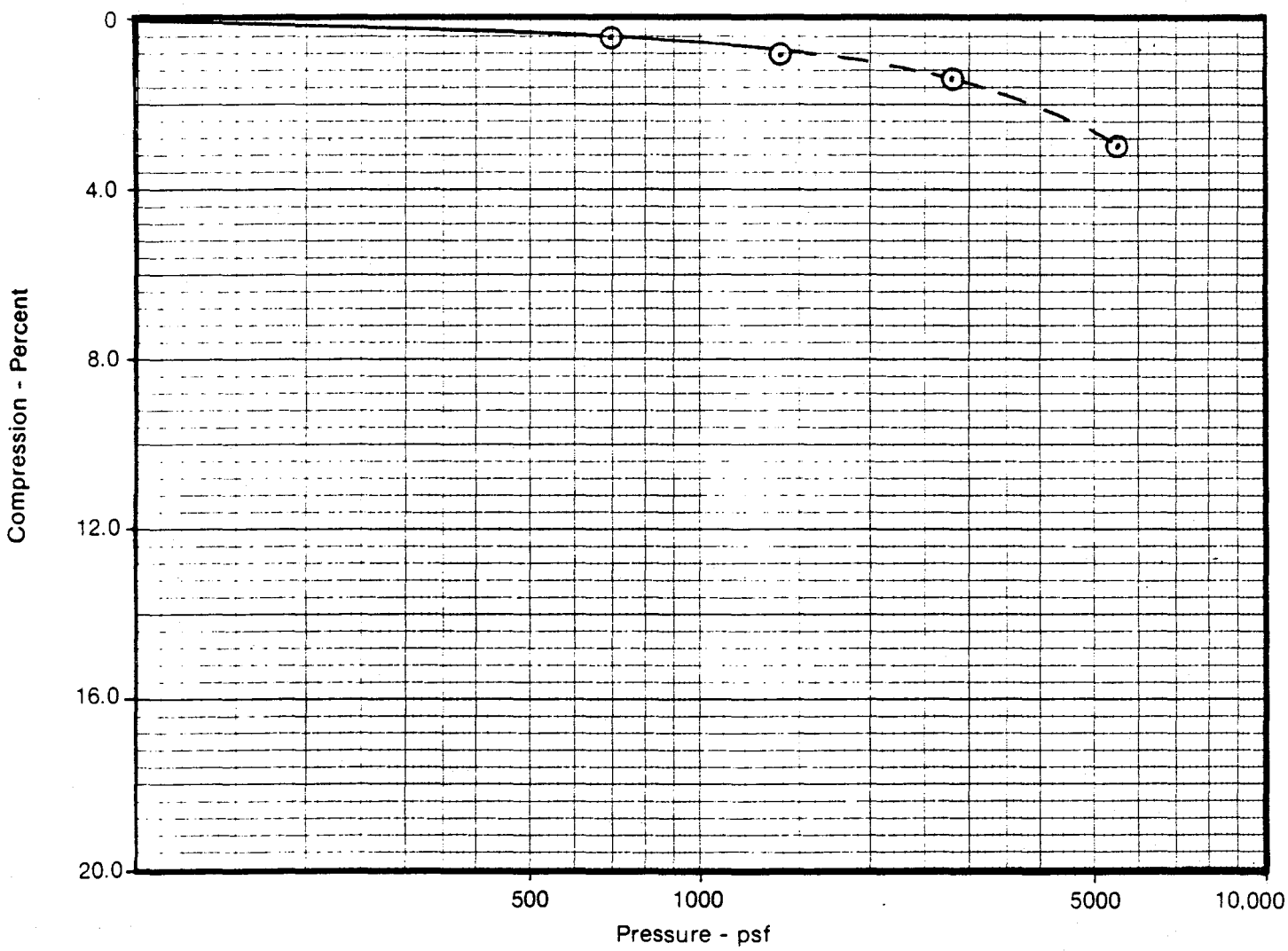
Source Test Boring 17; 9 - 10'

Type Driven ring sample; 103 pcf dry density; 17% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 1385 psf



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/28/91

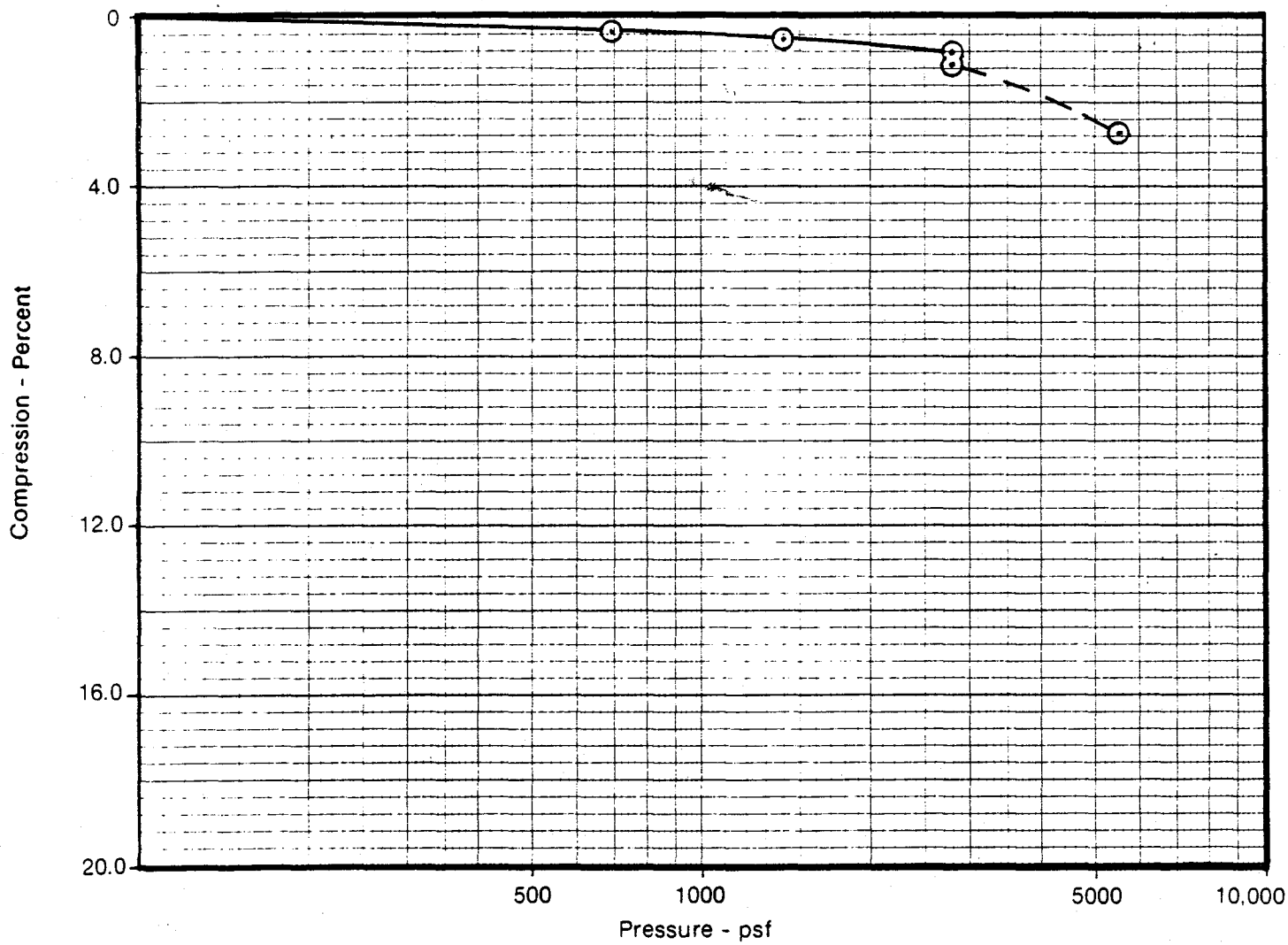
Source Test Boring 22; 4 - 5'

Type Driven ring sample; 119 pcf dry density; 8% field moisture

Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON pH, SOLUBLE SALTS, SULFATES, & CHLORIDES

SAMPLE: Date: 1/21/91

Source: Noted Below
Type: Bulk Sample
Material: Subsurface Soil
Sampled By: TH/Thompson

TESTED: pH, Soluble Salts, Sulfates, & Chlorides

TEST RESULTS

Sample	pH	Soluble Salts (%)	Sulfates Percent	Chlorides Percent
4; 0 - 3'	8.2	0.510	0.190	0.060
9; 8 - 16'	8.3	0.090	0.006	0.011
15; 8 -16'	8.5	0.180	0.027	0.030
19; 0 - 8'	8.0	0.084	0.012	0.009
22; 16 -24'	8.3	0.085	0.003	0.012

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 2; 0 - 4' & 4 - 9'

Type Bulk Sample

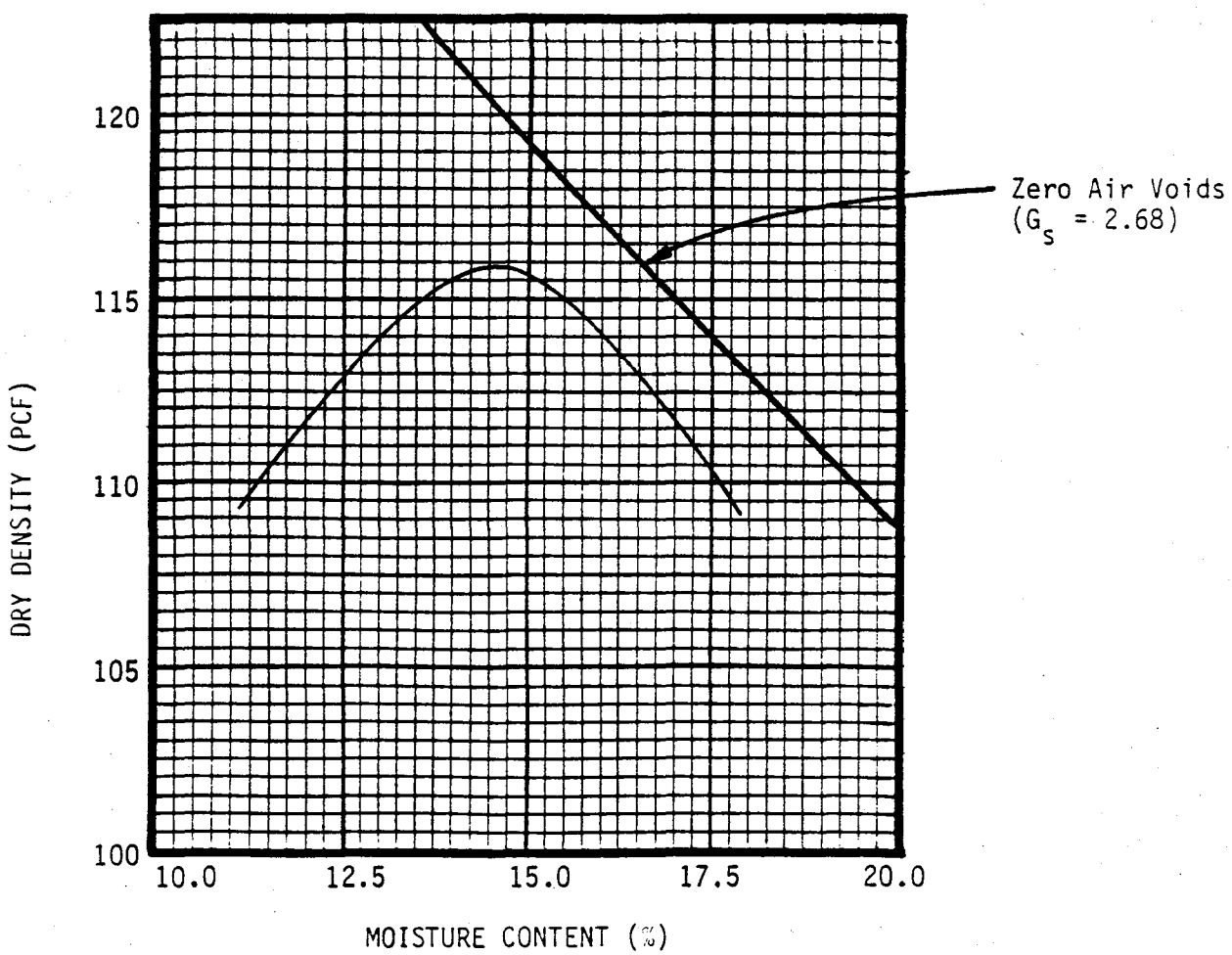
Material Clayey Sand/Sandy Clay (SC/CL)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 116 Optimum Moisture Content (%) 14.5



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/23/91

Source Test Boring 15; 0 - 8'

Type Bulk Sample

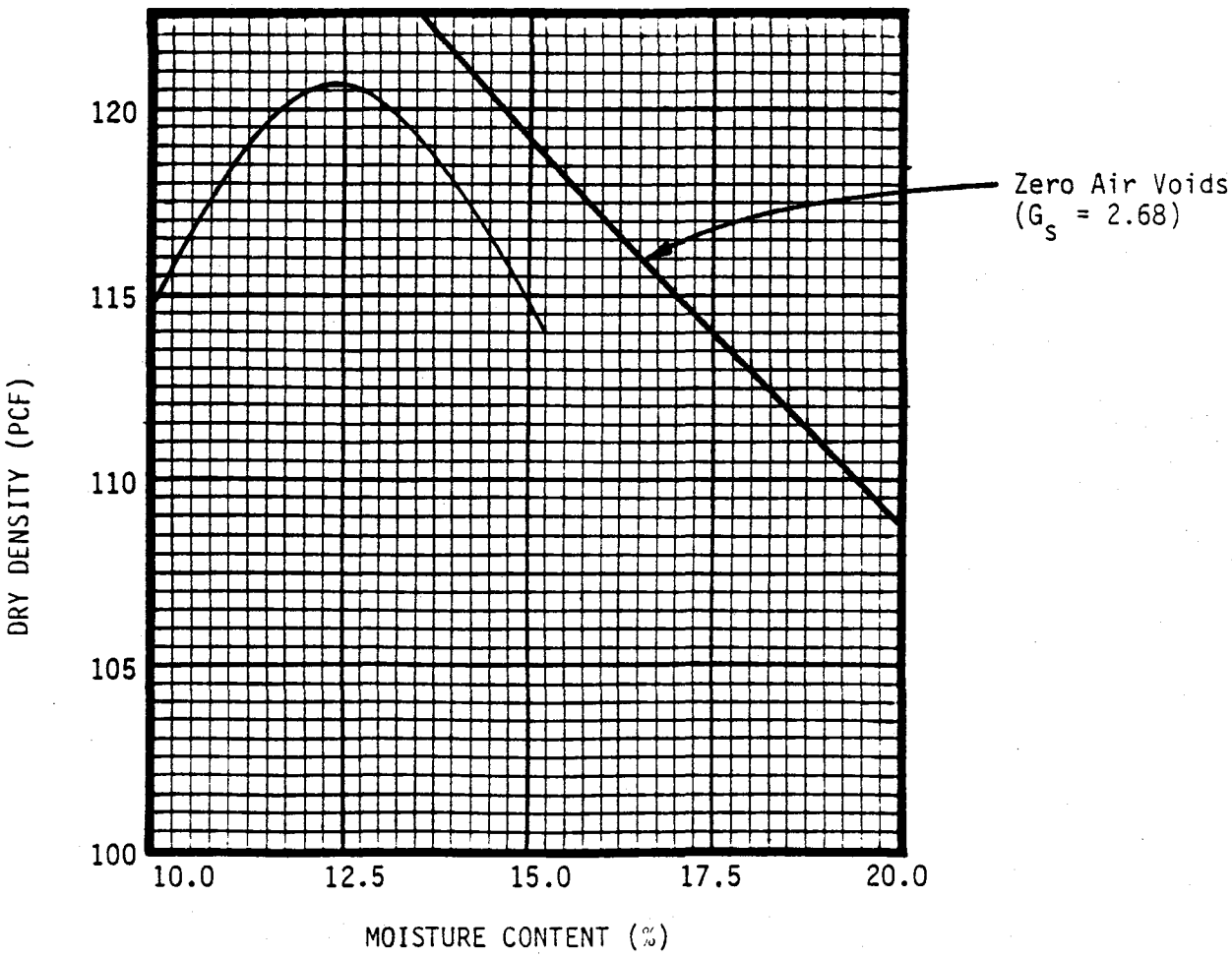
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 121 Optimum Moisture Content (%) 12.5



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/22/91

Source Test Boring 7; 0 - 8'

Type Bulk Sample

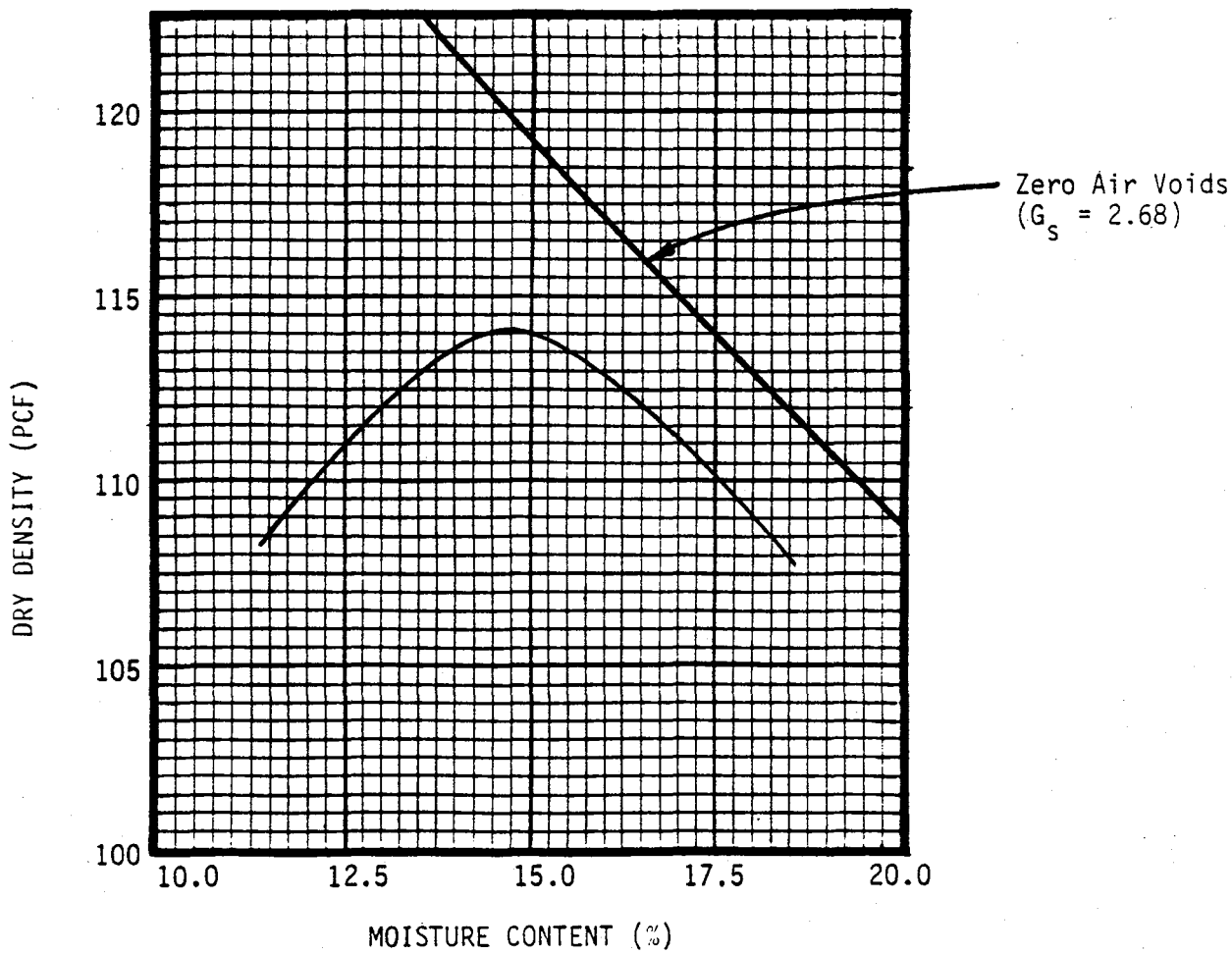
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 114 Optimum Moisture Content (%) 14.5



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 14; 0 - 9' & 9 - 14'

Type Bulk Sample

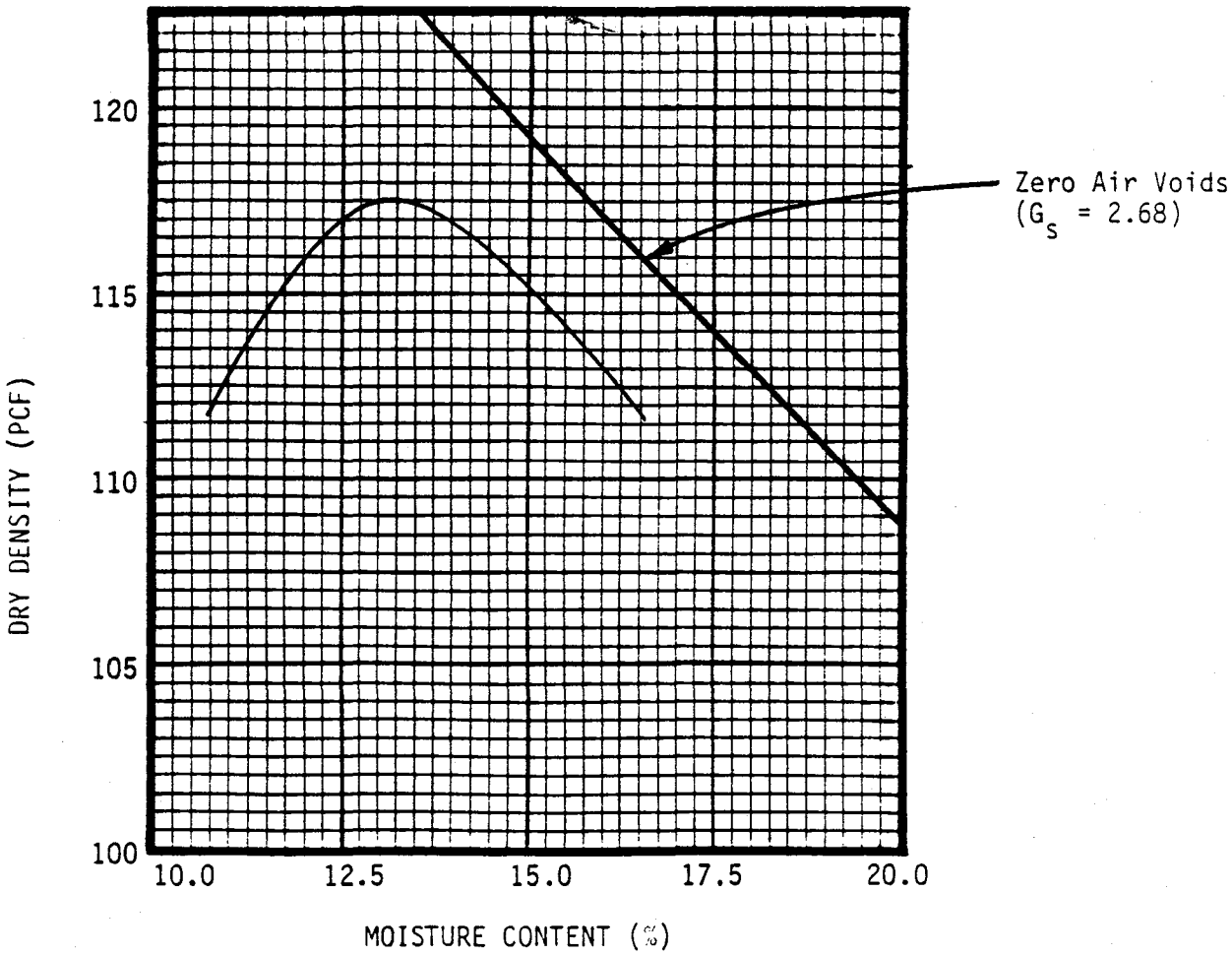
Material Clayey Sand (SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 117.5 Optimum Moisture Content (%) 13



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/24/91

Source Test Boring 19; 0 - 8' & 8 - 16'

Type Bulk Sample

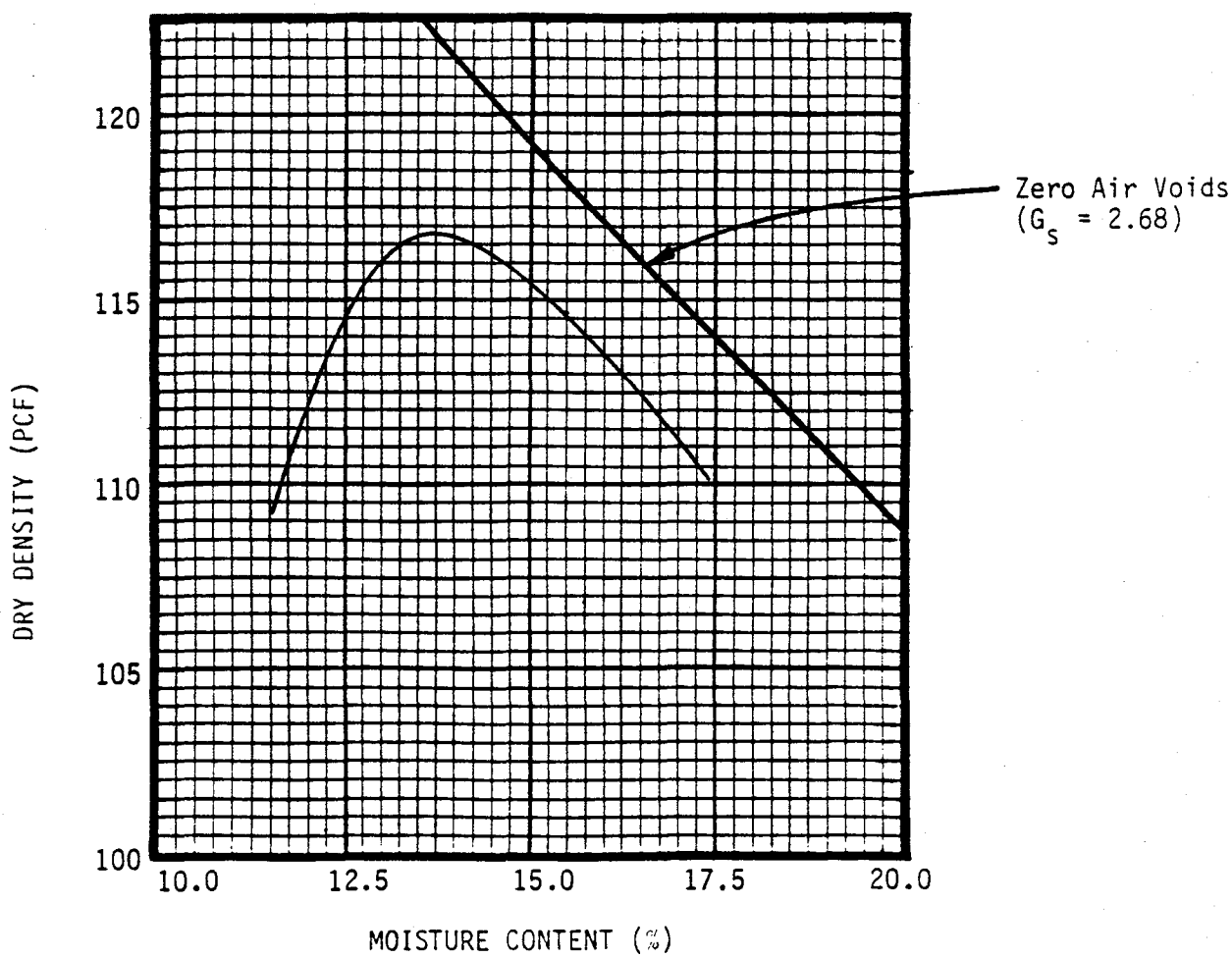
Material Sandy Clay/ Clayey Sand (CH/SC)

Sampled By TH/Thompson

TESTED: Moisture-Density Relationship Curve, ASTM D698, Method A

RESULTS:

Max. Dry Density (pcf) 117 Optimum Moisture Content (%) 14



Project No. 90-0863

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE:

Date: 1-22-91

Source: Noted Below
Type: Grab Samples
Material: Surface Soil
Sampled By: TH/Thompson

TESTED: R-Value

TEST RESULTS

Location	**R-Value
10; 0 - 7' & 7 - 14' *	34
12; 0 - 5' & 5 - 8' *	23
20; 0 - 8' & 8 - 16' *	35
22; 0 - 8' & 8 - 16' *	39

*Composite Samples
**Corrected to 300psi exudation pressure

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON REMOLDED EXPANSION TEST

SAMPLE: Date: 2/4/91

Source: Noted Below

Type: Bag Samples (*) and Driven Ring Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Percent expansion upon soaking of remolded sample compacted to approximately 95% of the maximum ASTM D698 dry density at approximately 2% less than optimum moisture content..

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
8; 4' - 5'	108	12.5	100	0.30
13; 0' - 8'*	112	11.0	100	1.40

Project No. 90-0863

Thomas-Hartig & Associates, Inc.

REPORT ON UNDISTURBED EXPANSION TEST

SAMPLE: Date: 2/4/91

Source: Noted Below
Type: Driven Ring Samples
Material: Subsurface Soil
Sampled By: TH/Thompson

TESTED: Percent expansion upon soaking of undisturbed sample.

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
16; 9' - 10'	106	17	100	0.62

Project No. 90-0863
Thomas-Hartig & Associates, Inc.

Huntingdon

Huntingdon Engineering & Environmental, Inc.
7351 West Ground Street
Chandler, AZ 85226
(602) 967-1199
Fax (602) 940-0352

Greiner, Inc.
7310 North 16th Street, Suite 160
Phoenix, Arizona 85020

13 May 1994

Attention: Vince Gibbons

Subject: Report on Geotechnical Engineering Services
Old Cross Cut Canal
Seepage Study
Indian School Road to McDowell Road
Phoenix, Arizona

Project No. 94-0198

Following your authorization, we have performed this evaluation of the groundwater along the proposed Old Cross Cut Canal Improvements. The proposed improvements consist of placing the existing open-channel conveyance into dual-barreled box culverts ranging from 10x10 feet to 18x10 feet per barrel. The box culvert will be covered with soil; typically 4 feet of cover over the top of the box will be provided. In our previous investigation for this project (Thomas Hartig and Associates Project No. 90-0863), groundwater was encountered in test borings along the proposed alignment at depths as shallow as 12 feet of the ground surface.

The scope of these additional services included:

1. Conducting field permeability tests using existing piezometer wells.
2. Review the results of periodic monitoring at the piezometer wells since their installation in 1991.
3. Evaluate the above information to estimate the quantity of seepage into construction excavations and to provide recommendations for side interceptors, weep holes, and drainage systems for construction and the final installation.

A member of the HTH group of companies

Field Tests

Test Borings 3, 8, 13, 16 and 21 were completed as piezometer wells during the geotechnical investigation in 1991 (see attached Site Plan). The piezometers are 2 inches in diameter, and are nominally 25 feet deep with well screen for the bottom 10 feet. Piezometer 3 has been dry since installation. Drawdown/recovery tests were performed at Piezometers 13, 16 and 21 on multiple occasions from 23 March through 13 April 1994. The tests were conducted using a variable speed submersible pump. The wells were pumped for periods of time ranging from about 10 minutes to about one hour. The flow rate was monitored during pumping, and ranged from about 1 gpm (Piezometer Wells 16 and 21) to about 2.5 gpm (Piezometer 3). The pump was then removed, and the residual drawdown measured and recorded to recovery. The total depth drawdown, maximum pumping rate, and duration of pumping which could be achieved were fairly limited due to the small well diameter.

The drawdown/recovery data were analyzed three different ways; all calculations are attached (Appendix A). Measured field permeabilities were around 0.01 cm/sec at Piezometer 13 and around 0.002 cm/sec at Piezometers 16 and 21. Maximum drawdowns created in the tests ranged from around 0.2 to 2 feet. The higher than expected permeabilities described above for the clayey soils along the alignment may results from the difficulty experienced in developing large drawdowns and pumping for extended time periods.

Review of Groundwater Records

The depth to water has been periodically measured in the piezometer wells since January 1991. Measurements were made approximately every month in 1991 and 1992, and sporadically since 1992. Copies of the water level data sheets are included in Appendix B. These data indicate that the groundwater level has fluctuated through about 5 to 8 feet of depth. The highest groundwater level identified at Piezometer 13, 1205.4 ft. MSL on 7 April 1993, is approximately the proposed level of the top of the box culvert at that location. This indicates that the fully buoyant condition can develop on at least portions of the proposed box culvert.

Results and Recommendations

We have evaluated the need for pressure relief by weepholes for the proposed box culvert. If cover of 4 feet over the top of the box cultert is maintained, we believe the factor of safety against buoyant uplift is at least 1.5. Therefore, we believe there is no

need for pressure relief using weepholes, drainage blankets, etc., as long as 4 feet of cover is maintained. If less cover is to be provided, then pressure relief using weepholes would be necessary to prevent buoyant uplift. We believe that the use of a geocomposite drainage material along the box culvert sides as described on the plans is adequate. Geocomposite drainage material should be supplied and placed in accordance with the ADOT Standard Specifications Sections 203-5.02 and 1014-6. Given the height of groundwater, we recommend decreasing the weephole spacing to 5 feet. Also, we understand that ADOT Urban Highways has had good experience with 4-inch diameter flap-type PVC weepholes sloping down 3 inches from back to face of lining, and would recommend that this detail be used.

We have also evaluated the potential for inflow into the excavation based on the field recovery/drawdown tests. We assumed a groundwater level approximately at the top of the box culvert, which corresponds to the maximum level observed during the monitoring period. We estimate inflow quantities of approximately 0.06 gpm/ft into the excavation. We believe this quantity of flow can be successfully handled using sump drainage areas and pumps. ?

The quantity of flow described above is conservative in that it is based upon unexpectedly high measured field permeabilities. These values may have been influenced by the gravel pack, the small size of the piezometer and the short duration of testing. A single confirmatory test should be conducted near Piezometer 13 in a larger diameter well (4-inches min.), using the existing Piezometer as an observation point. However, the quantity described above is not excessive, and may represent a conservative estimate which is acceptable for construction.

Please feel free to call this office if you have any questions or need further assistance.

Respectfully submitted,

HUNTINGDON ENGINEERING & ENVIRONMENTAL, INC.

Registered Professional Engineer (Civil) CERTIFICATE NO. 24033 KENNETH D. WALSH Date Signed 5-13-94 ARIZONA

By:
/cm
Copies to: Addressee (5)

Reviewed by:

Registered Professional Engineer (Civil) CERTIFICATE NO. 10371 KENNETH L. RICKER Date Signed 5-13-94 ARIZONA, U.S.A.

GREINER, INC.
7310 North 16th Street, Suite 160
Phoenix, Arizona 85020
(602) 275-5400

FAX COVER SHEET

☐ FAX: (602) 943-1891
☐ Marketing FAX: (602) 997-2037

DATE: 5/13/94 NUMBER OF PAGES: 4 (Including Cover Sheet)

TO: Don Rerick

FAX NO.: 506-4601

FROM: Vince Gibbons

REFERENCE: Old Cross Cut

MESSAGE: Don here is a copy of the Seepage study performed by Huntingdon (formerly Thomas Hartig). The entire report will be delivered to me next week and I will forward a copy or copies to you.

Vince

REPLY NEEDED:
☐ Yes
☒ FAX Only
☐ Copy & Distribute To: _____
☐ Copied & Distributed (Initials: _____)
☐ No
☐ Original to Follow by Mail

If you do not receive all pages clearly, please call (602) 275-5400 as soon as possible.

Greiner

MEMORANDUM

DATE: May 20. 1994

TO: Ken Walsh - Huntingdon

FROM: Vince Gibbons

SUBJECT: Old Cross Cut Canal

The following are questions or requested points of clarification pertaining to your Seepage Study prepared for the Old Cross Cut Canal project:

1. The estimated inflow quantity of 0.06 gpm/ft.
   - A. Is this per linear ft. of excavation (north/south) ?
   - b. How does the depth and/or width of excavation relate to this rate ?
   - c. Seepage versus SLOUGHING !
2. Weephole locations and details.
   - A. What will be the permanent drawdown effect on the ground water level (eg. possible consolidation settlement) if weep holes are at box invert, which is lower than the existing canal invert ?
   - B. On the other hand what is the anticipated effect on the ground water level due to the proposed box culvert blocking flow at an elevation where it is not impeded today ?
   - C. Can you provide us with a copy of the detail for the 4" PVC flap-type weepholes ?
3. Two foot of wash gravel bedding reccomended as a base under the box culvert.
   - A. Based on your estimated infiltration rate, do we still need a two foot thick base for the box culvert if the contractor employs a dewatering system ?
   - B. Is 100% compaction reasonable and attainable for this wash gravel base ?

Greiner

4. The additional single confirmatory test recommended in the report.

   A. How important do you feel this additional test is to the inflow quantities and weephole recommendations ?

   B. If this is needed, is it part of the scope or do you consider it as additional services ?

   C. If this test is a change of scope what is the addition cost to perform it ?

If you have any questions or need clarification in order to respond to these items please call. Your timely response will be greatly appreciated as we are dealing with a very demanding design schedule.

PPM - Rec'd 7/1/94

Huntingdon

Huntingdon Engineering & Environmental, Inc.

Greiner, Inc.
7310 North 16th Street, Suite 160
Phoenix, Arizona 85020

28 June 1994

Attention: Vince Gibbons

Subject: Report on Geotechnical Engineering Services
Old Cross Cut Canal
Seepage Study
Indian School Road to McDowell Road
Phoenix, Arizona

Project No. 94-0198
Supplement No. 1

This letter is intended to describe our observations of the test pit/pumping test conducted by the Flood Control District (FCD) of Maricopa County on June 13 and 14, 1994, and to address comments from Greiner, Inc., and FCD.

Test Pit/Pumping Test: A test pit was excavated on the west bank of the canal at approximately Earll Drive, in the vicinity of the highest observed groundwater levels along the alignment. The test pit was approximately aligned with the concrete bike path. Excavation was performed by FCD on 13 June 1994, and a pit roughly 20 feet long (north to south) at the surface was excavated using a backhoe with a 24-inch wide bucket. The County personnel reported that the soil profile consisted of approximately 7 feet of fill containing debris and trash, over about 9 to 10 feet of sandy clay, over a silty to clayey sand. The lower portion of the excavation, below the fill, had vertical walls to the total depth of the pit of about 20 feet; the upper portion through the fill stood at about 1H:1V to 3/4 H:1V. A hard caliche zone was encountered at 13 to 14 feet below grade and was approximately 1 to 1.5 feet thick. Groundwater was first noted during excavation at about 13 feet below grade.

The trench was allowed to stand open overnight, and the side slopes exhibited no signs of distress on 14 June 1994. FCD reported that the stabilized groundwater level after standing overnight was 11 feet - 9 inches below grade. This depth roughly

corresponded to the water level in the canal at the upstream end of the concrete transition section adjacent to the pit.

Several pumping tests were performed by FCD on 14 June 1994, and the results are summarized on the attached memo from Warren Rosebraugh with FCD. An estimated vertical profile of the trench based on field measurements is attached. Three pump tests were performed in which the pump rate was adjusted until a static water level was maintained, with each test at a different water level. The pump rate was roughly measured using a 5 gallon bucket and a stopwatch. We observed that it was difficult to throttle down the pump to a low level for these measurements such that a constant flow rate could be maintained, without allowing the water level to rise or fall.

Following the pump tests, a recovery test was performed. The trench was pumped down to a depth of about 18.6 feet below ground surface, and then the pump was shut off. The gradual rise of the water level in the trench was then measured. The attached graph shows the water level recovery, as well as the estimated inflow rate and the static pump test rates. The estimated inflow rate was computed from an approximation of the pit profile and the recovery measurements; computations are summarized on the attached pages. There were no observed signs of loss of trenchwall stability during any of the pump tests or the recovery test.

The estimated flow rate from the recovery test was fairly constant in the sandy clay layer, and increased significantly in the silty sand below the clay layer. The pump rates from the static level tests were roughly twice the flow rates interpreted from the recovery tests. The maximum flow rate from the static water level test was 32 gpm. Assuming only side inflow, this flow rate corresponds to approximately 1.3 gpm/ft, significantly higher than our estimate from the piezometer tests. The excavation for the box culvert will extend approximately twice as far below the groundwater table at this location as the test pit; we therefore estimate that the flow rate could roughly double in this portion of the box culvert excavation. The actual flow rate can be conservatively estimated using the higher static test pump rates to develop:

Approximate inflow = (0.22 gpm/ft/ft) (d) (P)

Where:

d = depth below groundwater (ft)

P = perimeter of trench at that depth (ft)

Comments from Greiner, Inc., 20 May 1994:

1. Comment refers to the estimated inflow quantify from the piezometer pump tests, which is superseded by the above estimate.

2. A. The comment suggests that consolidation settlement may occur if weepholes are lower than existing channel bottom. From the compression tests performed for the original study, the soils within the proposed excavation are heavily over-consolidated (OCR > 2). As such, we estimate maximum settlements from decreased groundwater elevations near the box to be less than 0.1 inch.

   B. The comment suggests that the box culvert will impede or block groundwater flows. Given the overall roughly southerly gradient, we do not expect the culvert will serve as a block.

   C. The 4-inch PVC flap-type weepholes described in the ADOT Urban Highways Channel Lining Guidelines were apparently included as a "wish list" suggestion. We could identify no such detail.

3. A. The water content of the natural site soils along the proposed invert is likely to be very high, even with de-watering schemes in place. Because of the plasticity of site soils, we expect that workability and mobility problems will result. Therefore, we believe the gravel base will be needed to provide a working platform.

B. We believe 100% compaction is attainable for the base.

4. The comment refers to our recommendation for confirmatory testing; the FCD test pit constitutes such confirmation.

Comments from Greiner, Inc., 27 May 1994:

1. A. The precipitation events referred to in the comment refer to particular storm intensities and short term events. The groundwater level would not be expected to be influenced noticeably by an individual storm event. As such, there is no correspondence between the design groundwater level and the design storm events.

   B. There will be friction between the sides of the box and the backfill which will resist uplift. This friction is conservatively ignored in the buoyancy analysis.

2. The recommendations in the previous letter were based on water table conditions.

3. The trench wall stability was maintained in a vertical condition for the 24-36 hours the FCD test pit was open, even with de-watering. The actual excavation walls will be less steep and we expect them to be stable as well.

4. The contractor will evaluate the cost of well points versus sump pumps. The impact of registration with ADWR for de-watering compared to the cost of pumps and electricity will have to be evaluated by the contractor as well.

Comments from FCD, 6 June 1994:

1./2. Adding the extended slabs would have an impact on buoyancy. However, we believe the calculations attached to our previous letter (dated 13 May 1994) demonstrate that a factor of safety of at least 1.5 exists without them, assuming about 4 feet of cover. Furthermore, the question of the actual cover was addressed in a tabulation from the project plans. This tabulation shows that the cover is at least 4 feet everywhere but the southern end. However, the piezometer at the southern end has been dry throughout the monitoring period.

The FCD suggests that the buoyant unit weight of the cover should be used instead of the total unit weight. The calculation is based on the assumption that groundwater is at the highest level observed in the monitoring period -- roughly the top of the box. We would only use the submerged unit weight for the cover if groundwater were at the ground surface. Groundwater may locally mound under the surface channel during the 100 year flow, but then again the box culvert won't be empty then, either. We understand from SRP well data in the area that the observed maximum levels are consistent with maximum levels stretching back to 1960; we believe the design groundwater level is conservative. This is especially true given that: a) the groundwater level at the box top really only occurs in a fairly small section of the box culvert length; b) the fill density of 110 pcf assumed is quite conservative; and c) we have ignored side friction.

3. As we have discussed, we believe that the box culvert will be safe against uplift without weepholes. If weepholes are considered desirable as an extra factor of safety or as a means of maintaining existing groundwater drainage conditions, then placing them at the channel invert elevation makes sense. However, we do no expect a significant impact

on the surrounding ground if the weepholes are located at the one foot level for easier detailing and constructability (see response to item 2 in Greiner 20 May comments).

We also could not find a near-horizontal flap-type weephole. If a standard weephole is used then outward flow from the box during flows should be expected. As there is obviously outflow from the current channel in high flows, we see no reason to expect problems from these outward flows.

4. In order for piping to occur into the wash gravel, a localized critical gradient has to occur. There doesn't appear to be a critical location for this to occur in this case. We certainly have no objection to cutoff walls, but we see no compelling reason for them either.

This supplement shall be attached to our previous letter dated 13 May 1994 and shall become a part thereof. Please call if we can be of further service.

Respectively submitted,

HUNTINGDON ENGINEERING & ENVIRONMENTAL, INC.

By: Registered Professional Engineer (Civil) CERTIFICATE NO. 24033 KENNETH D. WALSH Date Signed 6/29/94 ARIZONA, U.S.A.

Reviewed By: Registered Professional Engineer (Civil) CERTIFICATE NO. 10971 KENNETH L. RICKER Date Signed 6/29/94 ARIZONA, U.S.A.

/trm

Copies to: Addressee (5)

OLD CROSS CUT CANAL
TEST PIT - WEST BANK, NORTH OF EARLL DR.

6/13/94 - Excavation

Bucket width - 24"
Caliche encountered approx. 13' to 14' below grade.
Approx. depth of excavation - 20'

6/14, 8 am - Water level in excavation before pumping was approx. 11' 9"

6/14 - Pump tests

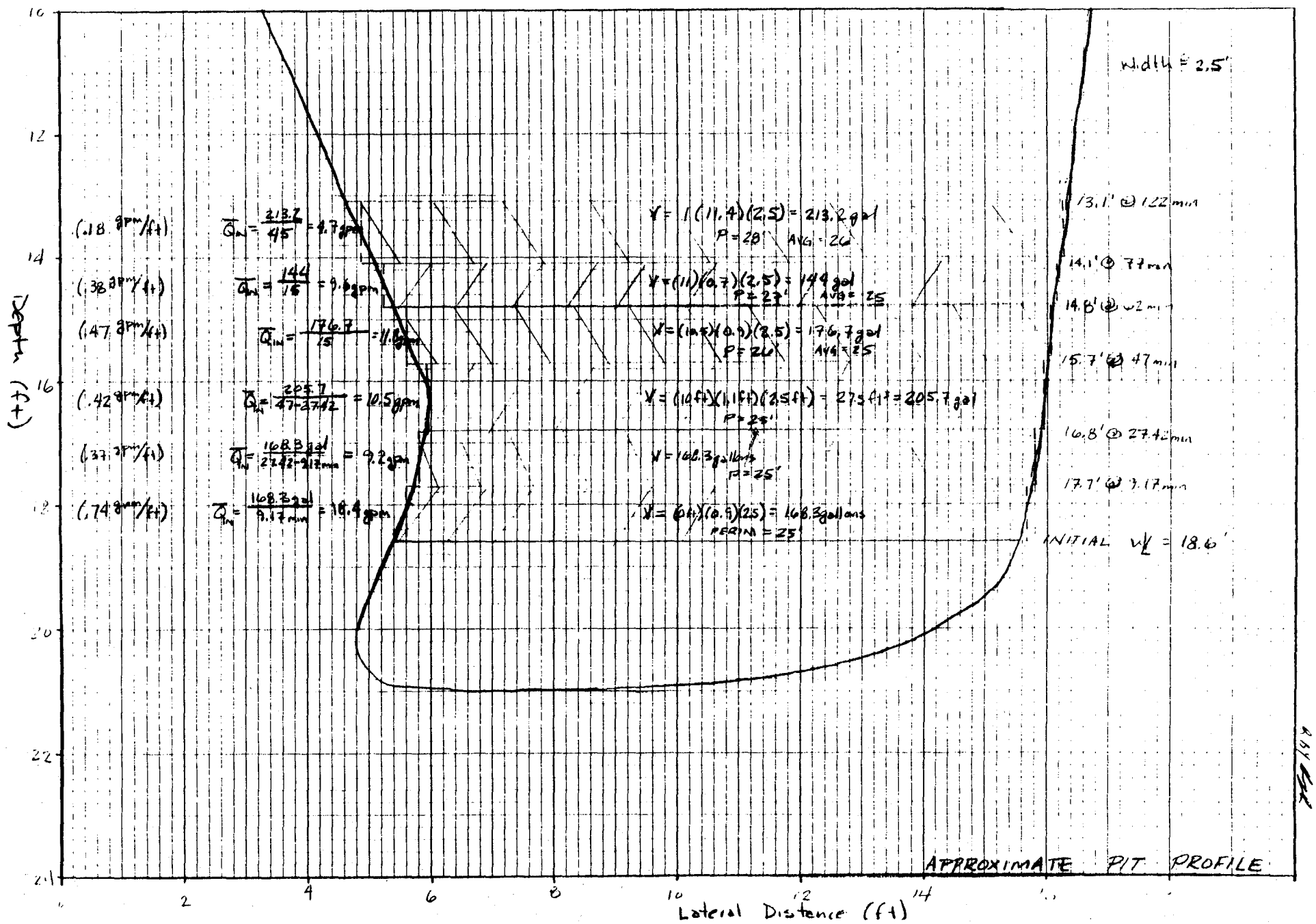
Approx. static water level (ft.)	Times to fill 5 gal. bucket (sec.)			Average time (sec.)	Calculated flow rate (gpm)
15.5	13	13	12	12.67	24
16.0	18	14	16	16.00	19
17.5	9	10	9	9.33	32

6/14 - Recovery times

Time	Water depth (ft.)
11:43:00	18.6
45:20	18.3
48:15	18.1
52:10	17.7
55:30	17.5
12:03:50	17.0
10:25	16.8
30:00	15.7
45:00	14.8
13:00:00	14.1
15:00	13.7
30:00	13.4
45:00	13.1

NO. 340R-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

DIETZGEN CORPORATION
MADE IN U.S.A.



RSY

CW 94-108

DIETZGEN GRAPH PAPER
10 X 10 PER INCH
DIETZGEN CORPORATION
MADE IN U.S.A.

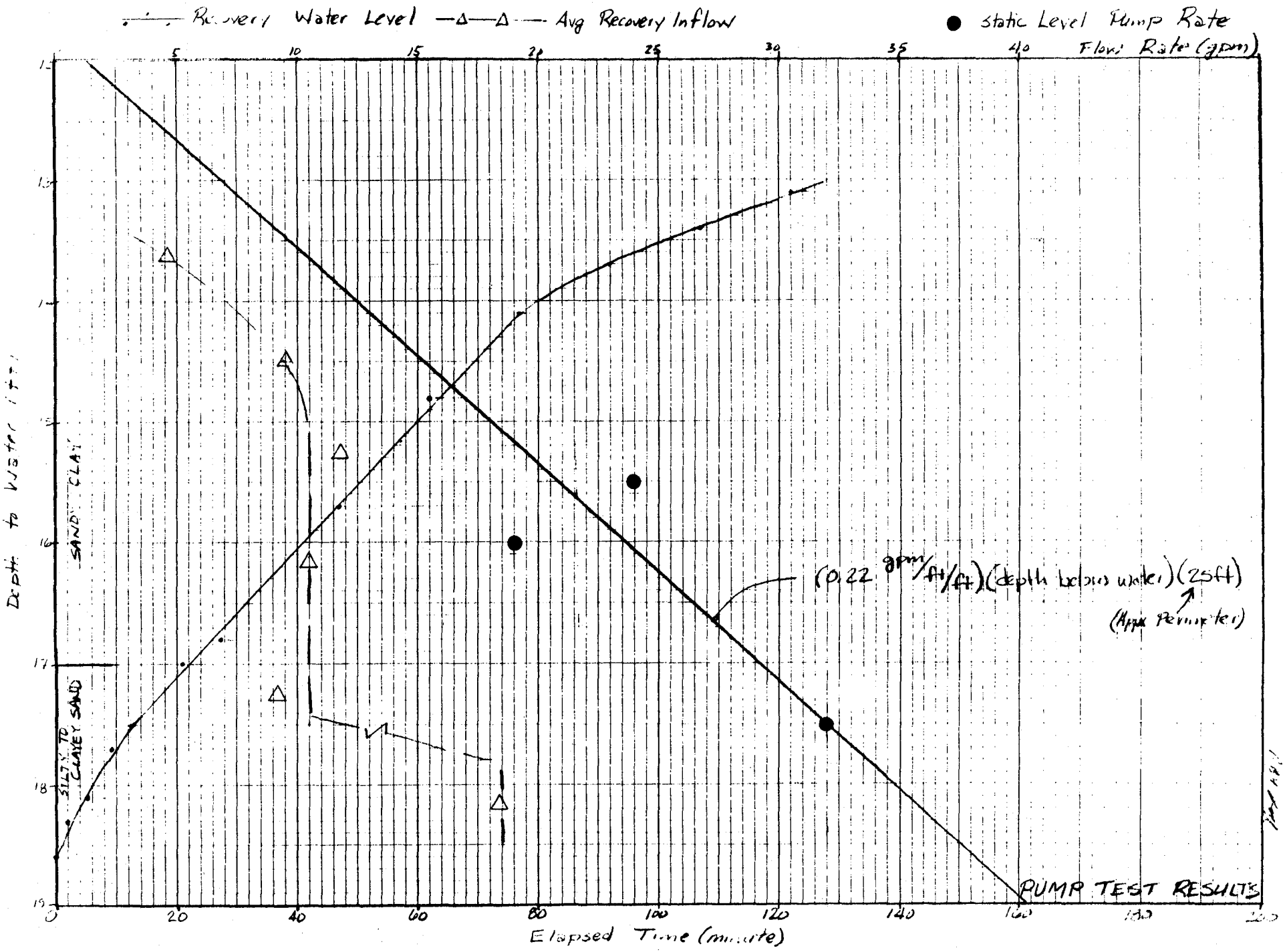


Fig 4-3

KW 94-198
RBV [illegible]

CONSOLIDATION SETTLEMENT

WORST CASE : #13 Minimum recorded GWT ≅ 1202 (July 1991)

Channel invert ~ 1195

Ground surface ~ 1214

Assuming 1202 lowest GWT ever :

1214 GROUND

$\gamma_{TOTAL} = 125$ pcf

1202

Average element @ 1198.5

1195

For Average element $\sigma_i' = 125(1214 - 1198.5) - 62.4(1202 - 1198.5)$

$= 1937.5 - 218.4 = 1719.1$ psf

$\sigma_f' = 125(1214 - 1198.5) = 1937.5$ psf

From consolidation tests, high degree of overconsolidation (2-6 at least), ∴ use C_R from loading curve

$C_R = 0.8\%/\text{cycle}$

use dummy initial point $p_o = 1000$ psf

$$\varepsilon = \varepsilon_o + C_R \log\left(\frac{p'}{p_o}\right)$$

$$\Delta\varepsilon = \left(\varepsilon_o + C_R \log\left(\frac{\sigma_f'}{p_o}\right)\right) - \left(\varepsilon_o + C_R \log\left(\frac{\sigma_i'}{p_o}\right)\right)$$

$$= C_R\left[\log\left(\frac{\sigma_f'}{p_o}\right) - \log\left(\frac{\sigma_i'}{p_o}\right)\right]$$

$$= 0.8\%\left[\log(1.94) - \log(1.72)\right] = .042\%$$

Ground settlement $= \left(\frac{.042}{100}\right)(7\text{ft})\left(12\frac{\text{in}}{\text{ft}}\right) = .04$ in ← NEGLIGIBLE