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# ACCESS BRIDGES ON THE ACDC

Squaw Peak Water Treatment Plant
Phoenix, Arizona

Registered Professional Engineer (Civil)
CERTIFICATE NO. 12575
STEVEN A. HAIRE
Date Signed 11-14-89
ARIZONA, U.S.A.

Prepared for

John Carollo Engineers
3877 North 7th Street, Suite 400
Phoenix, Arizona

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

A118.945

# THOMAS-HARTIG & ASSOCIATES, INC.

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16 November 1989

Attention: George Shirley, P.E., Partner

Project: Access Bridges and Pipeline Crossings over ACDC Squaw Peak Water Treatment Plant Phoenix, Arizona

Project No. 89-0872

In accordance with your request, we have performed geotechnical services at the subject site to determine subsurface conditions and to develop design recommendations for proposed bridges and pipeline crossings over the ACDC at the Squaw Peak Water Treatment Plant, which is located north of the Arizona Canal on the west side of 24th Street in Phoenix, Arizona. The services performed provide an evaluation at selected locations of the soil and rock materials throughout the zone of significant foundation influence. Our services have not included determination of geologic conditions or evaluation of potential geologic hazards such as seismic activity or faulting.

**SITE AND PROJECT DESCRIPTION**

Two single-lane bridges, the East Access Bridge and the West Access Bridge, will be constructed to provide access over the future ACDC channel at the locations shown on the site plans in Appendix A. Also, several existing pipelines, ranging from 36-inch to 66-inch diameter, will be re-aligned and supported where they will cross the future channel. We understand that the bridges and pipe supports will be constructed prior to excavating the underlying ACDC. Maximum bridge abutment loads will be about 15 kips per foot. Maximum vertical pipe support load will be about 70 kips, and the maximum horizontal thrust-block load at pipe bends near the supports will be about 200 kips.

The site is relatively flat, sloping gently down to the south. Vegatative ground cover consists of scattared areas with shrubs and small trees.

## EXPLORATION

Field exploration included drilling ten (10) test borings at the locations shown on the attached site plans. During the test drilling, the materials were visually classified and representative samples were obtained at selected depths. The results of the test drilling are presented in Appendix A.

As shown on the site plans in Appendix A, some previous field exploration has been performed by this firm at the treatment facility site. This previous exploration included drilling several borings and conducting seismic refraction surveys to provide information toward design of new facilities. The results of this previous investigation were summarized in our report to John Corollo Engineers, Project No. 89-0329, dated 25 May 1989. Selected boring logs and seismic refraction survey data from this previous report are presented in Appendix C. Information gathered in the previous investigation was used to help formulate recommendations presented in this report.

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

| Test | Sample(s) | Purpose |
|---|---|---|
| Direct Shear | Compacted Soil and Rock fragments (3) | Strength Characteristics |
| Sieve Analysis and Atterberg Limits | Surface Soil (3) | Classification and correlation to engineering properties |
| Soluble Salts and Sulfates | Soil and Rock Fragments (3) | Corrosion Potential |

The test results are presented in Appendix B.

## SOIL AND ROCK CONDITIONS

As illustrated on the graphical boring logs presented in Appendix A, the soil and rock stratigraphy is relatively uniform at the boring locations. In general, about 1 to 7 feet of soil cover overlies a deposit of cemented breccia fanglomerate. The surface soils encountered consisted primarily of dense clayey sands and gravels. The breccia fangolmerate consisted of angular gravel and cobble-sized rock clasts cemented in a matrix of caliche-cemented angular sand, with the degree of cementation generally varying from moderate to heavy. Zones of concrete-like consistency, with refusal to auger penetration, were encountered.

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

**DISCUSSION AND RECOMMENDATIONS**

Foundations: Bridge abutment footings and pipe support footings should be deepened as necessary to bear into hard breccia fanglomerate encountered typically below depths of about 1 to 7 feet at the test boring locations. The footings should also be deepened as necessary so that the edge of footing will be at least a lateral distance of 3 feet from the face of the future ACDC excavation. Figure 1 on the following page illustrates the profile for the East Bridge. The West Bridge will be similar. As shown on the drawing, the assumed slope of the ACDC excavation in the beccia fanglomerate will be no flatter than about 2V:1H. If the ACDC is excavated steeper than 2V:1H, then there will be no adverse effect on the bridge foundations, because the distance between the footing and the slope face will increase (assuming the bridge length and the width of the bottom of the ACDC excavation are fixed). Footings on the hard breccia fanglomerate will support the anticipated bridge and pipe support loads with negligable settlements. Foundation bearing design recommendations are presented below.

| Footing Depth | Bearing Material | Allowable Foundation Bearing Pressure |
|---|---|---|
| * | Cemented Breccia fanglomerate | 8000 psf |

* Footings should be deepened as necessary to bear at least 1 foot into cemented breccia fanglomerate and should also be deepened to maintain a lateral distance of at least 3 feet between the edge of the footing and the future ACDC excavation.

The recommended foundation bearing pressure should be considered as an allowable maximum for dead plus design live loads and may be increased by one-third when considering total loads including transient loads such as seismic forces. The weight of the foundation below grade way be neglected. Two (2.0) feet is the minimum recommended footing width.

Thrust Block Design: Thrust blocks should be constructed to bear directly upon undisturbed soil or breccia. If a thrust block spans across a trench backfill, no resistance should be assumed for the portion of the thrust block bearing on backfill. The lateral passive pressures presented in the "Lateral Design Parameters" section of this report may be used as allowable lateral thrust block pressures. For thrust block loads toward the open ACDC excavation, the lateral

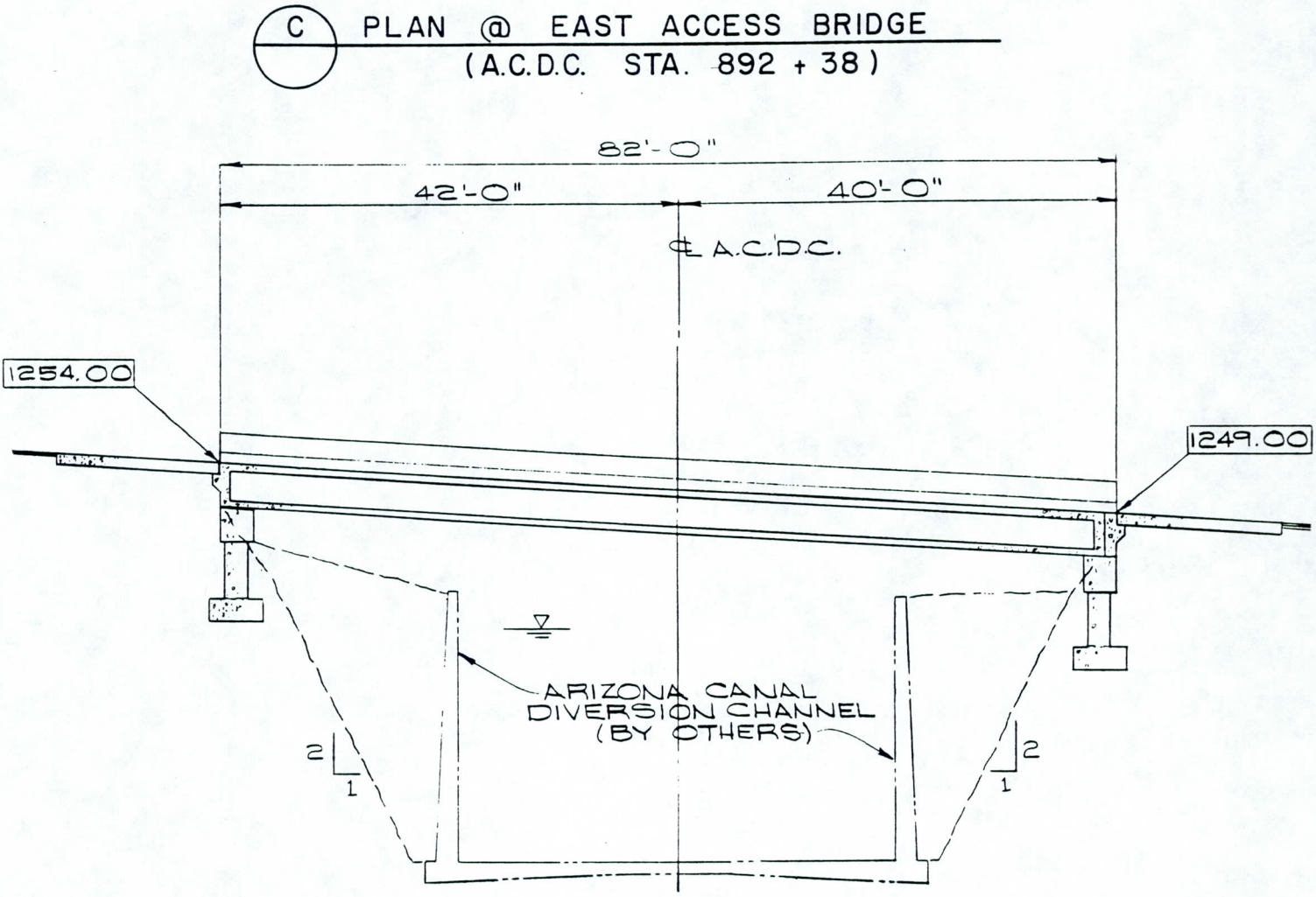


Figure 1. East Access Bridge Profile
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Project No. 89-0872

distance between the thrust block and the slope face should be at least 2.5 times the height of the thrust block.

Lateral Design Parameters: The following tabulation presents recommendations for lateral stability analyses:

[1]Foundation Toe Pressures----------1.33 X allowable
[2]Lateral Backfill Pressures:
Unrestrained walls----------35 psf/ft.
Restrained walls----------55 psf/ft.
[3]Lateral Passive Pressures:
Continuous walls/footings:
embedded in soil----------250 psf/ft. of depth
embedded in fanglomerate or bedrock----500 psf/ft., but not to exceed 10,000 psf
Spread columns/footings:
embedded in soil----------350 psf/ft.
embedded in fanglomerate or bedrock----700 psf/ft. but not to exceed 10,000 psf
Coefficient of Base Friction:
Independent of passive resistance-------0.40
In conjunction with passive resistance--0.30

[1]Increase in allowable foundation bearing pressure (previously tabulated) for foundation toe pressurs due to transient eccentric or lateral loading. The entire footing bearing surface should remain in compression.

2Equivalent fluid pressures for vertical walls and horizontal backfill surfaces. Pressures do not include temporary forces imposed during compaction of the backfill, swelling pressures developed by over-compacted clayey backfill, hydrostatic pressures from inundation of backfill, or surcharge loads.

[3]For footings near the ACDC excavation slope, passive resistance against movement toward the excavation should be neglected.

Compaction of the backfill soils against embedded footings or walls designed to provide passive resistance should be accomplished to a minimum 95 percent of the maximum ASTM D698 density to develop this resistence with low strains.

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 the 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.

Cemented soils and/or breccia are anticipated at depths below about 1 to 7 feet over most of the site. Excavations into these materials may be difficult and require the assistance of specialized equipment and/or rock removal techniques, especially if thick, continuous heavily cemented layers are present. Excavations should be braced or sloped as required to provide personnel safety and satisfy local safety code regulations.

Blasting: Blasting will probably be required to efficiently complete the excavation for the ACDC. Any blasting should be carefully controlled so that no disturbance or fracturing of the breccia fanglomerate will occur within 3 feet of the bridge foundations. Due to the approximity of the bridge and other treatment plant structures, pre-blast surveys of the structures and blast monitoring should be performed.

Deep Backfills: Backfill intended for structural support or intended to provided passive lateral resistance should be compacted to density criteria presented in the "Site Grading" section of this report. Compaction of all structural fill should be done by mechanical methods. If backfills are not compacted as recommended, subsidence may result in areas adjoining backfilled structures or over utilities. Even for well compacted granular backfills, long term settlements may approach 1/4 to 1/2 percent of the fill height, or more if water is introduced into the fill after construction. Even properly compacted deep backfills may tend to settle differentially relative to structural walls and should not be used for support of adjoining facilities or utilities prone to damage from differential settlements.

Corrosion: Corrosion is most likely to occur in fills and natural soils with high moisture contents and low electrical resistivity. Laboratory tests were performed on selected samples to determine soluble salts and soluble sulfates content. Soluble sulfates content of the tested soils ranged from 0.012 percent to 0.021 percent. Based on these values, the corrosion potential to concrete is low. Therefore, concrete in contact with soils or rock should use Type II cement. A low to moderate potential for corrosion of buried metal conduits is indicated in areas where soil moisture contents are high. Thus, special protection may be necessary were dissimilar metals are placed in close proximity or are joined.

Fill Materials: All fill materials should be soils free of vegetation, debris,

organic contaminants, or fragments larger than 6* inches in size. For structural fills, imported fill soils exhibiting low expansion potentials or granular site soils should be used. Care should be taken to remove or break down boulder-size fragments and blend with finer soil as necessary to minimize voids between large nested fragments.

Any imported structural fill or backfill soils for use beneath footings or concrete slabs should conform to the following specification requirements:

Maximum particle size----------6 inches*
Maximum percent expansion----------1.5 **

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

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

Site Grading: The following recommendations are presented for site grading for the inlet structures area. All phases of the earthwork should be performed under the observation and testing directed by the geotechnical engineer.

1. After mass excavation, observe the construction site and remove any remaining unstable (loose, disturbed, wet, etc.) soils encountered.

2. Widen any depressions as necessary to accomodate compaction equipment and provide a level base for placing fill.

3. Clean the exposed surface of all loose or disturbed soils.

4. Place fill materials required to elevate site areas to specified grade. Fill materials should be placed and compacted in horizontal lifts of thicknesses compatible with the compaction equipment being used.

5. Compaction of fills should be accomplished to the following density criteria.

| Material | Percent Compaction (ASTM D698) |
|---|---|
| *Fill and structural backfill: | |
| Structural fills and backfills shallower than 5 feet below finished grade | 95 min. |
| Structural fills and deep backfills deeper than 5 feet below finished grade | 100 min. |
| **Miscellaneous backfill | 90 min. |

*Deep structural wall backfills should not be used for support of facilities which are susceptible to damage from differential settlements of the fill section relative to walls. Bridge foundations should bear on undisturbed materials rather than on structural fill.

**Utility trench and exterior wall backfill not intended to provide passive resistance for pipeline, or for foundation or pavement support.

Compaction of on-site soils or imported fill materials with low expansive potentials should be accomplished at a moisture content of optimum -3 to optimum +3 percent.

Please call if you have any questions or if we may be of further service.

Respectfully submitted,

**THOMAS-HARTIG & ASSOCIATES, INC.**

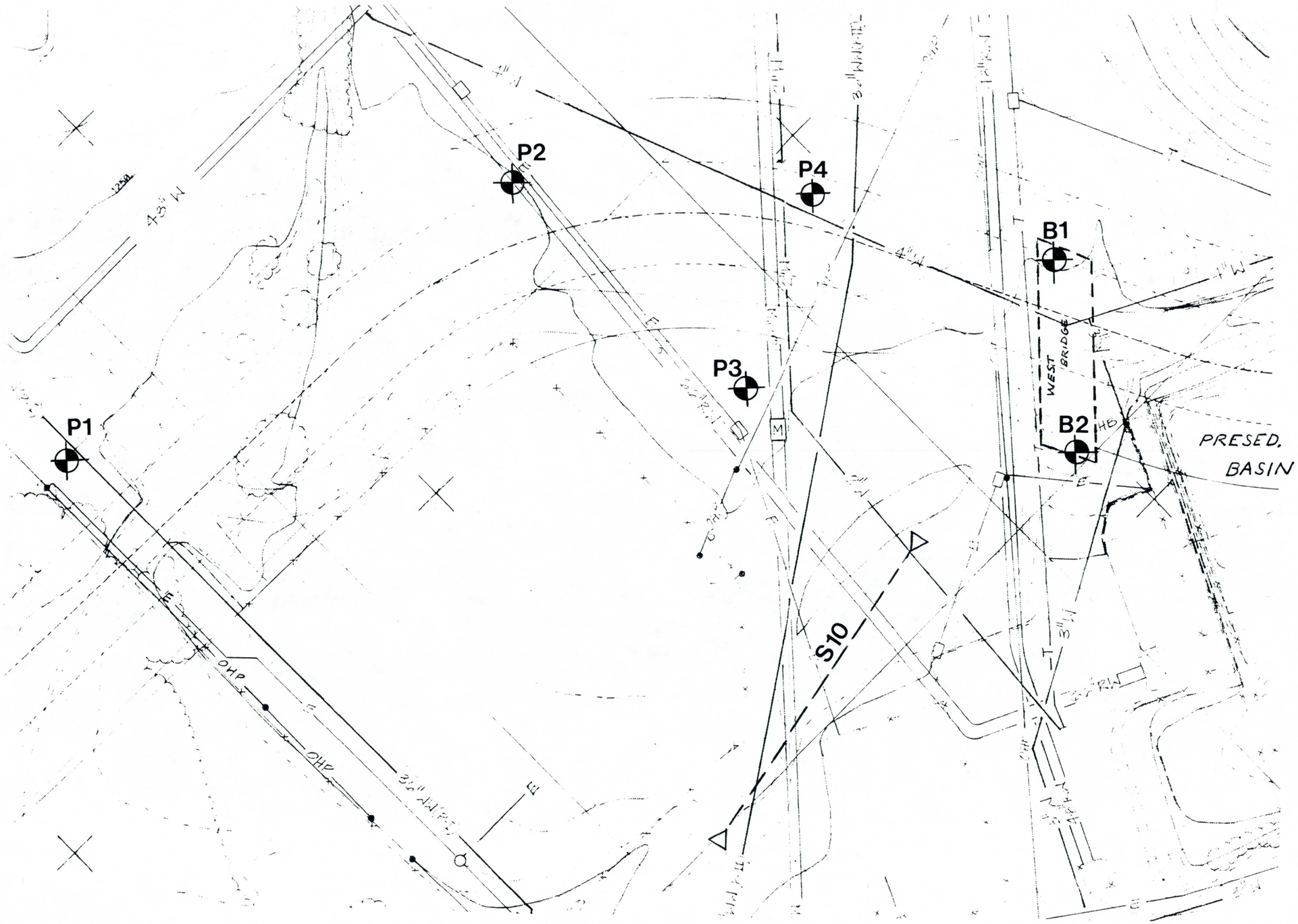
By: _____ Reviewed by: _____

Steven A. Haire, P.E. Kenneth L. Ricker, P.E.

/go

Copies to: Addressee (5)

# APPENDIX A
# FIELD RESULTS




SCALE
1" = 40'

LEGEND

Test Boring Location

Location of seismic refraction survey from previous investigation*

Location of test boring from previous investigation *

*Previous investigation for John Carollo Engineers, THA project No. 89-0326, dated 25 May 1989. Logs and refraction results are presented in Appendix C.

Boring Location Plan (sheet 1 of 2)
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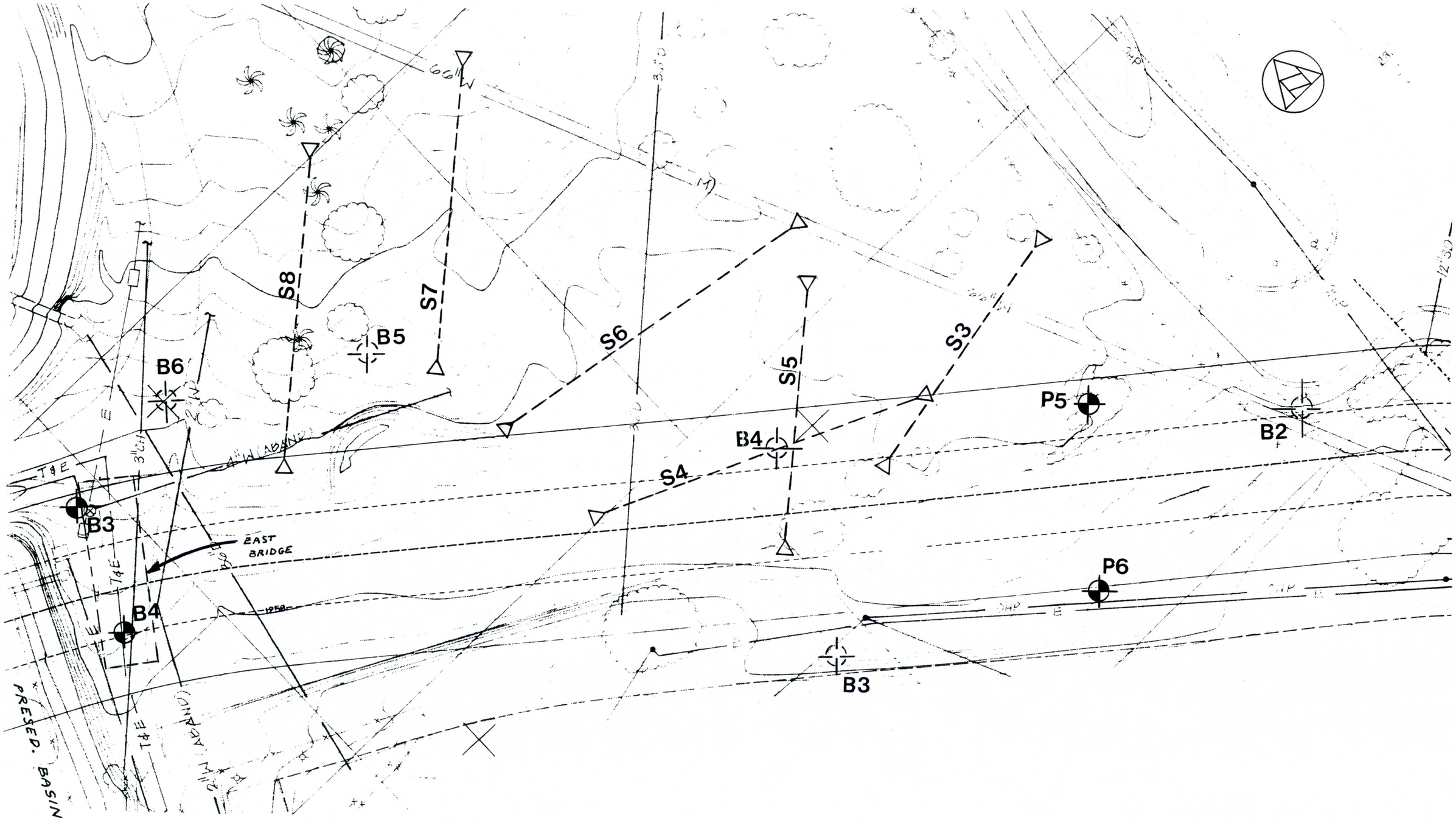
Project No. 89-0872




Boring Location Plan (sheet 2 of 2)
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Project No. 89-0872

# 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<br>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<br>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 | |

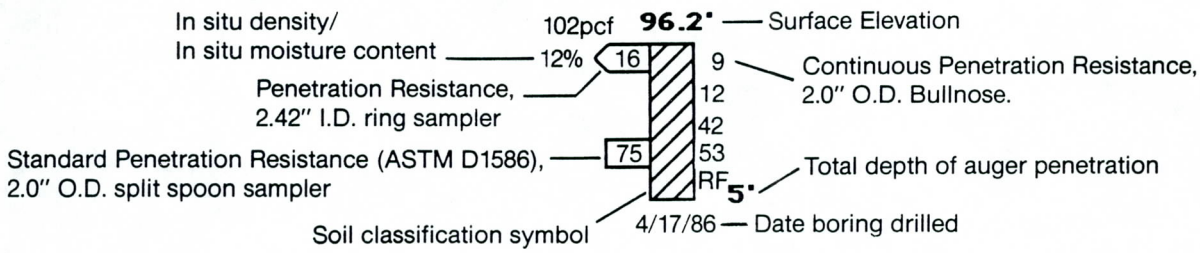
### FINE-GRAINED SOIL

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<br>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<br>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 | | | | CLEAR SQUARE SIEVE OPENINGS | | | |
| 200 | 40 | 10 | 4 | 3/4" | | 3" | 12" |
| SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY | SAND | | | GRAVEL | | COBBLES | BOULDERS |
| | FINE | MEDIUM | COARSE | FINE | COARSE | | |

**MOISTURE CONDITION (INCREASING MOISTURE →)**

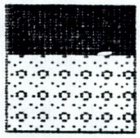
| DRY | SLIGHTLY DAMP | DAMP (Plastic Limit) | MOIST | VERY MOIST | WET (SATURATED) (Liquid Limit) |
|---|---|---|---|---|---|

| 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 | | |

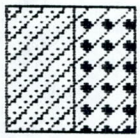
*Number of blows of 140 lb. hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

Project No. 89-0872

# LEGEND OF SOIL TYPES



ASPHALTIC CONCRETE OVER AGGREGATE BASE



CLAYEY SAND AND GRAVEL (SC&GC); brown to light brown; stratified; fine to coarse sand size; dense; generally low plasticity; angular; scattered cobble and boulder-sized fragments; intermittent light to moderate cementation.

BRECCIA FANGLOMERATE; white to grey; stratified; angular gravel-and cobble-size rock clasts cemented in a matrix of caliche-cemented angular sand; degree of cementation generally varies from moderate to heavy; with zones of concrete-like consistency and zones of light cementation.

NOTE: Boring elevations are approximate, based on interpolation between contours shown on site survey.

Project No. 89-0872
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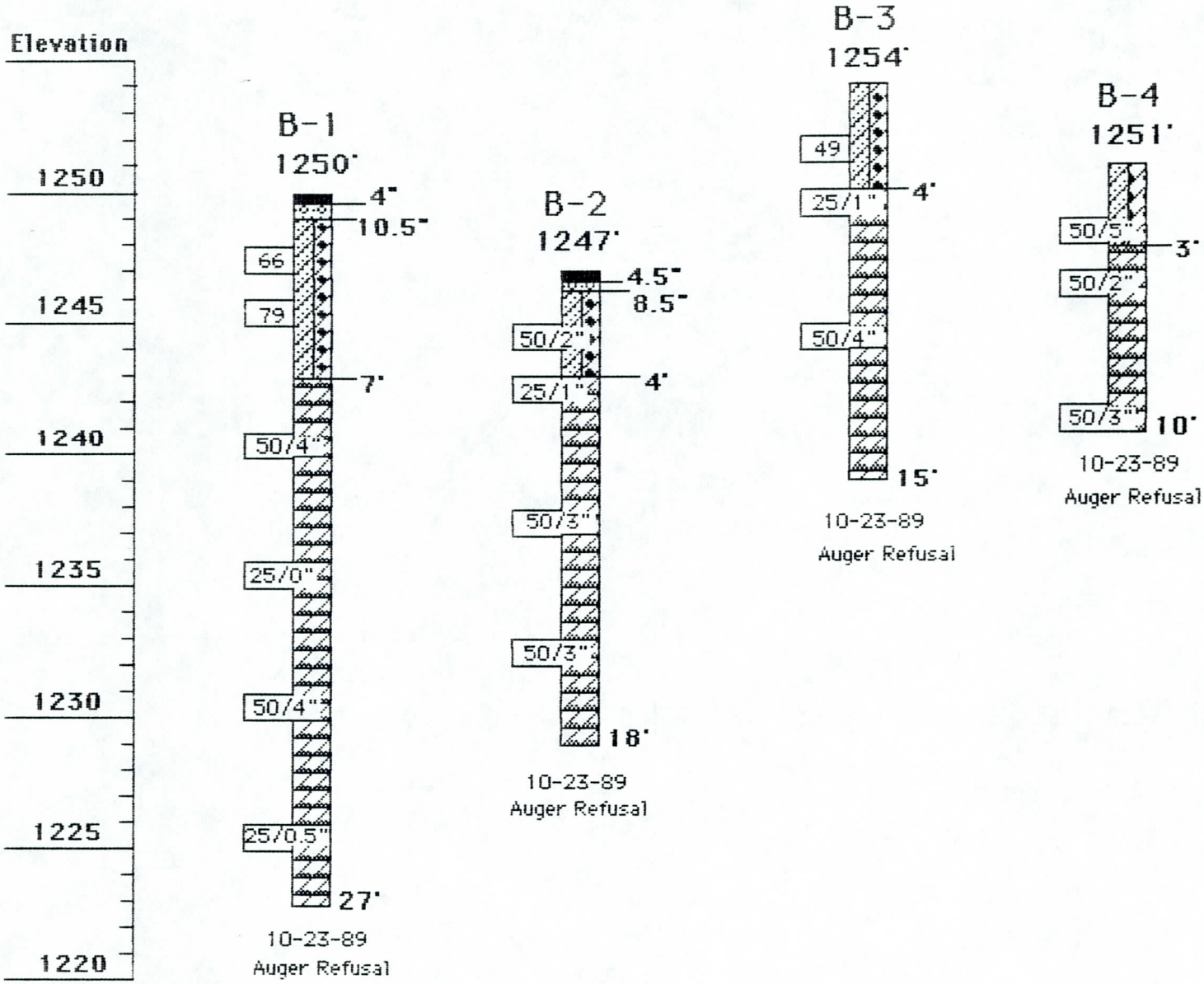
## GRAPHICAL BORING LOGS




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

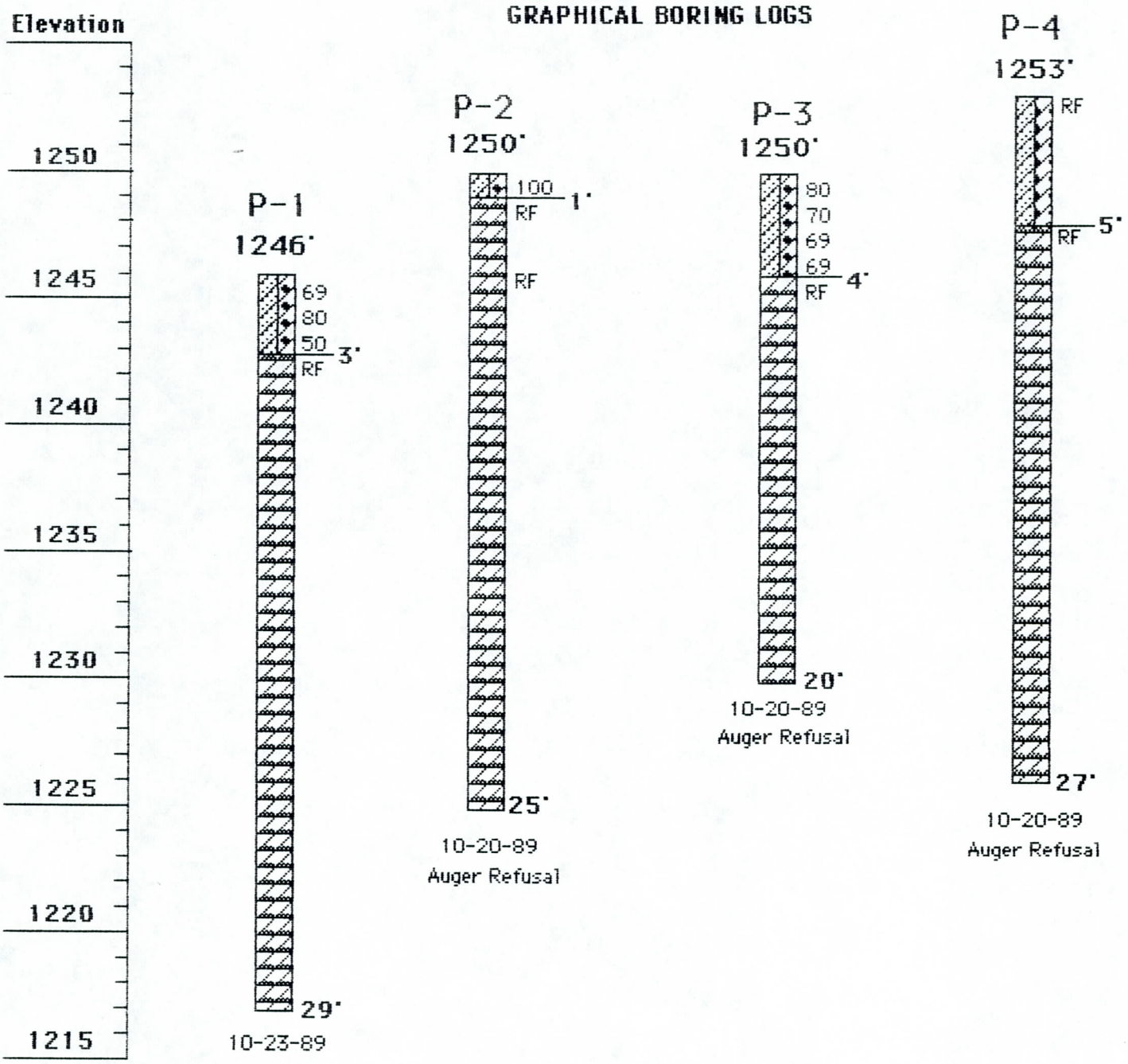
**All borings drilled with 7" diameter hollow stem auger unless otherwise noted.**

**Project No. 89-0872**

**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




RF = Refusal to penetration (i.e., more than 100 blows/foot)

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

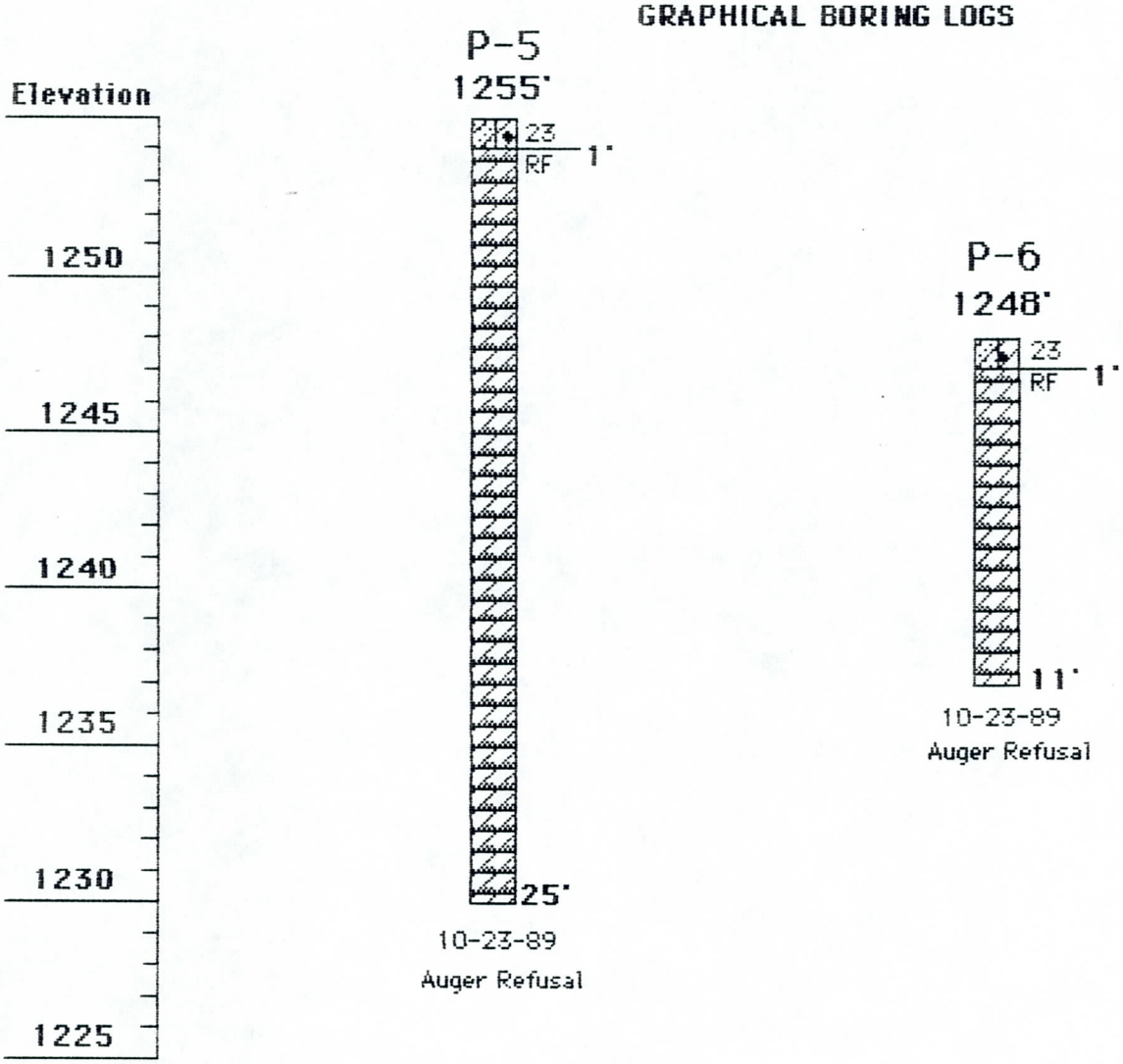
**All borings drilled with 4" diameter continuous flight auger unless otherwise noted.**

Project No. 89-0872

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




RF = Refusal to penetration (i.e., more than 100 blows/foot)

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

**All borings drilled with 4" diameter continuous flight auger unless otherwise noted.**

**Project No. 89-0872**

**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.

# APPENDIX B

# LABORATORY RESULTS

# REPORT ON LABORATORY TESTS

SAMPLE: Date 11-10-89

Source Composite of B1 & B2; 0 - 2'

Type Bulk sample, remolded to 126 pcf dry density

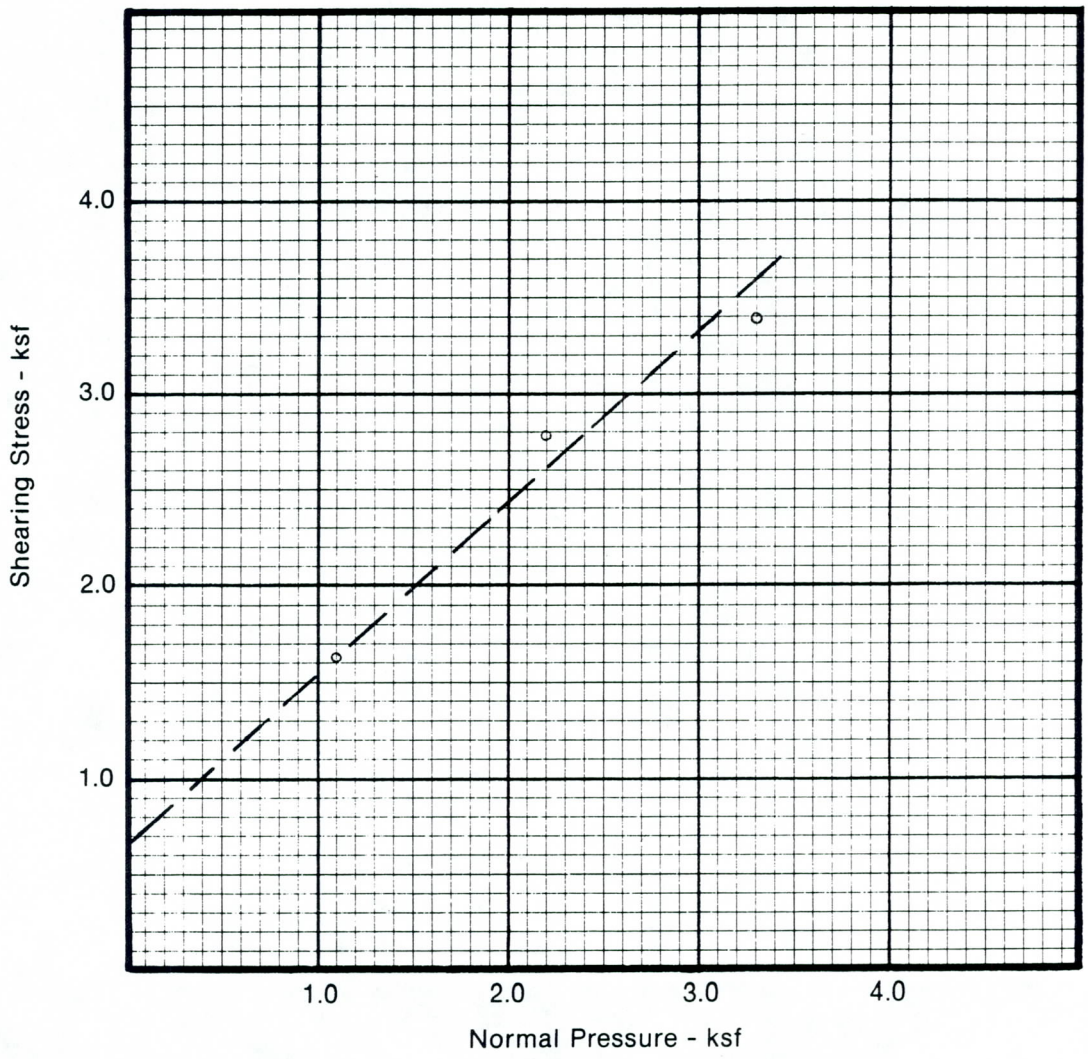
Material Clayey Sandy Gravel (GC)

Sampled By TH/Thompson

TESTED: Direct Shear, portion of sample passing a #4 sieve was remolded to about 95% of ASTM D698 maximum density, submerged, consolidated, then sheared at a displacement rate of 0.07 in./min.

RESULTS:

Friction Angle (Ø) = 42° Cohesion (c) = 0.65 ksf




Project No. 89-0872



TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

# REPORT ON LABORATORY TESTS

SAMPLE: Date 11-10-89

Source Composite of B3 & B4; 0 - 2'

Type Bulk sample, remolded to 120 psf dry density

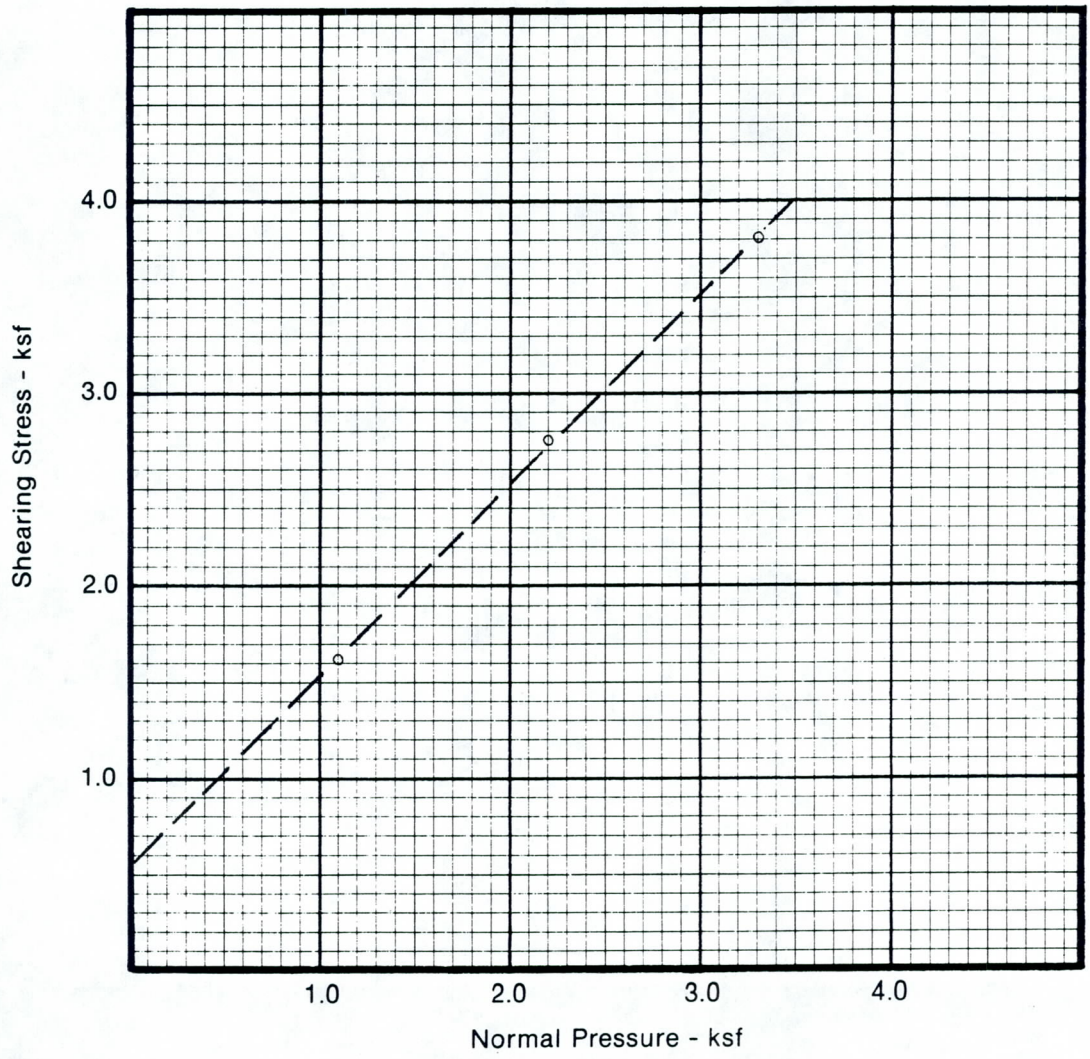
Material Silty Clayey Sandy Gravel (GC-GM)

Sampled By TH/Thompson

TESTED: Direct Shear, portion of sample passing a #4 sieve was remolded to about 95% of ASTM D698 maximum density, submerged, consolidated, then sheared at a displacement rate of 0.07 in./min.

RESULTS:

Friction Angle (Ø) = 44°  Cohesion (c) = 0.55 ksf




Project No. 89-0872

# REPORT ON LABORATORY TESTS

SAMPLE: Date 11-14-89

Source P5; 0 - 2'

Type Bulk sample, remolded to 120 pcf dry density

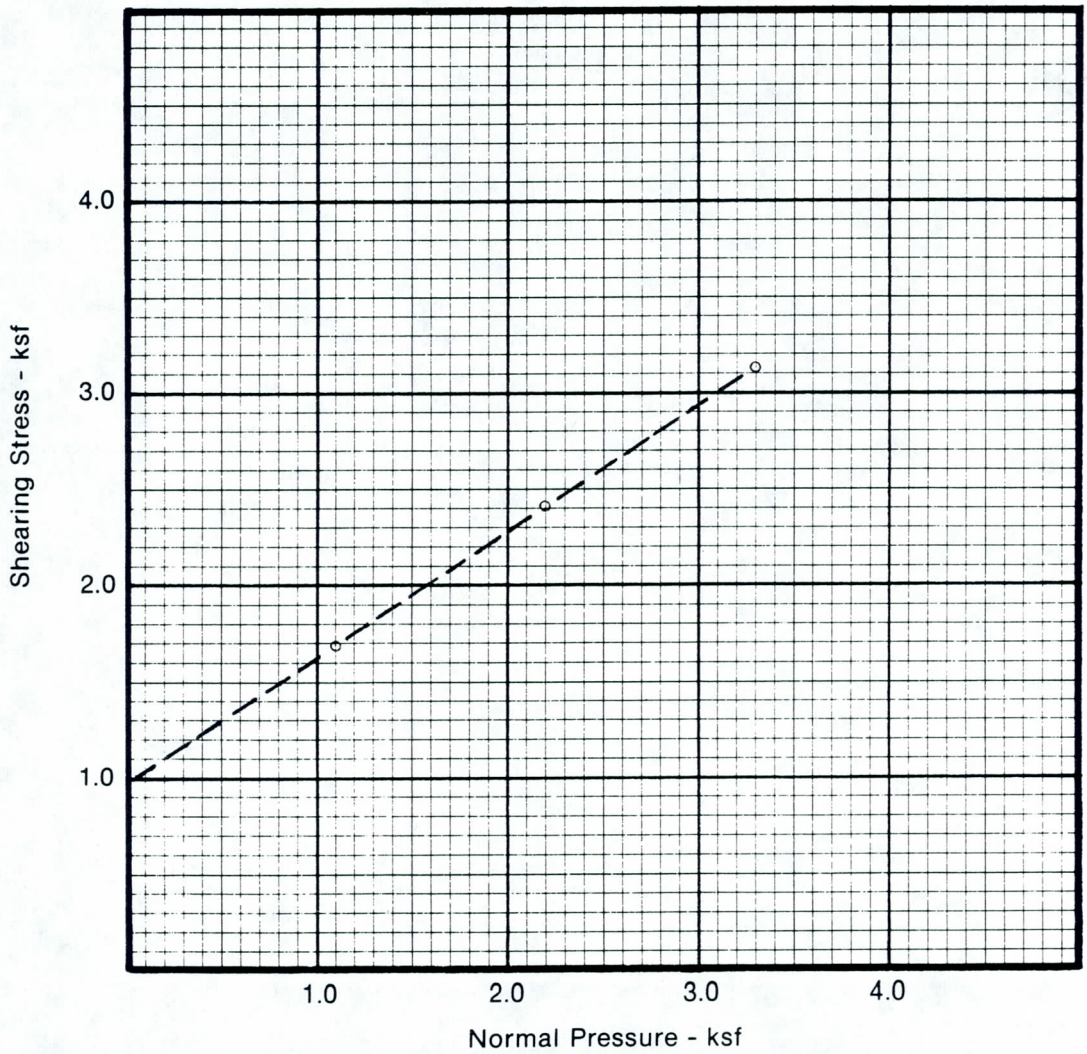
Material Silty Clayey Sandy Gravel (GP/GC-GM)

Sampled By TH/Thompson

TESTED: Direct Shear, portion of sample passing a #4 sieve was remolded to about 95% ASTM D698 maximum density, submerged, consolidated, then sheared at a displacement rate of 0.07 in./min.

RESULTS:

Friction Angle (Ø) = 33° Cohesion (c) = 1.0 ksf




Project No. 89-0872



TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

# REPORT ON LABORATORY TESTS

SAMPLE: Date 11-3-89

Source Noted Below

Type Bulk samples of auger cuttings

Material Soil

Sampled By TH/Thompson

TESTED: Sieve Analysis and Atterberg Limits

RESULTS:

| Sample | LL | PI | Sieve Size - 200 | 100 | 50 | 30 | 16 | Accum. % Passing 8 | 4 | 3/4" | 1" | 2" | 3" | * Class |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Composite B1&B2;0-2' | 30 | 14 | 21 | 25 | 28 | 32 | 37 | 45 | 54 | 94 | 98 | 100 | | GC |
| Composite B3&B4;0-2' | 25 | 4 | 20 | 24 | 27 | 31 | 36 | 44 | 54 | 88 | 93 | 100 | | GC-GM |
| P-5; 0 - 2' | 24 | 4 | 11 | 14 | 15 | 16 | 17 | 19 | 21 | 87 | 93 | 100 | | GP/ GC-GM |
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* Unified Soil Classification

Project No. 89-0872

# REPORT ON LABORATORY TESTS

SAMPLE: Date 11-17-89

Source Noted Below

Type Bulk samples of auger cuttings

Material Soil and rock fragments

Sampled By TH/Thompson

TESTED: Soluble Salts and Sulfates

RESULTS:

| Sample | Soluble Salts | Soluble Sulfates |
|---|---|---|
| B1; 5 - 10' | 0.091 | 0.021 |
| B4; 5 - 10' | 0.063 | 0.015 |
| P5; 0 - 5' | 0.21 | 0.012 |

# APPENDIX C

## SELECTED FIELD RESULTS
From Previous Geotechnical Investigation
(THA Project No. 89-0326)

## 3.3 RESULTS OF THE SEISMIC SURVEY (continued)

Table 1

SEISMIC REFRACTION GEOPHYSICAL SURVEY
DATA INTERPRETATIONS

(a) 66" Water Main Relocation Alignment

| Seismic Survey No. | S1 | | S2 | | S3 | |
|---|---|---|---|---|---|---|
| Profile Direction | F | R | F | R | F | R |
| V1 (fps) | 1000 | 1000 | 1250 | 1350 | 1150 | 800 |
| Material | t/f | t/f | wfg | wfg | t/f | t/f |
| Depth to Base (ft) | 2 | 3 | 4 | 2 | 5 | 2 |
| V2 (fps) | 2200 | 2400 | 4300 | 3800 | 2900 | 6000 |
| Material | wfg | wfg | mfg | mfg | mfg | sfg |
| Depth to Base (ft) | 19.1 | 22.8 | 19.6 | 19.2 | 26.6 | 33.4 |
| V3 (fps) | 8000 | 8000 | 7800 | 9500 | 14,000 | 13000 |
| Material | sfg | sfg | sfg | rock | rock | rock |

Seismic Velocity Interpretation / Material Correlation:

| | |
|---|---|
| t | Topsoil (residual soil) |
| t/f | Topsoil/fill |
| f/wfg | Fill/weakly cemented fanglomerate |
| wfg | Weakly cemented fanglomerate |
| mfg | Moderately cemented fanglomerate |
| sfg | Strongly cemented fanglomerate |
| sfg/rk | Strongly cemented fanglomerate (?) (possibly rock) |
| rock | Bedrock (probably schist; possibly quartzite) |

3.3 RESULTS OF THE SEISMIC SURVEY (continued)

Table 1 (continued)

SEISMIC REFRACTION GEOPHYSICAL SURVEY
DATA INTERPRETATIONS

(b) Intake Pipeline Relocation Alignment

| Seismic Survey No. | S4 | | S5 | | S6 | | S7 | | S8 | |
|---|---|---|---|---|---|---|---|---|---|---|
| Profile Direction | F | R | F | R | F | R | F | R | F | R |
| V1 (fps) | 1000 | 750 | 800 | 900 | 800 | 1000 | 800 | 1000 | 800 | 1000 |
| Material | t | t | t/f | t/f | f/wfg | f/wfg | t/f | t/f | t/f | t/f |
| Depth to Base (ft) | 2 | 4.5 | 5 | 11 | 6.5 | 9.5 | 8 | 5 | 3 | 2 |
| V2 (fps) | 4400 | 3400 | 4000 | 2250 | 2600 | 9500 | 6800 | 2700 | 3800 | 2100 |
| Material | mfg | mfg | mfg | wfg | wfg | sfg | sfg | wfg | mfg | wfg |
| Depth to Base (ft) | 42.8 | 13 | 33.5 | 22.8 | 19.7 | -- | -- | 18.1 | 12.7 | 11.5 |
| V3 (fps) | 17000 | 13000 | 8500 | 8000 | 9000 | -- | -- | 5200 | 5300 | 5300 |
| Material | rock | rock | rock | rock | rock | -- | -- | sfg | sfg | sfg |
| Depth to Base (ft) | -- | -- | -- | -- | -- | -- | -- | 56 | 32.3 | 28.5 |
| V4 (fps) | -- | -- | -- | -- | -- | -- | -- | 15000 | 15000 | 8000 |
| Material | -- | -- | -- | -- | -- | -- | -- | rock | rock | rock |

3.3 RESULTS OF THE SEISMIC SURVEY (continued)

Table 1 (continued)

SEISMIC REFRACTION GEOPHYSICAL SURVEY
DATA INTERPRETATIONS

(c) Discharge Line and 60" Water Main Relocation Alignments

| Seismic Survey No. | S9 | | S10 | | S11 | | S12 | |
|---|---|---|---|---|---|---|---|---|
| Profile Direction | F | R | F | R | F | R | F | R |
| V1 (fps) | 950 | 950 | 1350 | 1350 | 1750 | 1000 | 1500 | 1025 |
| Material | t | t | wfg | wfg | mfg | wfg | wfg | wfg |
| Depth to Base (ft) | 4 | 1 | 9 | 9.5 | 6 | 2.5 | 8.5 | 6 |
| V2 (ft) | 4000 | 5000 | 4600 | 5200 | 3500 | 3500 | 5600 | 5500 |
| Material | mfg | mfg | mfg | sfg | mfg | mfg | sfg | sfg |
| Depth to Base (ft) | 9.5 | 16.9 | 29.7 | -- | 32.4 | 19.1 | 43.9 | 45.4 |
| V3 (fps) | 8000 | 8000 | 6000 | -- | 8000 | 5700 | 15000 | 15000 |
| Material | sfg/rk | sfg/rk | sfg | -- | rock | sfg | rock | rock |

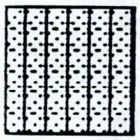
Seismic Velocity Interpretation / Material Correlation:

| | |
|---|---|
| t | Topsoil (residual soil) |
| t/f | Topsoil/fill |
| f/wfg | Fill/weakly cemented fanglomerate |
| wfg | Weakly cemented fanglomerate |
| mfg | Moderately cemented fanglomerate |
| sfg | Strongly cemented fanglomerate |
| sfg/rk | Strongly cemented fanglomerate (?) (possibly rock) |
| rock | Bedrock (probably schist; possibly quartzite) |

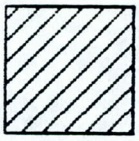
# LEGEND OF SOIL TYPES

**PROBABLE FILL, CLAYEY GRAVELLY SAND** (SC); brown; fine to coarse sand size; angular; dense; low plasticity; slightly damp.

**CLAYEY SAND & GRAVEL** (SC & GC); brown to light brown; stratified; fine to coarse sand size; medium dense to dense; low plasticity; angular; scattered cobbles and boulders.



**SILTY SAND** (SM); brown; fine; medium dense; non-plastic; damp.



**SILTY CLAY** (CL); light brown to dark brown; very stiff; medium plasticity; blocky; vesicular.

**BRECCIA FANGLOMERATE**; white to grey; angular gravel-and cobble-size rock clasts cemented in a matrix of caliche-cemented angular sand; degree of cementation generally varies from moderate to heavy; with zones of concrete-like consistency.

**SCHIST**; grey; generally hard but with some soft zones; numerous joints and fractures.

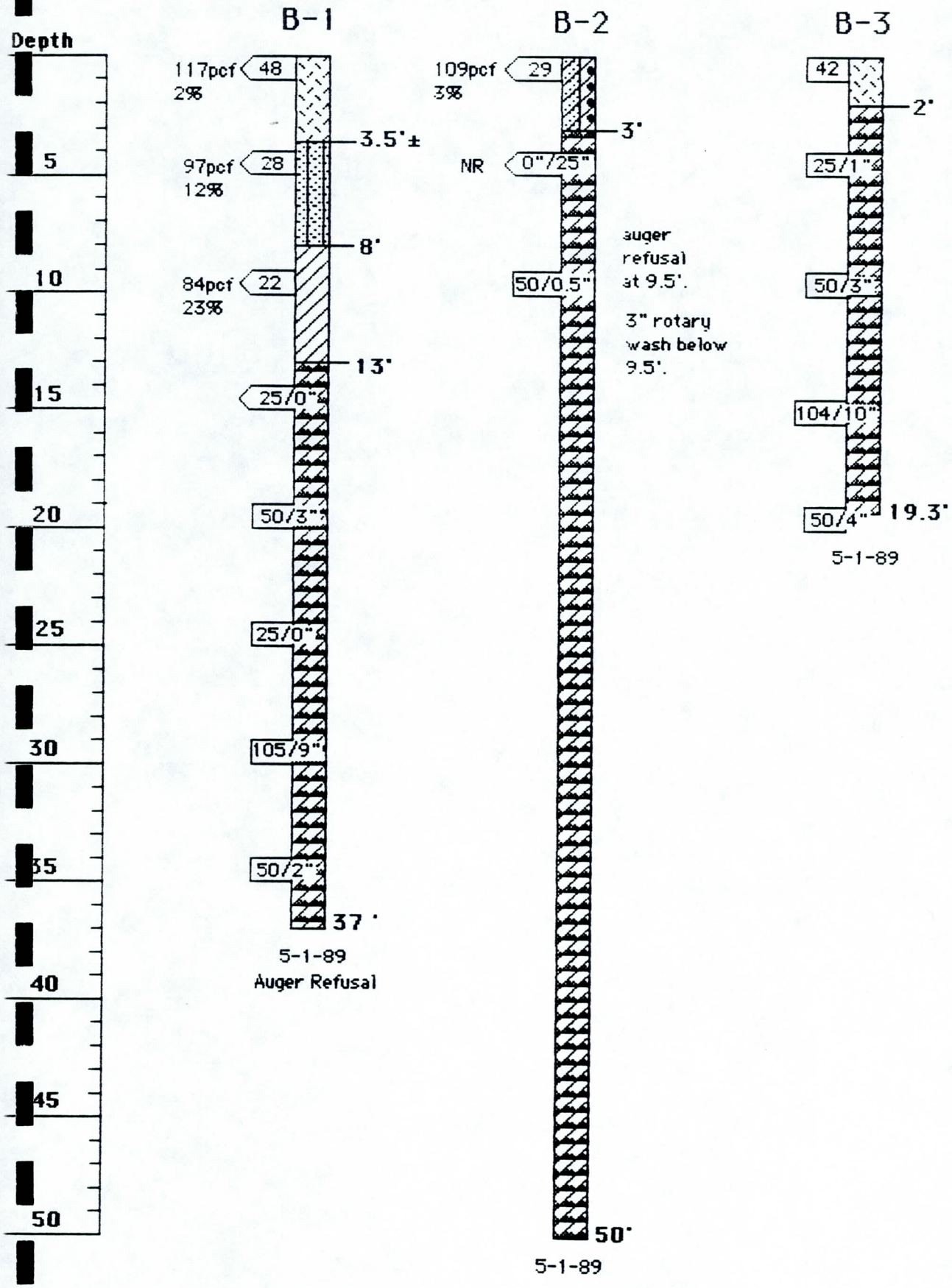
No free groundwateer was encountered in any of the borings during drilling.

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

Project No. 89-0326
Thomas-Hartig & Associates, Inc.

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.

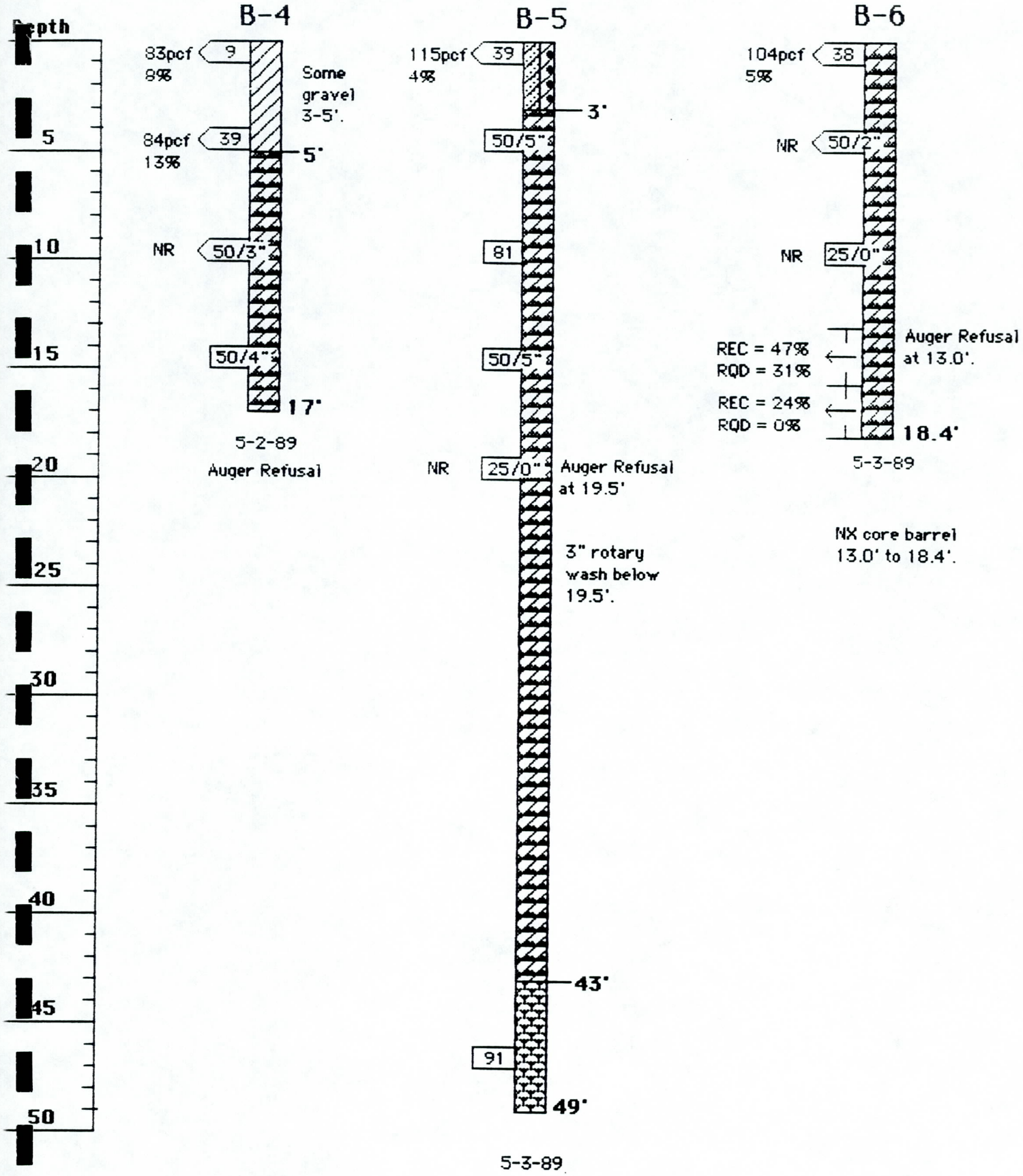
## GRAPHICAL BORING LOGS




**Project No. 89-0326**

**Thomas - Hartig & Associates**

# GRAPHICAL BORING LOGS




**Project No. 89-0326**

**Thomas - Hartig & Associates**

# APPENDIX D

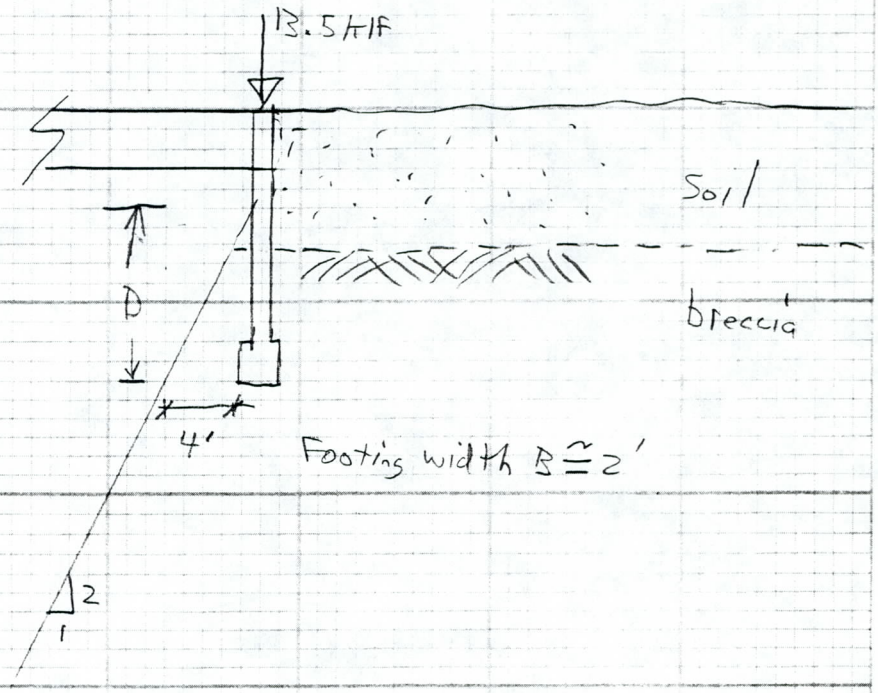
## BRIDGE FOUNDATION GEOTECHNICAL CALCULATIONS

PROJECT NO. 89-0872 COMPUTED BY SH DATE 11-10-89
SUBJECT Bridge Foundations CHECKED BY KJR DATE 11/17/89

Bridge load is fairly low, about 13.5 k/lf

Sequence:

1. Construct bridge
2. Then make 2:1 cut in breccia




For a footing 2' wide and 4' laterally from slope face, calculate bearing capacity. To be conservative, assume footing is only 2' from face. For that case,

$D = 4'$

$B = 2$

slope $\angle \beta = 63.4°$

Breccia properties:

Actual cohesive strength will be quite high, but due to variable cementation and some possible fractures, use a very conservative value.

PROJECT NO. 89-0972 COMPUTED BY SH DATE 11-10-89
SUBJECT Bridge Foundations CHECKED BY KM DATE 11/17/89

Assume $c = 5000$ psf. This is comparable to a hard clay.

Actual cohesion should be much higher

Assumed breccia properties:

$\phi = 35°$

$c = 5000$ psf $= 35$ psi

$\gamma = 130$ pcf

Previous testing indicated a breccia unconfined strength of over 4000 psi. For that sample

$c = 4000/2 = 2000$ psi

We are using $\frac{35}{2000} \cong 2\%$ of that strength.

Use figure 4b on p 7.2-136 of NAVFAC DM-7.2

Case II

$D/B = 4/2 = 2$

Use $D/B = 1$ on chart (this is conservative)

$B < H$, ∴ use $N_o = 0$

from chart, $N_{\gamma q} = 40 \pm$

$N_{cq} = 4$

PROJECT NO. 89-0972 COMPUTED BY SH DATE 11-10-89
SUBJECT Bridge Foundations CHECKED BY [illegible] DATE 11/17/89

$$q_u = cN_{cq} + \gamma \frac{B}{2} N_{\gamma q}$$

$$q_u = 5000(4) + 130\left(\frac{2}{2}\right)40$$

$$q_u = 25{,}200 \text{ psf}$$

$$q_{all} = q_u/3 = 8400 \text{ psf}$$

Report ABP = 8000 psf

## Settlement

Use Figure 11-8 of NAVFAC DM-7, 1971

Assume $k_{v1}$ for breccia is at least as high as $k_{v1}$ for dense to very dense gravelly sand

$\therefore$ use $k_{v1} = 300$ T/ft³, Assume B = 2', $q = \frac{13.5}{2} = 6.75$ ksf
$k_{v1} = 600$ k/ft³

$$\delta_v = \frac{2qB^2}{k_{v1}(B+1)^2} = \frac{2(6.75)(2)^2}{600(2+1)^2}$$

$$\delta_v = 0.01 \text{ ft}$$

$$\delta_v = 0.12 \text{ inch}$$

$\boxed{\text{Report settlement} < 1/4''}$

PROJECT NO. 99-2872 COMPUTED BY SH DATE 11-10-99

SUBJECT Bridge Foundations CHECKED BY [illegible] DATE 11/12/99

## Lateral Pressures

Use Rankine values, which for passive pressure is extremely conservative

By using these values, deflections will be small

$$K_A = \tan^2\left(45 - \frac{\phi}{2}\right)$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$ neglect cohesion (very conservative in breccia)

1. For soil or backfill, assume $\phi = 32°$, $\gamma = 110$ pcf

Active Equivalent fluid wt $= K_A \gamma = 0.31(110) = 34$ psf/ft (report 35 pcf)

Passive EFW $= 3.2(110) = 352$ psf/ft (report 250 pcf)

At rest EFW $= (1 - \sin 32°)110 = 52$ pcf (report 55 pcf)

2. For breccia, use $\phi = 38°$ and neglect cohesion, $\gamma = 130$ pcf

Passive EFW $= 4.2(130) = 546$ pcf (report 500 pcf) for continuous abutment

Increase by 50% for square loaded area, such as a pipe thrust block