GEOTECHNICAL SERVICES
RIO SALADO IMPROVEMENT

Near Farmer Avenue to McClintock Drive
Tempe, Arizona

Registered Professional Engineer (Civil)
CERTIFICATE NO.
10971
KENNETH L.
RICKER
1-10-90
Date Signed
ARIZONA, U.S.A.

Prepared for

City of Tempe
P.O. Box 5002
Tempe, Arizona

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

CRSS

RECEIVED
FEB 28 1990

D.J.R.
BG 3/7/90

Memo

To	Howard Hargis
From	George Cotton GKC
Re	Bed Material Gradation
Date	February 19, 1990
Copies	180.06.1

Thomas Hartig recently completed sampling of channel bed material at 21 locations for the Rio Salado Project. Laboratory analysis of the portion of the material which is less than six inches in diameter was conducted. An estimate of the fraction of the oversized material was made prior to preparation of the laboratory work. Table 1 summarizes the measured gradations and the estimated fraction of oversized material for each sampled location.

The measured gradations were plotted for the purpose of identifying a design gradation (see Figures 1 to 4). Two groupings of gradation measurements were identified. The first group consisted of 12 samples taken at various locations in the Salt River channel (samples 25, 26, 27, 28, 29, 33, 35, 38, 39, 40, 44 and 45). The second group consisted of six samples taken at locations in the Indian Bend Wash channel (samples 41 [surface and subsurface], 42 and 43) and at two locations immediately downstream of the Indian Bend outlet in the confluence with the Salt River channel (samples 36 and 37).

Based on this sampling, the mean diameter of the Salt River bed material ranges from 10mm to 60mm, while the mean diameter for the Indian Bend Wash gradations is substantially finer and ranges from 0.1mm and 1.0mm. Three remaining samples (samples 23, 24, and 34) occur in the Salt River channel and include finer sediment sizes relative to the more typical Salt River gradation. Samples 23 and 24 are gap graded with sizes less than 1.0mm and greater than 50mm occurring. Sample 34 is generally finer relative to the typical Salt River gradation, with a mean diameter of about 3.0mm compared to 10.0mm, respectively.

The design gradation was based on an envelope curve which was plotted at the upper limit of the group of 12 samples for the Salt River. The three gap-graded samples were excluded from the analysis, since they do not represent any consistent gradation trend in the river but rather a local variation in the gradation due to the sedimentation process.

The design gradation was compared to the Simons, Li & Associates design gradation which was developed for the ADOT channel design (see Figure 5). The two design gradations are essentially the same, which indicated that there is a fairly representative bed material gradation in the Salt River channel through Tempe. The confluence with Indian Bend Wash locally changes the gradation of the Salt River; however, most of this change is surficial and has not mixed with the characteristic Salt River gradation. Therefore, from the standpoint of channel response in the Salt River, the characteristic Salt River gradation will be most important for use in evaluating the potential scour.

Table 1. Sampled Bed Material Gradations

Boring	Location	Range	Sieve Size - 200	100	50	30	16	8	4	3/4"	1"	2"	3"	Over Size
23	400+00, 430' lt	0-6	5	31	71	82	87	89	90	94	95	100		35
24	409+00, 230' lt	0-16	12	24	33	35	36	36	37	42	47	73	95	0
25	421+00, 240' lt	0-11	1	2	5	13	25	34	41	61	68	87	100	55
26	428+00, 350' lt	8-17	2	3	5	14	27	36	43	67	74	87	100	35
27	434+00, 475' lt	0-8	1	2	5	17	36	44	51	71	77	94	100	35
28	440+00, 258' lt	8-18	2	3	6	11	18	26	34	59	66	83	100	40
29	448+00, 525' lt	0-8	1	2	4	12	24	31	37	53	59	71	100	45
33	458+00, 450' lt	0-8	1	1	4	11	25	34	42	63	69	85	100	45
34	464+00, 325' lt	9-17	5	7	12	22	35	43	49	68	74	91	100	0
35	474+00, 350' lt	0-8	1	3	7	17	29	37	44	69	77	93	100	35
36	482+00, 525' lt	0-7	1	4	20	68	91	95	96	98	98	100		0
37	490+00, 600' lt	0-10	0	2	14	41	68	80	84	94	95	98	100	0
38	498+00, 600' lt	8-17	2	3	7	13	23	32	39	60	68	90	100	55
39	508+00, 700' lt	0-8	1	3	12	27	39	46	52	70	77	91	100	35
40	516+00, 650' lt	8-17	1	2	6	15	27	36	43	63	70	90	100	45
41	98+00, 325' rt	0-6	36	64	93	97	98	99	99	100				0
41	98+00, 325' rt	6-10	7	15	52	85	94	96	97	100				0
42	96+00, 325' rt	0-5	9	17	27	36	45	51	56	79	84	98	100	0
43	308+00, 0' rt	0-8	68	79	84	86	89	92	95	100				0
44	91+00, 325' rt	0-8	3	4	11	25	44	53	59	74	81	97	100	35
45	207+00, 0' rt	0-8	4	8	15	23	32	40	48	69	76	92	95	25
avg	26,27,28,29,33, 35,39,40,44,45		1.8	3.3	7.8	17.7	30.7	38.9	46.0	66.6	73.5	89.3	100.0	37.9

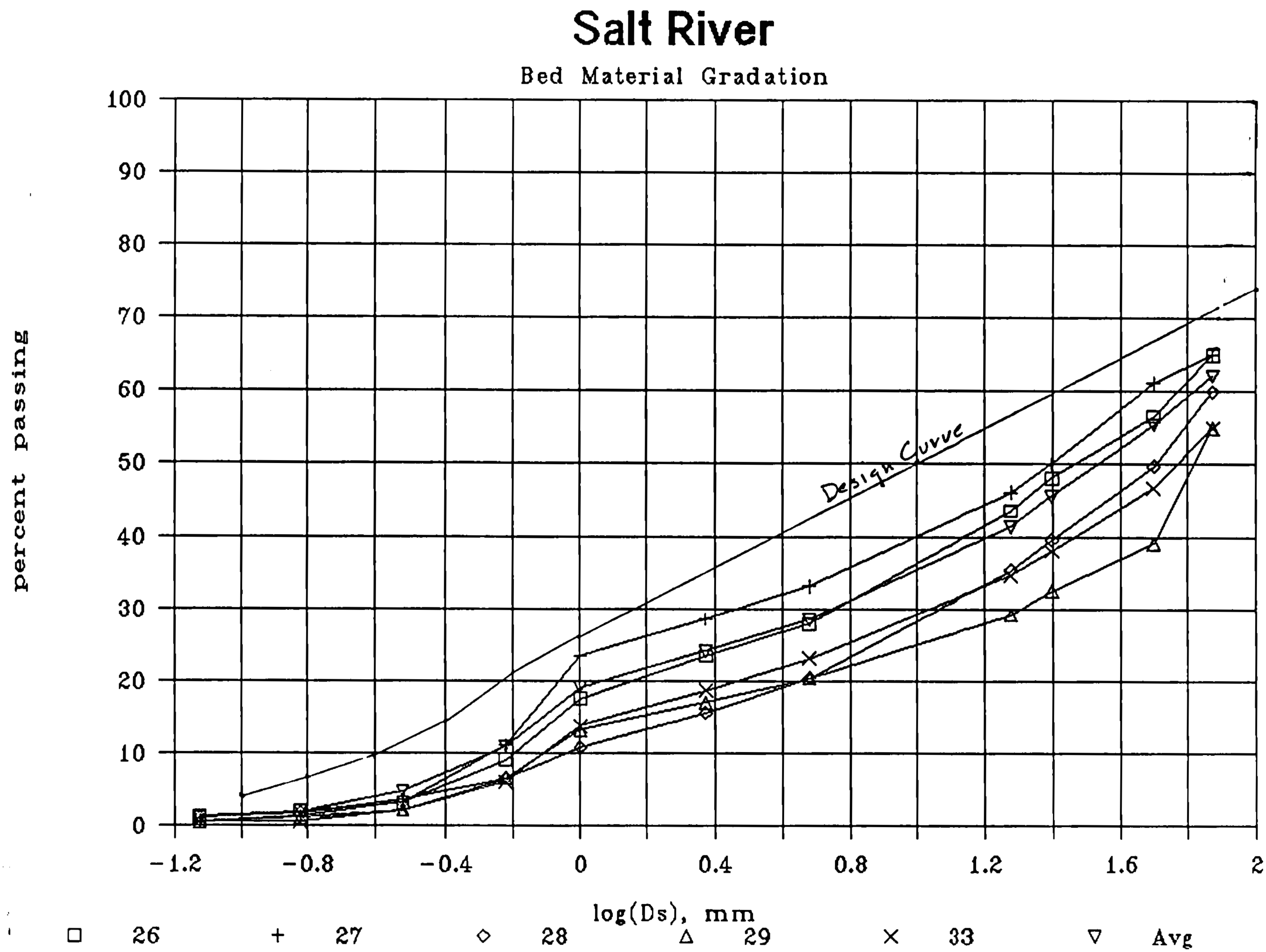

Salt River
Bed Material Gradation
100
90
80
70
60
50
40
30
20
10
0
percent passing
Design Curve
−1.2
−0.8
−0.4
0
0.4
0.8
1.2
1.6
2
log(Ds), mm
26
27
28
29
33
Avg

Figure 1

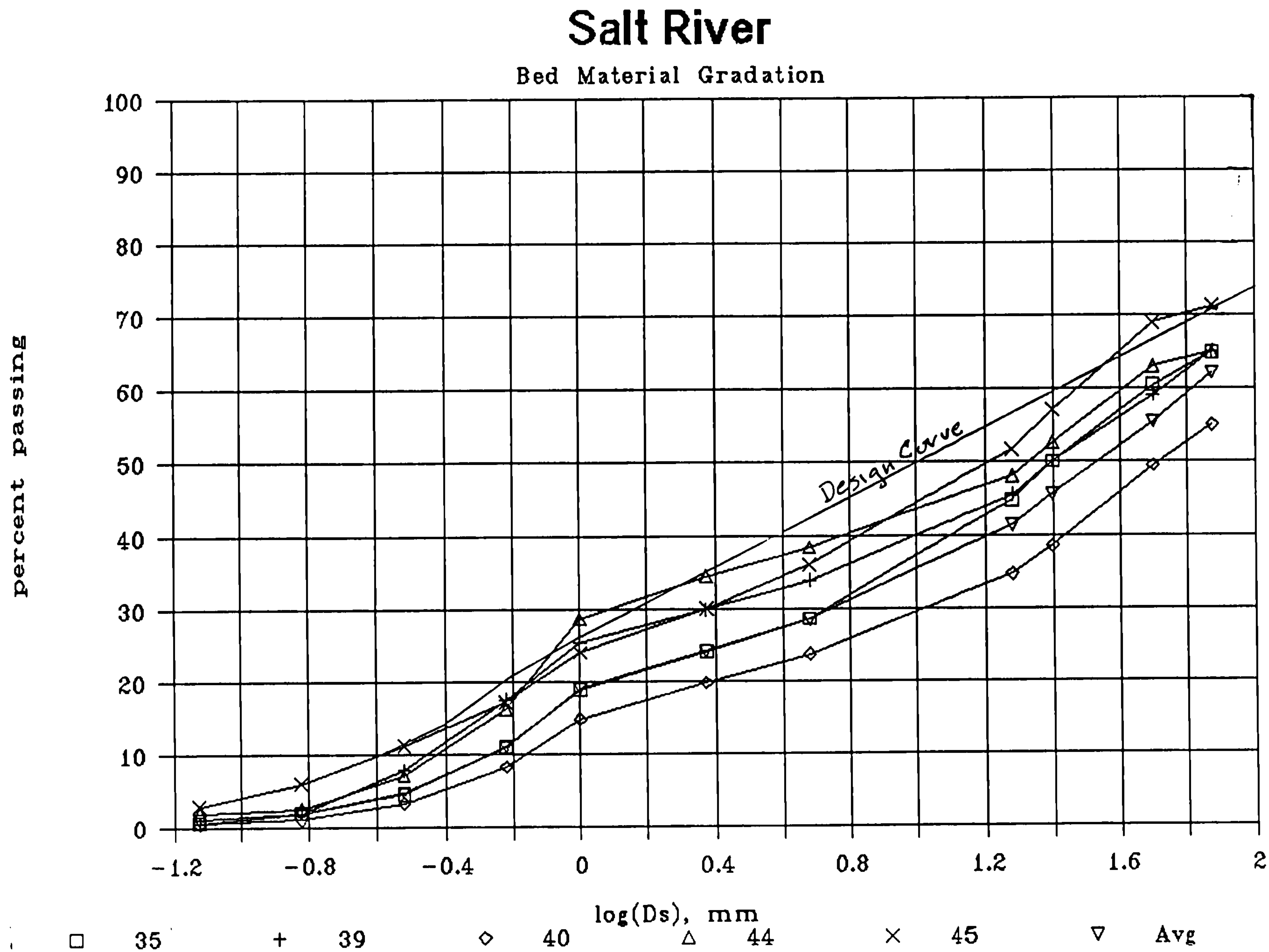
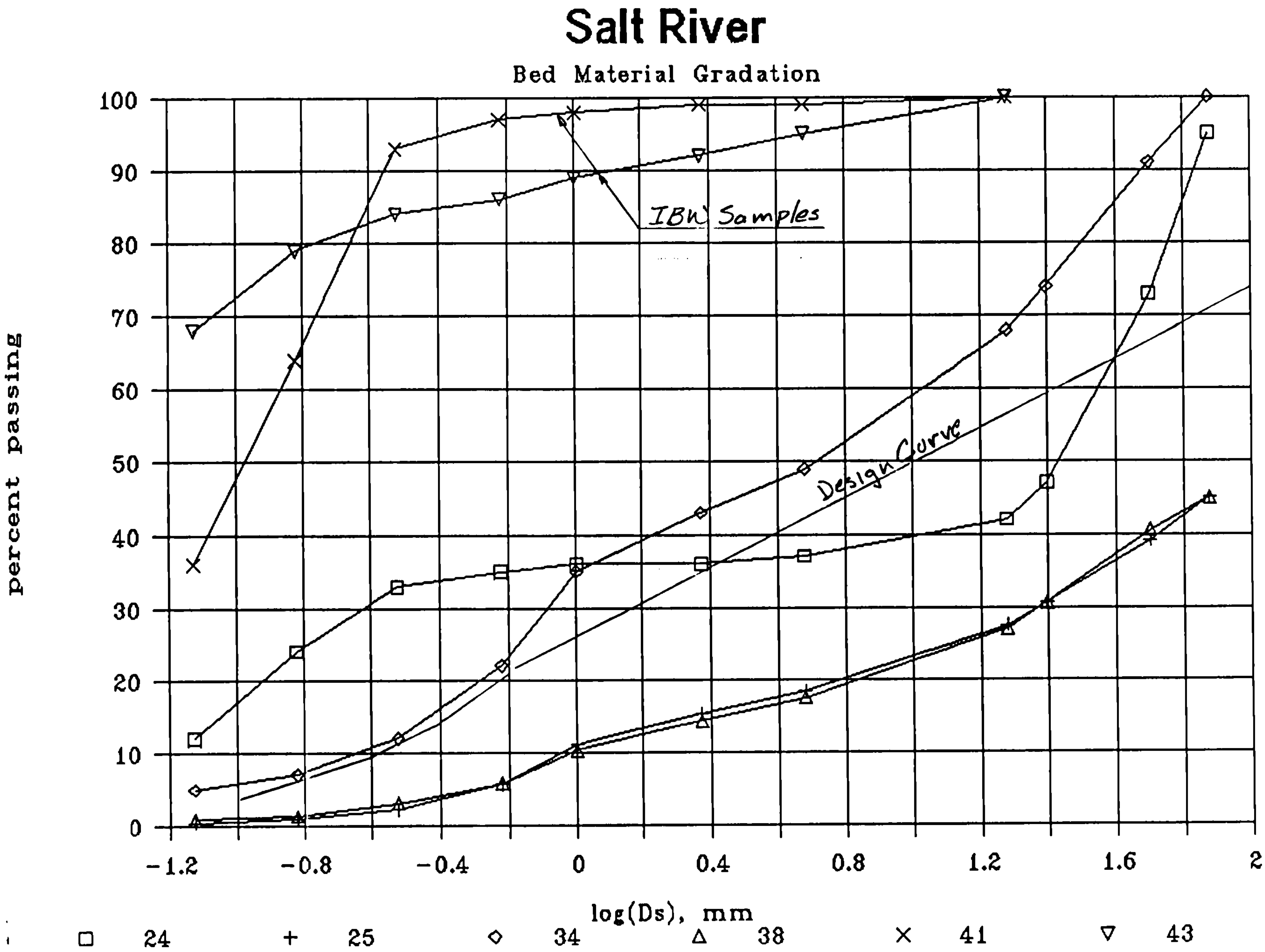
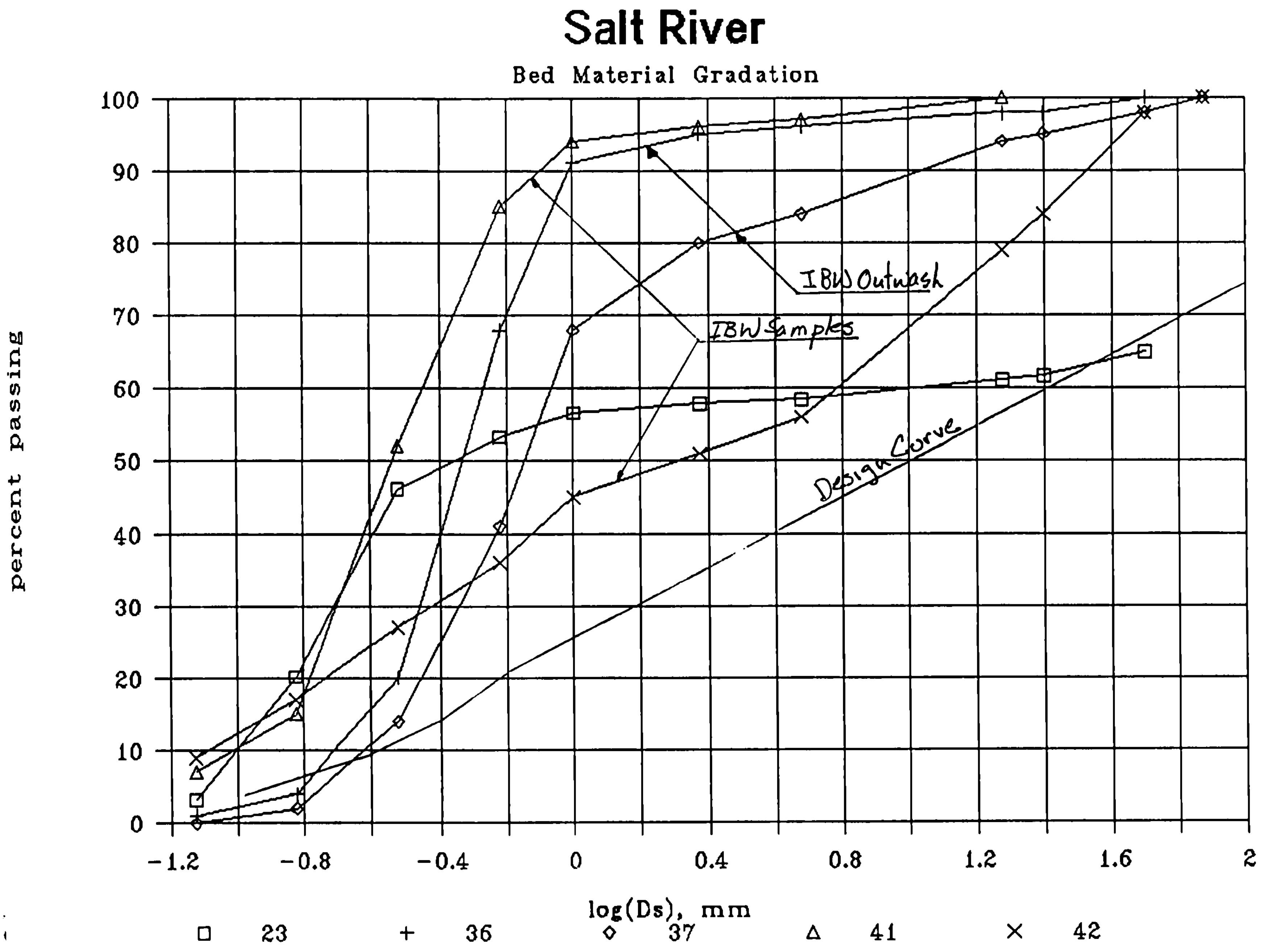

Salt River
Bed Material Gradation
percent passing
0
10
20
30
40
50
60
70
80
90
100
-1.2
-0.8
-0.4
0
0.4
0.8
1.2
1.6
2
log(Ds), mm
Design Curve
35
39
40
44
45
Avg

Figure 2



Salt River
Bed Material Gradation
percent passing
log(Ds), mm
100
90
80
70
60
50
40
30
20
10
0
-1.2
-0.8
-0.4
0
0.4
0.8
1.2
1.6
2
IBW Samples
Design Curve
24
25
34
38
41
43

Figure 3



Salt River
Bed Material Gradation
percent passing
100
90
80
70
60
50
40
30
20
10
0
-1.2
-0.8
-0.4
0
0.4
0.8
1.2
1.6
2
log(Ds), mm
IBW Outwash
IBW Samples
Design Curve
□ 23
+ 36
◇ 37
△ 41
× 42

Figure 4

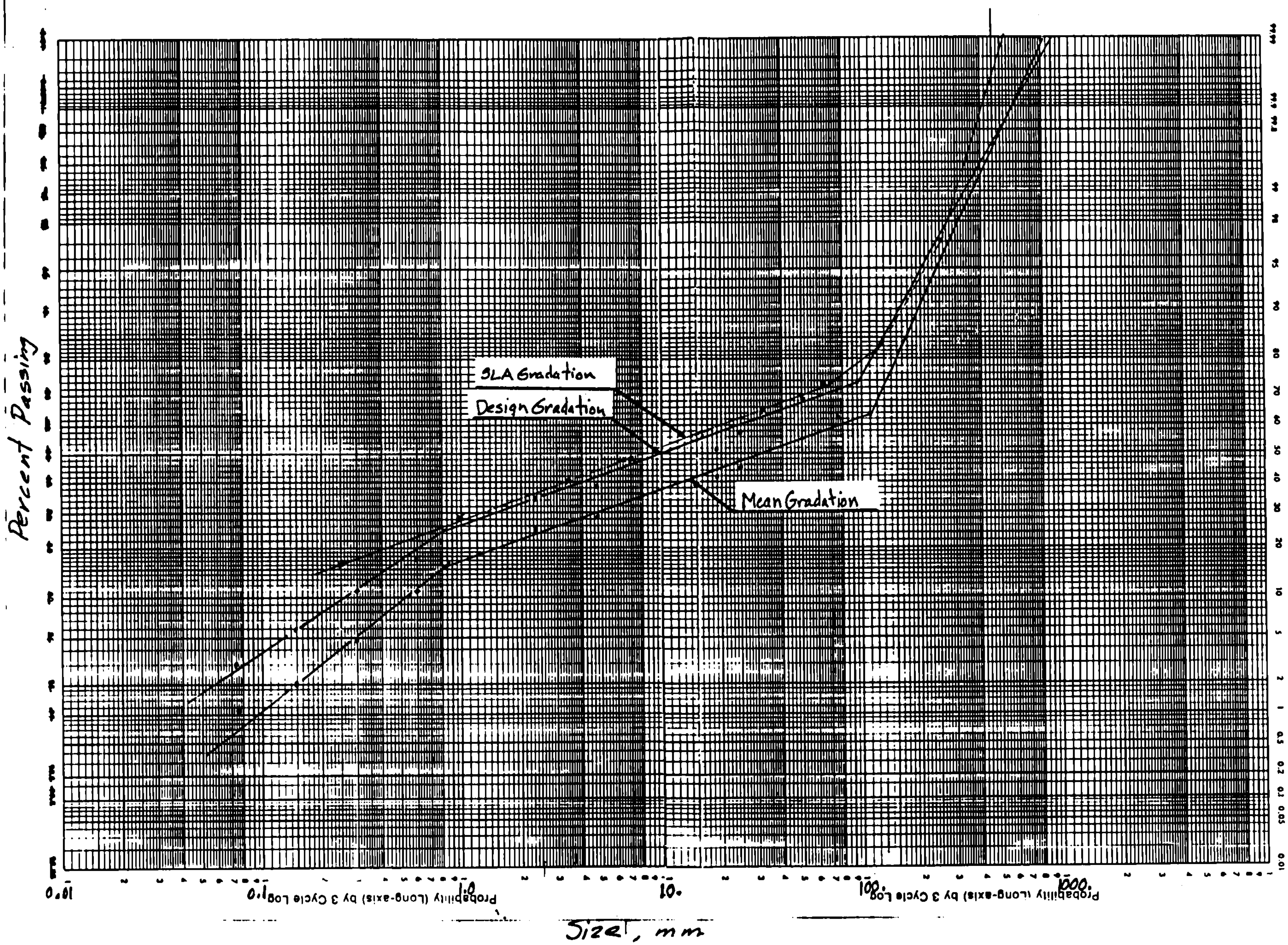

SLA Gradation
Design Gradation
Mean Gradation
Percent Passing
0.01
0.1
1.0
10.
100.
1000.
Size, mm

Figure 5

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FEB 28 1990

CH ENG		P & PC	
DEI		HYDRO	
ADMIN		MGT	
FINANCE		FILE	
C & O	1	DLR	
ENGR	2		

REMARKS

3/7/90

Memo

To Howard Hargis

From George Cotton GKC

Re Indian Bend Wash Drop Structure

Date February 27, 1990

Copies 180.06.1

This memo presents my assessment of baseline conditions for Indian Bend Wash at the outlet to the Salt River and at the proposed crossing of the East Papago Freeway. It also responds directly to comments by SLA and Gannett Fleming on the drop structure location and design.

The SLA baseline, which was used by Gannett Fleming in the design of the bridge foundation, assumed six feet of general scour and 29 feet of local scour for each pier. We have found that base level lowering of the Salt River has depressed the profile of the Salt River about ten feet below the profile of Indian Bend Wash. This difference in the respective profiles should have been accounted for in the scour analysis for the bridge. The channelization project will result in an additional lowering of the Salt River on the order of three to four feet. The 19.5 foot height for the drop structure which was identified for the Rio Salado project is therefore the sum of the following:

Base Level Difference	10.0 feet
General Scour	6.0 feet
Channelization	3.5 feet
Drop Height	19.5 feet

The increased foundation depth required due to channelization will be offset by a reduction in pier scour depth due to the reduction in approach velocity. This reduction in flow velocity would occur naturally in the existing system after propagation of the headcut upstream through the bridge or as a result of the energy dissipation provided by the drop structure.

Regarding the letters by Dennis Richards, SLA, and Terry Koons, Gannett Fleming, on structure type, design and construction issues, I have the following response:

1. The baffle-block spillway drop structure suggested by SLA is a reinforced concrete structure and is significantly more expensive compared to the planned CSA multi-drop structure.

2. Relocating the drop structure downstream of the bridge significantly increases the length of bank protection required for the north bank and reduces the developable land area on the south bank. Again, this is a significant cost increase rather than a cost reduction as claimed by SLA.

3. SLA points out the need for 200 feet of levee to act as a spur to deflect local scour away from their proposed location of the drop structure. This points out that by placing the drop structure downstream of the bridge, the structure is at increased risk of scour and that to mitigate this risk, additional structures (at additional cost) are, in fact, required.

4. The junction levee was removed from Tempe's channel design after two-dimensional modeling of the confluence indicated it was unnecessary and that flows from either Indian Bend Wash or the Salt River would not adversely impinge on channel banks. This analysis was presented in the initial river mechanics report, a copy of which was provided to SLA for review.

Regarding Gannett Fleming's comments:

1. General and local scour will be unchanged and, in all likelihood, improved.

2. The drop structure maintains the existing invert elevation at the crest of the upper drop structure. The high water elevation will be reduced as the flow accelerates over the drop structure.

3. Water velocity will be decreased not increased due to energy dissipation provided by the drop structure.

4. Unequal Frame Action at Piers - Sketch No. 1 inaccurately depicts existing conditions at the crossing. Since it is ADOT's policy to disregard

the effect of the drop structure in bridge foundation design, the combined affect of local, general and headcutting scour would be considered in any case.

5. Construction conflicts identified can be easily resolved by a logical construction sequence. Since the drop structure and bridge will be part of the same construction package, there seems to be no impediment to such a logical sequence.

6. Higher bridge cost due to construction difficulties are extremely unlikely. For example, construction of the bridge substructure, followed by construction of the drop structure, with final construction of bridge super structure is one alternative for a simple logical sequence of construction that would add nothing to the cost of the bridge.

7. New levee heights are below the roadway profile throughout Section 6 of the East Papago Freeway.

8. Tempe and CRSS are coordinating closely with both APS and SRP on the location of new and existing overhead power lines. We are well aware of the profile of the four-city sanitary trunkline at the location of the drop structure.

9. The north levee will be realigned to avoid a conflict with Ramp B of the East Papago Freeway.

In conclusion, we see no evidences that either design or construction costs will be increased for the Indian Bend Wash bridge of the East Papago Freeway due to construction of a drop structure. The drop structure is necessary in order to control an existing difference between the profile of Indian Bend Wash and the Salt River. ADOT policy requires that the bridge structure design account for such scour conditions.

CRSS

Memo

To Howard Hargis

From George Cotton GKC

Re Relocation of Grade Control No. 4

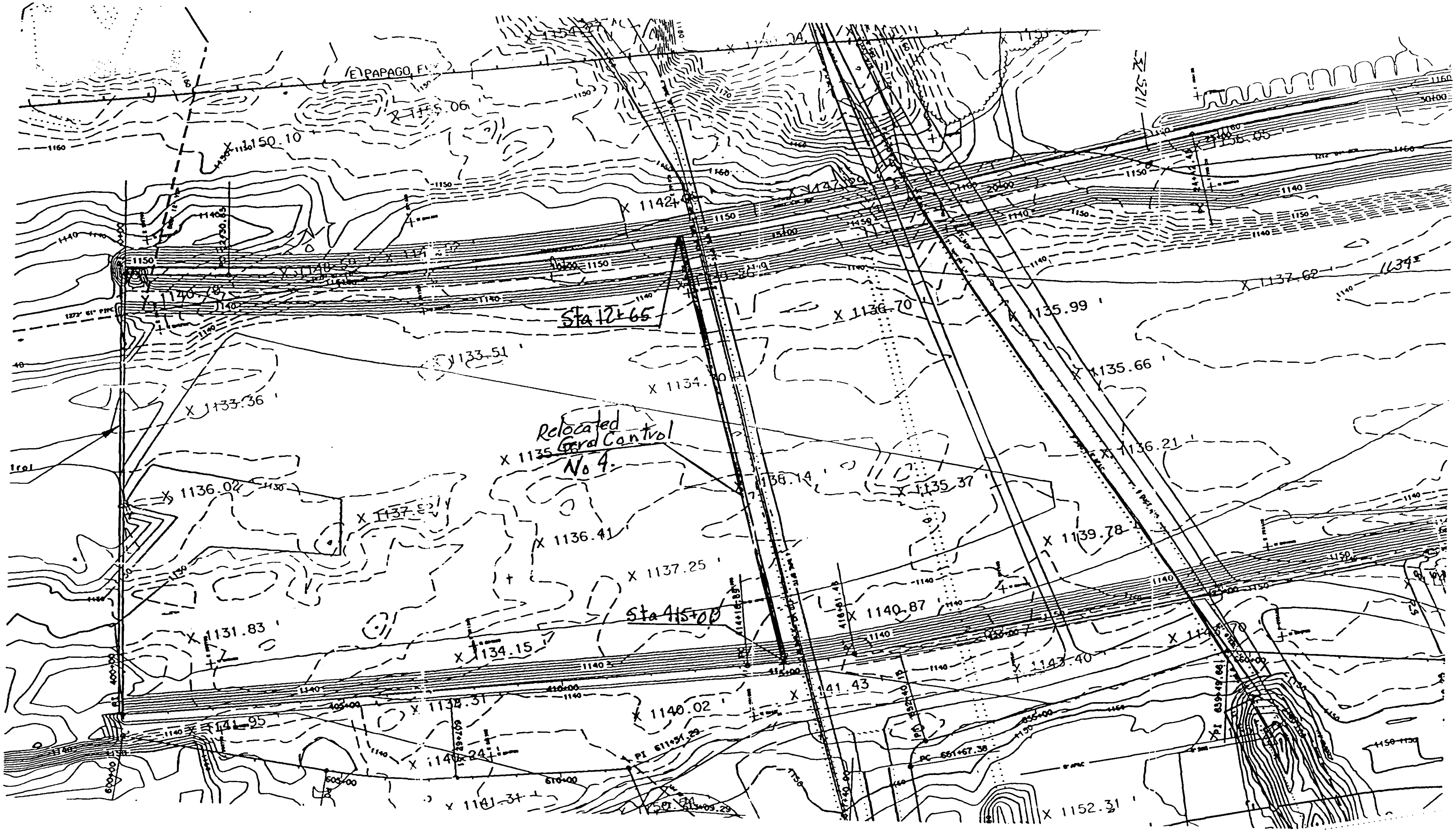
Date February 19, 1990

Copies 180.06.1

In the Salt River Working Group meeting conducted February 14, 1990, Tom Monchak (ADOT East Papago Management Consultant) requested additional information on the relocation of Grade Control No. 4. Attached is a profile for the proposed new location of this grade control showing existing ground, the proposed channel section, an estimate of the rock profile, and the location of SPRR bridge piers. Also attached is a plan for new location of the grade control structure.

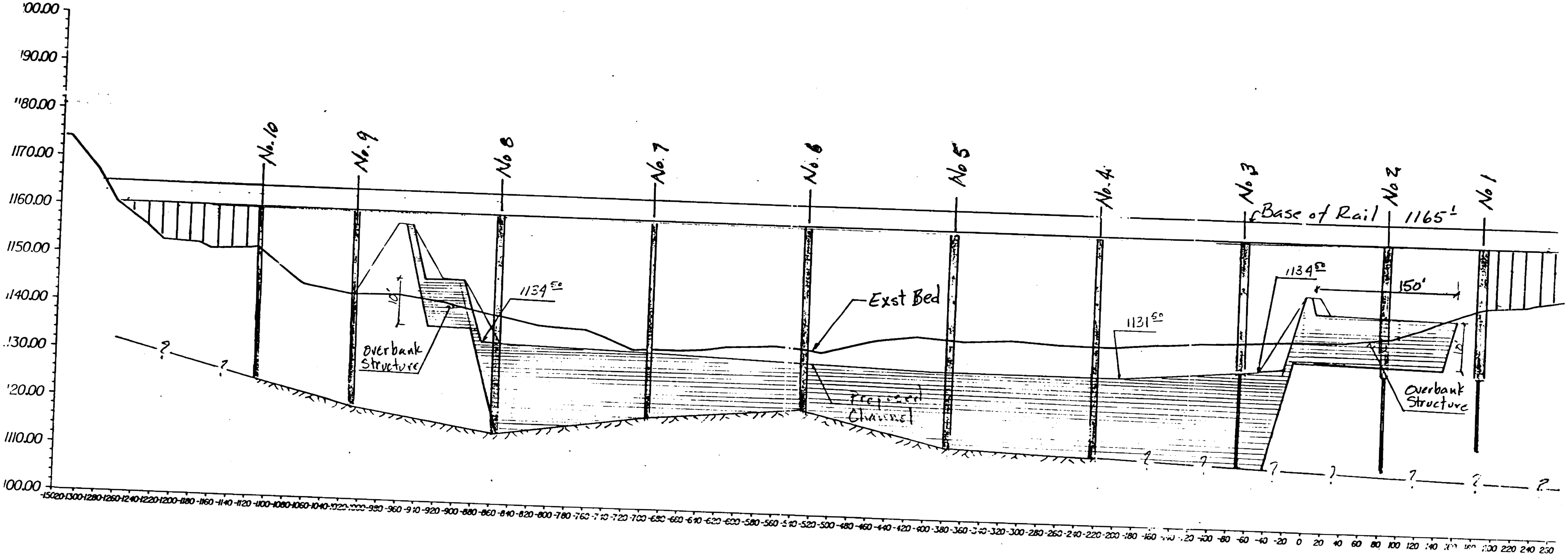
ADOT would like to change order this relocation into the present construction contract. In order to do this, they will need to build not only the grade control structure but also the associated levee and bank stabilization downstream of the SPRR bridge which is planned for the Tempe Rio Salado Channelization.

1 DJe
2 BGP 3/7



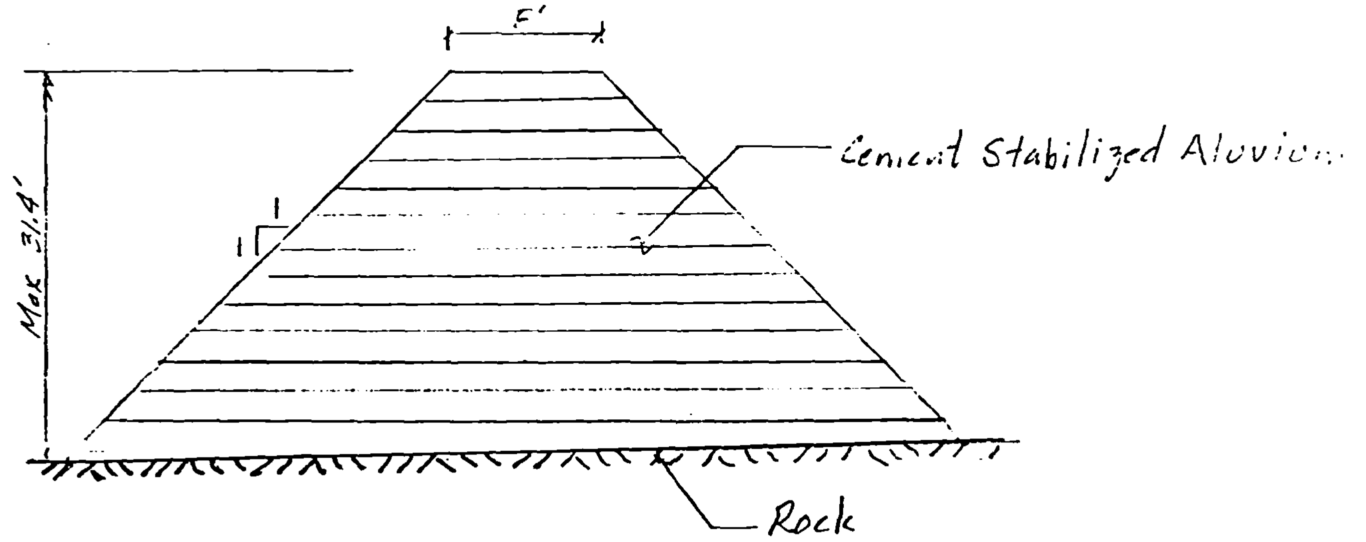
Plan View

Relocated Grade Control

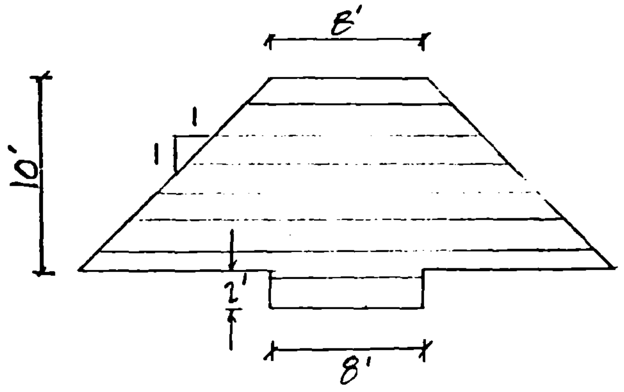


Profile along
Grade Control Axis

CRSS CRS SIRRINE		
PROJECT Rio Salado	JOB NO [illegible]	NO
CLIENT Tempe	DESIGNED BY GKC / DATE 5/19/[illegible]	
SUBJECT Relocated Grade Control	CHECKED BY / DATE	OF



Typical Section
Main Channel Section



Typical Section
Overbank Section

CRSS

Memo

To	Howard Hargis
From	George Cotton GKC
Re	Indian Bend Wash Drop Structure Alternatives
Date	February 19, 1990
Copies	180.06.01

At the request of the Flood Control District, an alternative to the multiple drop design which was submitted in the initial design plans for the Rio Salado channelization has been prepared. A single drop design was studied. Assuming that the cement stabilized stilling basin can perform acceptably at the high velocities which result from a single drop, the drop structure will require 19 percent less cement stabilized alluvium (CSA) to construct. The velocity in the stilling basin is quite high for this design, 42 fps compared to 28 fps for the three drop design. For this velocity, a CSA stilling basing may erode severely during a flood event.

To check this, I called Ken Hansen at PCA. He said that based on experience of soil cement installations in Tucson, soil cement is abrasion resistant up to maximum velocities in the range of 15 to 18 fps. The maximum velocity can be increased if the +4 fraction of the mix is increased, creating a roller-compacted-concrete (RCC) type mix. He felt that velocities of 30 fps could be achieved with this type of mix. He felt that the Salt River gradation (he was familiar with the CSA design for the East Papago Freeway) would be similar to a RCC mix. He recommended the use of 18-inch high steps on the spillway face to promote air entrainment which would decrease problems associated with abrasion and negative pressures.

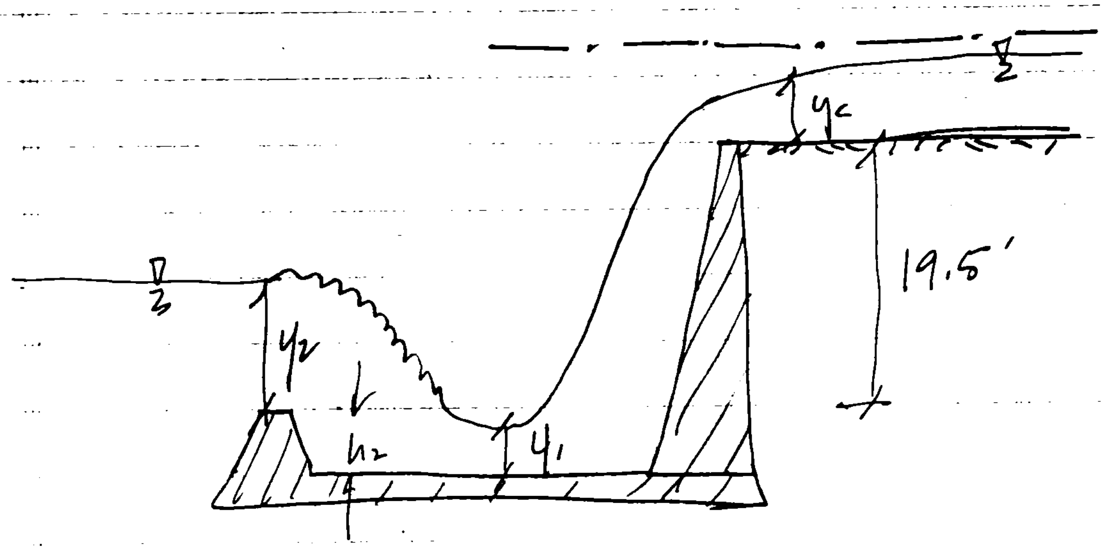
3/7/90

IBW Drop Structure
Study of Single Drop

$q = 90$ cfs/ft $\qquad h_d = 19.5$ ft

$y_c = (q^2/g)^{1/3} = (90^2/32.2)^{1/3} = 6.3$ ft

$E = \frac{3}{2} y_c = 1.5 \cdot 6.3 = 9.5$ ft



$$E = \frac{(q/y_1)^2}{2g} + y_1$$

$$y_1^3 - E y_1^2 + q^2/2g = 0$$

$$y_1^3 - 29 y_1^2 + 125.8 = 0$$

$$y_1 = 2.1652 \text{ ft}$$

$$V = q/y = 90/2.1652 = 41.6 \text{ fps}$$

$$\mathbb{F} = V/\sqrt{g y_1} = \frac{41.6}{\sqrt{32.2 \times 2.1652}} = 5.0$$

$TW = (TW/y_1)\,y_1 = 7.2\,(2.2) = 15.8$ ft

$L = (L/d_2)\,d_2 = 6\,(15.8/1.1) = 86.2$ ft

$h_2 = 1.25\,y_1 = 1.25\,(2.2) = 2.75$

<u>Estimate quantities per linear foot</u>

Drop:

$((2{\times}8 + 1.5(19.5+8)\cdot 2)/2)\,(19.5+8)/27 = 50.2$

Stilling Basin

$(5\,(86 + (86 - 27.5))/2)/27 = 14.5$

End sill

$((2{\times}8 + 1.5{\times}2{\times}8)/2)\times 8/27 = \underline{5.9}$

70.6 CY/ft

- This is 19% less compared to the 86.9 CY/ft for the triple drop.

- Velocity at base of drop is 50% higher

CRSS

Memo

To: Howard Hargis

From: George Cotton

Re: SPRR Bridge/Grade Control No. 4 Relocation

Date: February 6, 1990

Copies: 180.06.1

Because of the proposed lower profile for the Salt River, it is recommended that Grade Control No. 4 be relocated to the downstream side of the Southern Pacific Railroad bridge. The dam axis should be located parallel to the bridge with the south abutment at Station 415+00 on the south levee control line (N 284, 809.26, E 291, 984.74). The dam crest should be at elevation 1131.50.

The first three piers of the SPRR bridge which are north of the south abutment are founded on piles. Piers I and II are located on the south overbank terrace, while Pier III is in the main channel. The following tabulation gives a comparison of pier caisson elevation to ground, rock and minimum pile tip elevations.

Location	Caisson Depth	Minimum Pile Tip	Rock	Proposed Ground
Pier I	1133.39	1125.39	?	1144.97
Pier II	1133.64	1127.39	?	1144.97
Pier III	1127.72	1126.39	1116.5	1131.50

CRSS
CRS SIRRINE

Transmittal

CRSS, INC. - 4041 North Central, Suite 650, Phoenix, Arizona 85012-3306, (602) 263-5309

To: Warren Rosebraugh
Flood Control District of Maricopa County

Enclosed:
- ☐ Letter/Memo
- ☐ Specification(s)
- ☐ Plans
- ☐ Samples
- ☐ Drawing(s)
- ☑ Other – Calculations

Transmitted as Checked:
- ☐ For Information
- ☑ For Approval
- ☐ For Review and Comment
- ☐ Other
- ☐ As Requested
- ☐ Approved as Submitted
- ☐ Approved as Noted
- ☐ Return for Correction
- ☐ Return to Us
- ☐ Resubmit for Approval

Copies	Date	Rev. No.	Description	Action Code
1	5/15/90		Sieve analysis for large bulk samples, Thomas-Hartig	
1	5/16/90		Revised design gradation, CRSS	
1	5/16/90		Revised bed scour computations	

Action Code
A. Action indicated on item transmitted
B. No action required
C. For signature and return to this office
D. For signature and forwarding as noted below under REMARKS
E. See REMARKS below

Remarks Calculations of bed scour for the reach from Grade Control No. 3 to Grade Control No. 4 are also included in the computations using both the SLA gradation and the revised CRSS gradation.

By: George K. Cotton GKC

Distribution: Don Rerick
Howard Hargis
180.06

Job: Rio Salado Channelization
Location: City of Tempe
Job Number: RIOIN
Date: May 17, 1990
Via: Courrier

DATE REC'D ________ BY ________

WHITE: Original CANARY: Acknowledgement Copy (return this copy immediately) PINK: Project File GOLD: Originator

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

15 May 1990

City of Tempe
P.O. Box 5002
Tempe, Arizona 85281

Attention: Howard Hargis

Project: Geotechnical Services
Rio Salado Improvements
Tempe, Arizona

Project No. 89-0919
Supplement No. 2

At your request, this firm has performed supplemental services for the subject project. The supplemental services consisted of performing sieve analysis of the plus 3 inch material at 6 previously sample locations. A large bulk sample (4,840 to 10,200 pounds) was obtained from each location with a backhoe and placed in a dump truck which was then weighed on a truck scale. The material was sieved over a 3 inch mesh standing screen. The plus 3 inch size material was then hand sorted into minus 6 inch, 9 inch, 12 inch, and 18 inch material. The particles in each hand sorted size were weighed and the percent passing was calculated.

The results of these sieve analysis are attached.

This supplement shall be attached to the original report and shall become a part thereof. Please call if you have any questions or if we may be of further assistance.

Respectfully submitted,
Thomas-Hartig & Associates, Inc.

By: Kenneth L. Ricker, P.E.

Registered Professional Engineer (Civil) Certificate No. 10071 KENNETH L. RICKER Date Signed 5-15-90 ARIZONA, U.S.A.

Reviewed by: Frank M. Guerra

Registered Professional Engineer (Civil) Certificate No. 15272 FRANK M. GUERRA Date Signed 5-15-90 ARIZONA, U.S.A.

/go
Copies to: Addressee (1)
CRS/Sirrine (4)

Chandler Phone (602) 961-1169 Fax (602) 940-0952 • Phoenix Phone (602) 437-5450

REPORT ON SIEVE ANALYSIS

SAMPLE: Date: 5-15-90

Source: As Noted Below
Type: Large Bulk Samples
Material: River Bed
Sampled By: TH/White

TESTED: Sieve Analysis of plus 3 inch material

TEST RESULTS

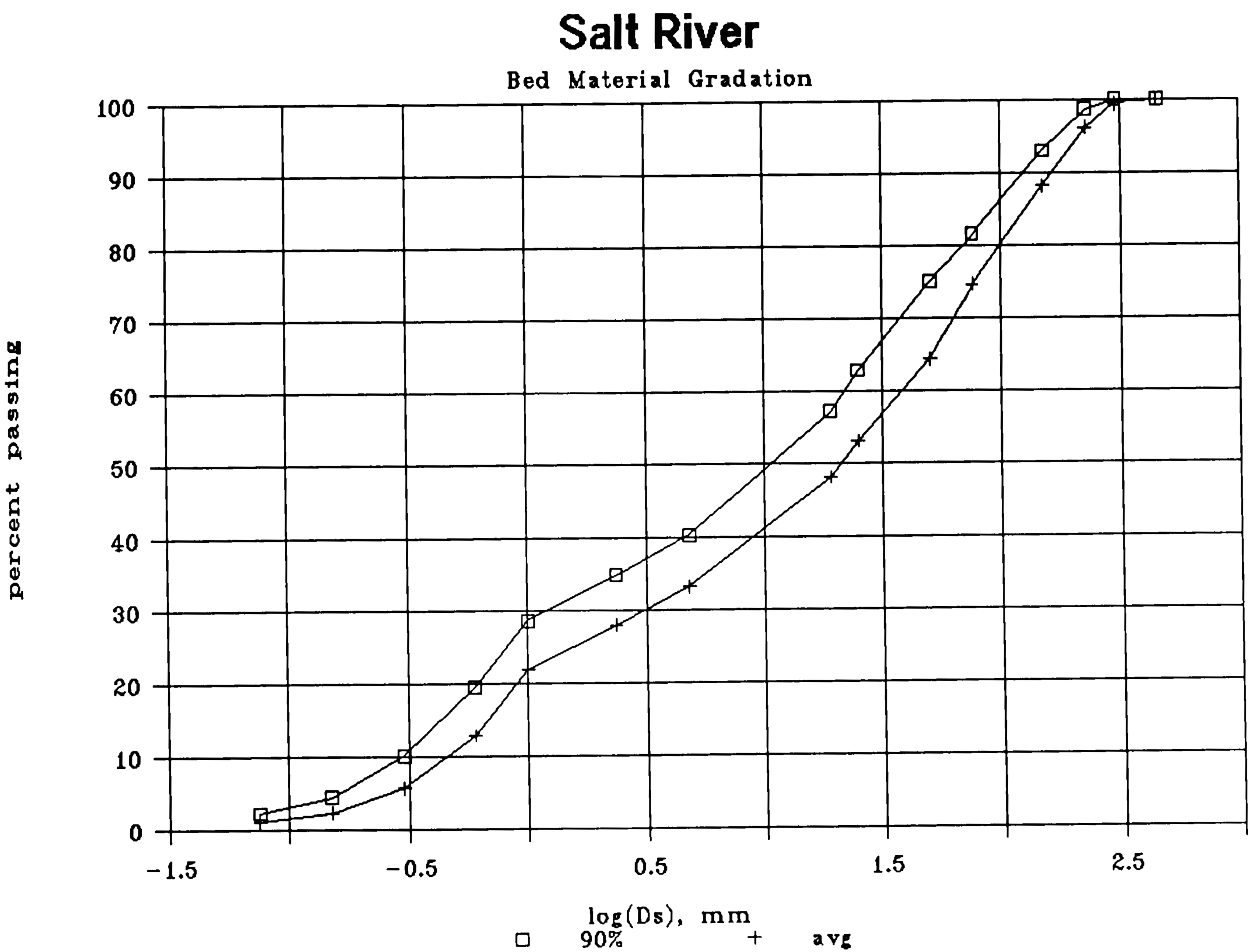
Location	Depth (Feet)	Total Weight of Sample (Pounds)	Percent Passing 3"	6"	9"	12"	18"
26	2-5	5900	81	91	98	100	-
29	2-5	4840	69	84	97	100	-
33	2-5	5720	72	86	94	98	100
39	2-5	7480	83	95	99	100	-
40	2-5	6220	70	85	94	100	-
45	2-5	10,200	73	88	95	99	100

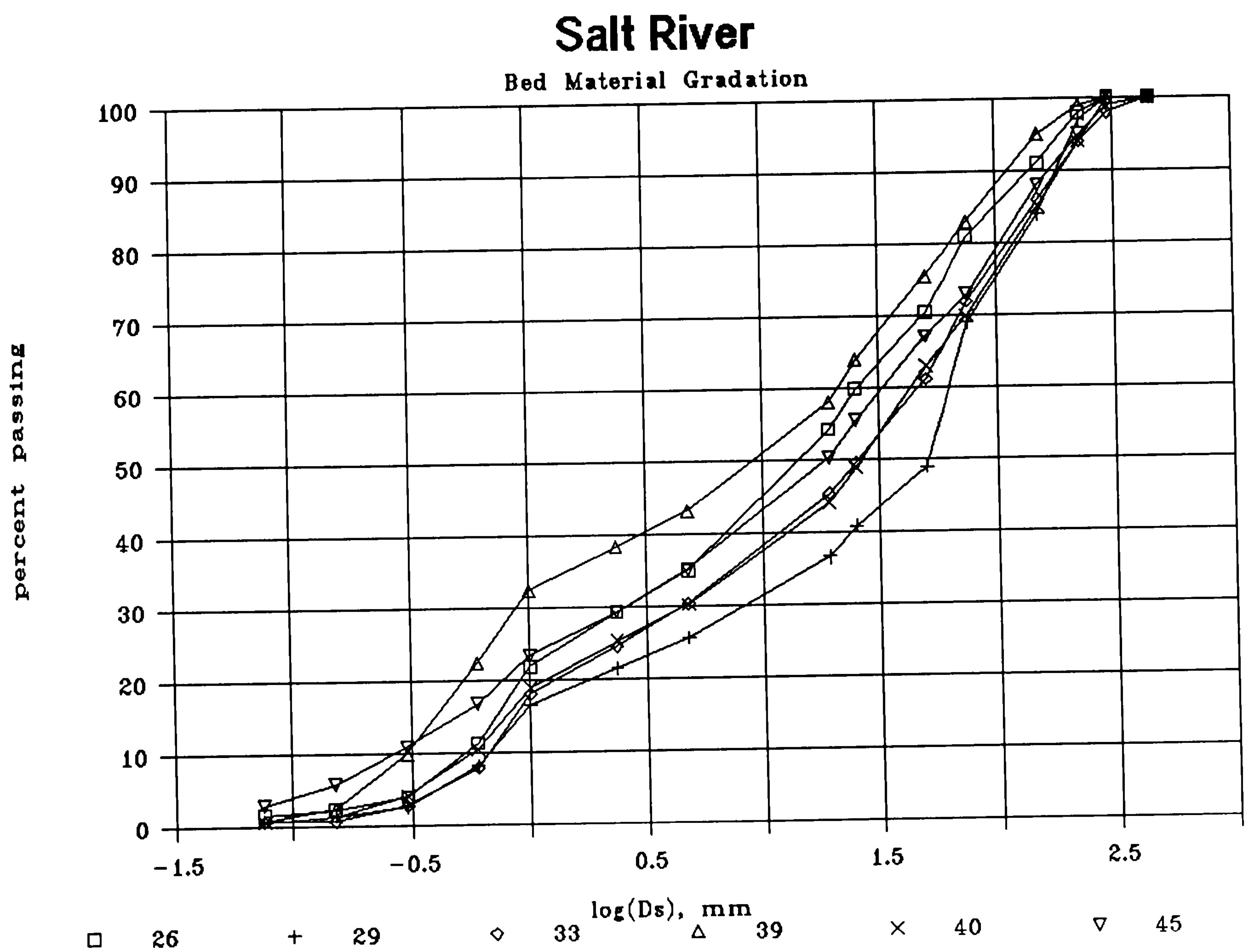
Project No. 89-0919

Thomas-Hartig & Associates, Inc.

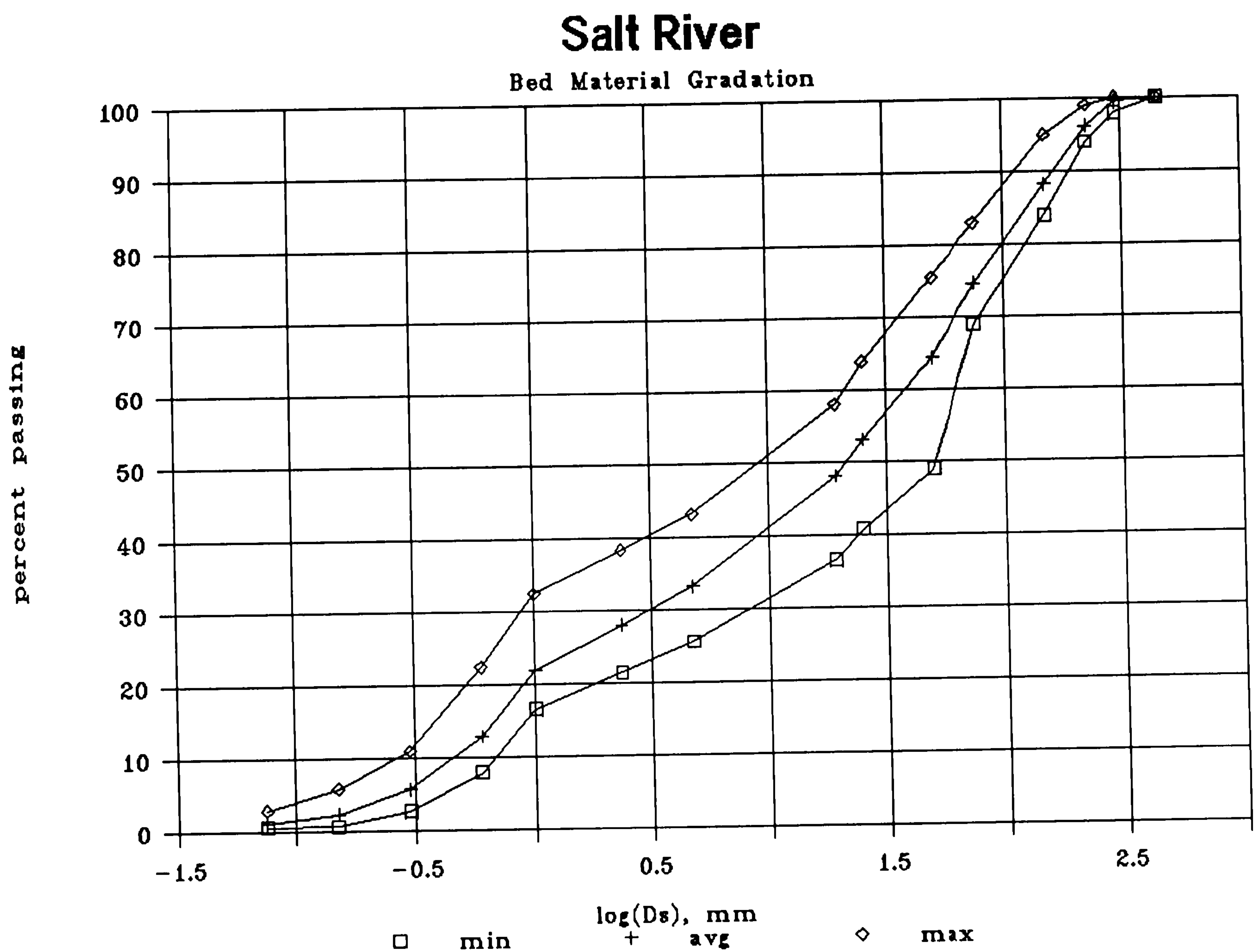
Table 4a. Sampled Bed Material Gradations

Boring Number	Location (So. Levee CL)	Sieve Size - Partical size, mm 200 0.075 -1.12	100 0.15 -0.82	50 0.3 -0.52	30 0.6 -0.22	16 1 0	8 2.36 0.372	4 4.76 0.677	3/4" 19 1.278	1" 25 1.397	2" 50 1.698	3" 75 1.875	Over 6" 150 2.176	9" 225 2.352	12" 300 2.477	18" 450 2.653
26	428+00, 350' lt	1.6	2.4	4.1	11.3	21.9	29.2	34.8	54.3	59.9	70.5	81.0	91.0	98.0	100.0	100.0
29	448+00, 525' lt	0.7	1.4	2.8	8.3	16.6	21.4	25.5	36.6	40.7	49.0	69.0	84.0	97.0	100.0	100.0
33	458+00, 450' lt	0.7	0.7	2.9	7.9	18.0	24.5	30.2	45.4	49.7	61.2	72.0	86.0	94.0	98.0	100.0
39	508+00, 700' lt	0.8	2.5	10.0	22.4	32.4	38.2	43.2	58.1	63.9	75.5	83.0	95.0	99.0	100.0	100.0
40	516+00, 650' lt	0.7	1.4	4.2	10.5	18.9	25.2	30.1	44.1	49.0	63.0	70.0	85.0	94.0	100.0	100.0
45	207+00, 0' rt	2.9	5.8	11.0	16.8	23.4	29.2	35.0	50.4	55.5	67.2	73.0	88.0	95.0	99.0	100.0
avg		1.2	2.4	5.8	12.9	21.8	27.9	33.2	48.1	53.1	64.4	74.7	88.2	96.2	99.5	100.0
min		0.7	0.7	2.8	7.9	16.6	21.4	25.5	36.6	40.7	49.0	69.0	84.0	94.0	98.0	100.0
max		2.9	5.8	11.0	22.4	32.4	38.2	43.2	58.1	63.9	75.5	83.0	95.0	99.0	100.0	100.0
sdev		0.8	1.7	3.3	5.2	5.2	5.3	5.5	7.1	7.7	8.3	5.4	3.8	2.0	0.8	0.0
90% confidence limit		2.3	4.5	10.1	19.5	28.5	34.8	40.2	57.2	62.9	75.1	81.5	93.0	98.7	100.0	100.0

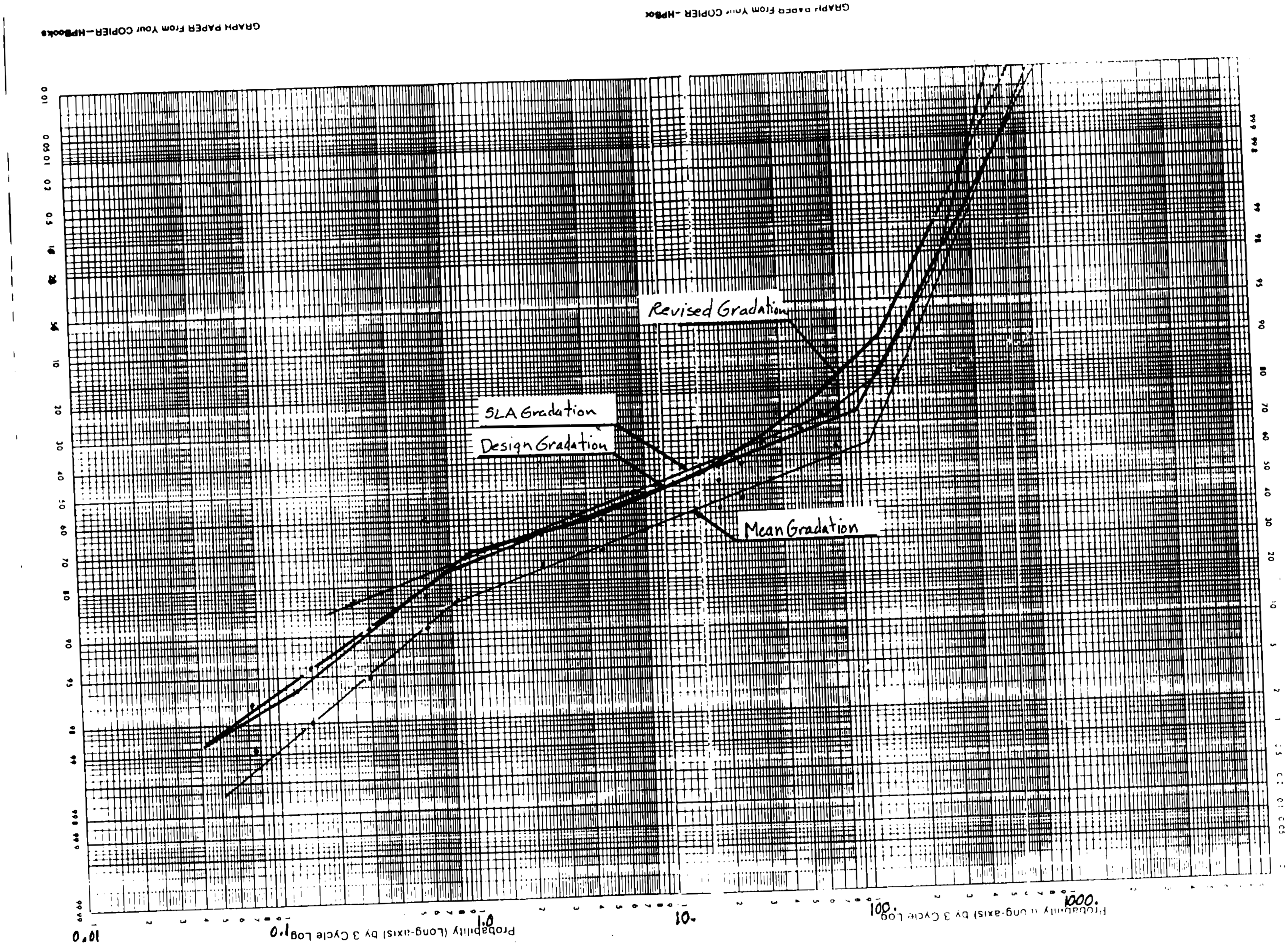

Salt River
Bed Material Gradation
percent passing
100
90
80
70
60
50
40
30
20
10
0
-1.5
-0.5
0.5
1.5
2.5
log(Ds), mm
90%
avg



Salt River
Bed Material Gradation
percent passing
100
90
80
70
60
50
40
30
20
10
0
-1.5
-0.5
0.5
1.5
2.5
log(Ds), mm
26
29
33
39
40
45



Salt River
Bed Material Gradation
percent passing
100
90
80
70
60
50
40
30
20
10
0
-1.5
-0.5
0.5
1.5
2.5
log(Ds), mm
min
avg
max


Revised Gradation
SLA Gradation
Design Gradation
Mean Gradation
0.01
0.1
1.0
10.
100.
1000.
GRAPH PAPER From Your COPIER—HPBooks
Probability (Long-axis) by 3 Cycle Log

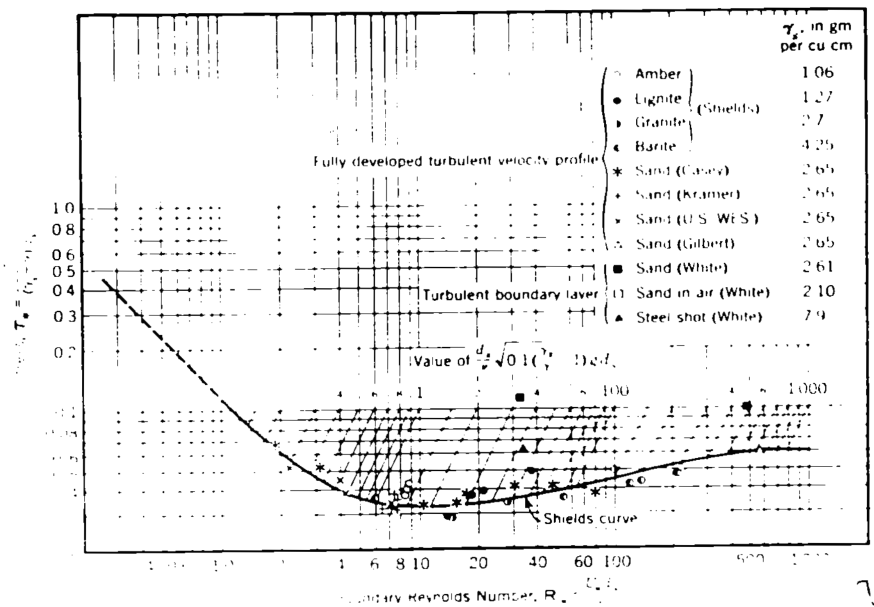
CRSS PROJECT	Rio Salado Channelization	JOB NO. R101N		NO
CLIENT	Tempe	DESIGNED BY GKC	DATE 5/16/9?	
CRS SIRRINE SUBJECT		CHECKED BY JFM	DATE 5/15/9?	OF

Maximum Bed Scour

$$y_s = 3 D_a (1/\rho_c - 1)$$

where $D_a = \tau_p / (\gamma_s - \gamma) \tau_*$

for boundary Reynolds number in excess of ≈ 500, $\tau_* = 0.060$



$\nu = 1 \times 10^{-5}\ ft^2/sec$

FIG. 2.43.—**Shields Diagram with White Data Added**

ASCE Manual No. 54 pg. 96

Therefore $D_a = \tau_p / (0.06 \times 103)$

$$= \tau_p / 6.2 \qquad (1)$$

$$\tau_p = \left(\frac{n_p}{n}\right)^2 \cdot \tau_{max} \cdot SF \qquad (2)$$

$$n_p = 0.034\, D_a^{1/6} \qquad (3)$$

where n_p is the roughness due to the partical

τ_p is the shear on the partical

CRS SIRRINE

PROJECT Rio Salado Channelization	JOB NO. RICIN		NO
CLIENT City of Tempe	DESIGNED BY GRC	DATE 5/16/9[illegible]	
SUBJECT Max Bed Scour	CHECKED BY JTM	DATE 5/[illegible]	

SF = Safety factor 1.5 for Q = [illegible]
1.3 for Q = 250,000 cfs

Combining equations 1, 2 and 3 gives

$$D_a = \left(\frac{SF \cdot \tau_{max}}{5360\ n^2}\right)^{3/2} \qquad (4)$$

(see derivation pg. 3)

Q = 215,000 cfs $\qquad D_a = 0.11\ \tau_{max}^{3/2}$

Reach	τ_{max}	D_a	P_c	y_s	R_*
GC3 - GC4	2.46	0.42	17.0*	6.2	47,300
GC4 - Mill	1.85	0.28	16.5	4.3	27,300
Mill - Rural	2.26	0.37	11.9	8.2	39,900
Rural - IBW	2.31	0.39	11.1	9.4	42,600
IBW - McCl	2.81	0.52	7.1	20.4	62,600

Q = 250,000 cfs $\qquad D_a = 0.088\ \tau_{max}^{3/2}$

Reach	τ_{max}	D_a	P_c	y_s	R_*
GC3 - GC4	2.70	0.39	18.5*	5.2	46,000
GC4 - Mill	2.02	0.25	18.4	3.3	25,500
Mill - Rural	2.21	0.29	16.0	4.6	31,000
Rural - IBW	2.38	0.32	14.3	5.8	35,400
IBW - McCl	2.87	0.43	9.6	12.1	52,300

*SLA Gradation, would be 9.9% and 11.1% for CRSS gradation giving a y_s of 11.5 and 9.4, respectively for the Q = 215K and Q = 250K discharges.

CRSS CRS SIRRINE			
PROJECT: Rio Salado Channelization	JOB NO.: R101N		NO: 3
CLIENT: City of Tempe	DESIGNED BY: GKC	DATE: 5/16/90	
SUBJECT: Max Bed Scour	CHECKED BY: JFM	DATE: 5/16/90	

Derivation of Eq. 4

Combining eq 1, 2 and 3 gives

$$D_a = \frac{(n_p/n)^2 \, \tau_{max} \, SF}{6.2}$$

$$= \frac{(0.034 \, D_a^{1/6})^2 \, \tau_{max} \, SF}{6.2 \, n^2}$$

$$D_a^{2/3} = \frac{SF \, \tau_{max}}{6.2 \, (n/0.034)^2}$$

$$= \left(\frac{SF \cdot \tau_{max}}{5360 \, n^2} \right)^{3/2} \qquad (4)$$

for $n = 0.035$ and $SF = 1.5$

equation 4 becomes

$$D_a = 0.11 \, \tau_{max}^{3/2}$$

for $n = 0.035$ and $SF = 1.3$

equation 4 becomes

$$D_a = 0.088 \, \tau_{max}^{3/2}$$

CRSS — CRS SIRRINE			
PROJECT: Rio Salado Channelization	JOB NO: 12101N		NO: 1 OF -
CLIENT: City of Tempe	DESIGNED BY: GKC	DATE: 5/16/90	
SUBJECT: Max Pred Scour	CHECKED BY: JFM	DATE: 5/10/90	

Design Gradation

Size (mm)	Percent Finer	ΔP
0.0625	2.0	
0.125	3.9	1.9
0.25	8.6	4.7
0.50	17.0	8.4
1.0	28.5	11.5
2.0	33.6	5.1
4.0	38.9	5.3
8.0	46.6	7.7
16.0	55.1	8.5
32.0	67.2	12.1
64.0	79.0	11.8
128.0	90.4	11.4
256.0	99.3	8.9
512.0	100.0	0.7

$D_{50} = 10.6$ mm

The percent coarser:

$$P_c = 100 - \left[P_L + (P_u - P_L)\left(\frac{\ln d_c - \ln d_L}{\ln d_u - \ln d_L}\right)\right]$$

where
- P is the cummulative percent finer
- L is the index for the lower bound
- U is the index for the upper bound
- c is the index for a specific size

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John C. Patton

City of Tempe
P.O. Box 5002
Tempe, Arizona 85281

11 April 1990

Attention: Howard Hargis

Project: Geotechnical Services
Rio Salado Improvements
Tempe, Arizona

Project No. 89-0919
Addendum No. 1

At the request of George Cotton of CRS Sirrine, this letter presents our responses to the following review comments from Donald J. Rerick of the Flood Control District of Maricopa County in a letter dated March 26, 1990.

1. Ground elevation at the field exploration locations.
2. Sample size used in the mechanical analysis.
3. Review of direct shear results on Samples 10 (9'-14') and 41 (0'-6').

Our response to these comments are:

1. Ground elevations at the field exploration locations are as follows:

Field Exploration Location	Ground Surface Elevation (feet)
1	1145
2	1141
3	1152
4	1159
5	1155
6	1143
7	1142
8	1145
9	1140
9A	1140

Field Exploration Location	Ground Surface Elevation (feet)
10	1139
11	1146
12	1147
13	1150
14	1150
14A	1150
15	1151
18	1151
19	1150
19A	1150
20	1138
21	1164
21A	1164
22	1169
23	1136
24	1135
25	1140
26	1137
27	1141
28	1142
29	1141
33	1147
34	1146
35	1150
36	1155
37	1151
38	1149
39	1153
40	1159
41	1157
42	1158
43	1171
44	1156
45	1164
U-1	1148
U-1A	1148
U-2	1147
U-2A	1147
U-3	1143
U-3A	1143
1 (Sup.)	1139
2 (Sup.)	1138
3 (Sup.)	1139
4 (Sup.)	1140

2. The Bulk Samples obtained in the field for mechanical analysis were approximately 1.0 to 1.2 cubic feet in size and are representative of the material smaller than 3 inches in size encountered in the sample interval. Material greater than 3 inches in size was for the most part removed from the sample prior to bagging the material. A visual estimate of the oversized material was recorded on each log. The samples were then processed and mechanical analysis performed in accordance with ASTM C136.

3. The apparent cohesion reported on the Direct Shear Test for Sample 10 (9' - 14') is probably the result of variations in the amount of coarse sand and fine gravel along the shear plane of each of the three samples tested. An increase in the percentage of this size particle along the shear plane could result in higher values from one or two of the points. The same holds true for Sample 41 (0' - 6') for the relatively clean sample. The apparent cohesion is ignored during the use of these values in engineering analysis.

This supplement shall be attached to the original report and shall become a part thereof. Please call if you have any questions or if we may be of further assistance.

Respectfully submitted,

THOMAS-HARTIG & ASSOCIATES, INC.

By: ______________________ Kenneth L. Ricker, P.E.

Reviewed by: ______________________ Glen K. Copeland, P.E

/go

Copies to: Addressee (3)

CRSS

CRS SIRRINE

Transmittal

CRSS, INC. - 4041 North Central, Suite 650, Phoenix, Arizona 85012-3306; (602) 263-5309

FLOOD CONTROL DISTRICT
RECEIVED
MAY 02 1990

CH ENG | P & PM
HYDRO

2 2 45 1.5.1
DJR 5/2

To: Mr. Donald J. Rerick
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Enclosed:
- ☐ Letter/Memo
- ☐ Specification(s)
- ☒ Plans
- ☐ Samples
- ☐ Drawing(s)
- ☐ Other

Transmitted as Checked:
- ☐ For Information
- ☐ For Approval
- ☒ For Review and Comment
- ☐ Other
- ☐ As Requested
- ☐ Approved as Submitted
- ☐ Approved as Noted
- ☐ Return for Correction
- ☐ Return to Us
- ☐ Resubmit for Approval

Copies	Date	Rev. No.	Description	Action Code
3	4/1990	DRJ NPK PPM	Rio Salado - Salt River Hydraulic Design Report	
2	4/1990	NPK DRJ	Rio Salado - Salt River Hydraulic Desing Calculations	
1	4/1990	DRJ	5-1/4" Floppy Disks - HEC-2 Input/Output Data (2 Disks)	
3	4/11/90		Response to comments on Geotechnical Investigation Report	
2	4/1990	ALL	65 percent plans	

Action Code
A. Action indicated on item transmitted
B. No action required
C. For signature and return to this office
D. For signature and forwarding as noted below under REMARKS
E. See REMARKS below

Remarks Please review the enclosed reports. In order to maintain our construction schedule together with ADOT, we are requesting that you complete your review by the first week in June. To assist you in expediting the review of these reports, I am available at any time should you need my assistance.

By: George K Cotton

Distribution:
180.06
Howard Hargis, COT

Job Rio Salado, Salt River Channelization
Location City of TEmpe
Job Number RIOIN
Date May 1, 1990
Via Hand Delivered

DATE REC'D ________ BY ________

WHITE: Original CANARY: Acknowledgement Copy (return this copy immediately) PINK: Project File GOLD: Originator

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

City of Tempe
P.O. Box 5002
Tempe, Arizona 85281

20 February 1990

Project: Geotechnical Services
Rio Salado Improvement
near Farmer Avenue to McClintock Drive
Tempe, Arizona

Project No. 89-0919

This report presents the results of the geotechnical engineering services authorized for the Rio Salado Improvements located from near Farmer Avenue to McClintock Drive, Tempe, 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.

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.

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 L. Ricker, P.E.

Reviewed by: ______________________
Glen K. Copeland, P.E.

/sb

Copies to: Addressee (3)
CRS Sirrine, Inc. (3)

Chandler: Phone (602) 961-1169, Fax (602) 940-0952 • West Phoenix Phone (602) 437-5450

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REPORT

APPENDIX A - FIELD RESULTS

APPENDIX B - LABORATORY RESULTS

Registered Professional Engineer (Civil)
CERTIFICATE NO.
KENNETH L.
RICKER
2-20-90
Date Signed
ARIZONA, U.S.A.

PART 1
REPORT

SCOPE

The proposed Rio Salado Improvements will extend from 1300 feet west of Farmer Avenue to the McClintock Drive bridge along the Salt River and from Gilbert Road south to the Salt River along Indian Bend Wash. The improvements will be accomplished in phases. The initial phase of the project will consist of channelizing the Salt River and Indian Bend Wash. This phase will include:

1. The construction of approximately 12,000 linear feet of levee on each side of the river. Slope protection will be provided on the channel side of the levee.
2. An overbank control levee on the south bank on either side of Mill Avenue.
3. A 225 feet wide, 4 to 6 foot deep low flow channel.
4. A grade control structure on Indian Bend Wash at Gilbert Road. The grade control structure will extend across the channel and will provide a stair stepped drop along the channel from the north end to the south end of the structure of 19.5 feet in four steps. The drop will occur over a distance of approximately 254 feet.
5. A grade control structure on the Salt River west of McClintock Drive. The grade control structure will extend across the channel and will provide a stair stepped drop along the channel from east to west of 5.4 feet in two steps. The drop will occur over a distance of approximately 105 feet.
6. Relocation of a 36 inch diameter storm drain parallel to and on the west side of Rural Road.

Future phases of the Rio Salado Improvements will include:

1. Numerous interconnected and individual lakes ranging in depth from 25 to 40 feet. Initially the lakes would be unlined.
2. Approximately 1-1/2 miles of new roadway along Rio Salado Parkway.
3. The extension of Miller Road from Curry Road to the Salt River.
4. An inflatable rubber dam/spillway.
5. Parks and recreation areas, ramadas, playgrounds and other related facilities.

Approximately 6.5 million cubic yards of excess materials may be generated by the various phases of the proposed improvement. The excess material may be sold to provide revenue for the improvements.

PURPOSE

The purpose of these geotechnical services is to provide subsurface information, laboratory test data and geotechnical engineering information with respect to:

1. How site conditions such as the location and extent of landfills, the depth to and location of bedrock and the depth of groundwater will effect the proposed and future improvements.

2. Foundation recommendations for the levee and grade control structures.

3. Subsurface conditions along the utility relocations at Rural Road.

4. Site preparation and fill placement criteria for levees, grade control structures, and the lakes.

5. Evaluation of leakage from unlined lakes and the effects of groundwater and bedrock condition on the lakes.

6. Recommendation for cement treated alluvium.

7. A general evaluation of the quality of the materials encountered for use in mineral aggregate production.

8. An evaluation of excavation conditions.

9. Recommendations for temporary and permanent cut and fill slopes.

SITE DESCRIPTION AND SITE HISTORY

The proposed improvements are within the existing channel and adjoining flood plain of the Salt River and extend from 1300 west of Farmer Avenue to the McClintock Road bridge in Tempe, Arizona. At the time of our field explorations the limits of the high water channel were fairly well defined but these limits are constantly being changed by filling along the banks. The high water channel area contained gravel and sand bars, gravel roads and trails, and debris and rubble areas. The channel bottom was very irregular with numerous depressions and local rises.

A review of aerial photographs on file at Landiscor and at the City of Tempe indicates that in 1955 the site area was relatively undeveloped with the high water channel considerably wider east of Mill Avenue than the present channel. Over the years the channel east of Mill Avenue has been narrowed by development and filling of the old high water channel to its present configuration. Based on a review of the available records a majority of the landfilling which has occurred over the years in the areas of the proposed improvements was accomplished with alluvial materials from the adjoining active channel and to a lesser extent from waste material off site. This waste material is primarily soil. In addition, some debris and rubble has been placed along the channel banks to limit erosion. Only two areas have been identified along the proposed levee areas where domestic landfilling has occurred. These areas are located on the north side of the Salt River east of and west of Indian Bend Wash. These two areas are owned by SRP. Based on proposed levee plans and available records from SRP, the landfill east of Indian Bend Wash may be below the proposed levee from approximately Sta 208+00 to Sta 226+00. The deepest part of this landfill was reportedly at the west end near Sta 208+00, where the original pit may have been up to 16 feet deep. Field explorations were not accomplished in this area due to restricted access by fences and private property. The other area west of Indian Bend Wash is beyond the limits of the proposed improvement in the area.

The aerial photographs also indicated that the high water channel has been mined for aggregate over the years at various locations. The majority of this mining has been refilled over the years by high water flows in the river. In the 1965 aerial photograph, a large pond was observed in the Salt River channel just south of Indian Bend Wash. This pond was missing in subsequent aerial photographs which show the Indian Bend Wash channelization.

INVESTIGATION

Test borings were drilled with a CME 55 drill rig using hollow stem auger at 25 locations along the proposed levee and storm drain relocation and test pits were excavated with a Case 780 extend-a-hoe using a 24 inch wide bucket at 28 locations along the levee, the storm drain relocation and in the channel area. The stations and off-sets of the field explorations are indicated on the boring logs. The general location of the various field explorations are shown on the attached site plan. During the field explorations the soils encountered were visually classified, the amount of plus 3 inch material estimated and representative samples of the minus 3 inch material obtained at selected depths. The results of the field explorations are presented on Appendix A "Field Results". An estimate of the amount of plus 3 inch material present in each field exploration is presented in Appendix A. Exploration Locations 16, 17, 30, 31, and 32 were planned but not accomplished due to various access restrictions.

Representative samples obtained during the field exploration were subjected to the following laboratory analyses:

Test	Sample(s)	Purpose
Sieve Analyses & Plasticity Index	Representative (43)	Classification and aggregate evaluation
Direct Shear	Recompacted Minus No. 4 Material (5)	Shear strength, foundation bearing capacity and slope configuration
Standard Proctor	Representative (4)	Compaction characteristics

The results of the laboratory testing are presented in Appendix B.

SOIL CONDITIONS

The soil profile encountered at the field exploration locations was relatively uniform. Detailed descriptions of the materials encountered are presented on the boring logs in Appendix A. The soils in the proposed levee area along the existing high water banks to the depth explored were silty to clayey sands, gravels and cobbles with various amounts of boulders. In the existing high water channel of the Salt River, the materials for the full depth explored were relatively clean sand, gravel and cobble deposits with various amounts of boulders. These deposits contained occassional lenses of sand, sand and gravel, and silty sand and a very occassional lense of sandy clay and sandy silt. In the Indian Bend Wash area, the soils in the channel bottom and slope east of the low flow channel were sandy clays and sandy silts underlain by sand, gravel and cobble deposits. In the remainder of the Indian Bend Wash area, the soils encountered for the full depth of exploration were the sand, gravel and cobbles deposits. It is anticipated that the alluvial deposits extend to at least 100 feet below the existing channel in the area east of Tempe Butte. It is anticipated that the alluvial soils in the remainder of the improvement areas is underlain by relatively shallow bedrock. Along the existing Mill Avenue bridge, bedrock was encountered during bridge construction at depths ranging from 10 to 38 feet with a majority of the area in the 15 to 25 foot deep range. The depth to bedrock east and west of this location will vary somewhat from this depth range.

Groundwater in and adjacent to the river channel will vary considerably with location, releases from upstream dams and discharges from local runoff from Indian Bend Wash and drainage facilities at and upstream from the improvements. Groundwater was encountered at Field Exploration Locations 10, 20, 23 and 24 at depths ranging from 12 to 14 feet below existing grade at the time of our field exploration. These field explorations are located at the west end of the improvement area. No groundwater was encountered in the remainder of the field explorations accomplished for this project. Groundwater was recently encountered at a depth of 65 feet in the grade control structure area at Indian Bend Wash during preliminary field explorations for the East Papago Freeway. In the area west of Mill Avenue the shallow bedrock is probably acting like a subsurface dam to the shallow groundwater flowing in the alluvial soil below the Salt River channel which is causing the depth to groundwater in the channel area to be relatively shallow. It is anticipated that groundwater in this area will not be substantially lower even during very long dry periods.

DISCUSSION AND RECOMMENDATIONS

General: Geotechnical engineering recommendations for development of the levee, storm drain relocation, grade control structure and future phases of the proposed Rio Salado improvements are presented in the following sections. These recommendations are based upon the results of the field and laboratory testing which are presented in Appendix A and B of this report.

Levee Development: In order to provide permanent channelization and bank protection of the Salt River and a short portion of Indian Bend Wash in the Rio Salado improvements project area, approixmately 12,000 linear feet of levee will be developed on each side of the Salt River. The levee section heights will vary along different reaches of the channel improvements.

The levee on the north bank from Sta 4+80 to 44+80 will consist of a relatively low (16 foot high) fill embankment levee with 3 to 1 (horizontal to vertical) (H to V) side slopes, a 15 foot wide crest and rock rip-rap slope protection on the channel side. A level area will extend up to 120 feet horizontally from the slope protection to the crest of the new channel. The width of the horizontal bench will taper to zero at both the east and west end of the reach. At the crest the channel slope will extend down to the channel bottom at a 3 to 1 (H to V) slope. Bank protection will extend down from the channel crest at 1.5 to 1 (H to V) to the channel thalweg. The channel protection which will be buried by the channel slope will consist of an 8 foot wide cement stabilized alluvium.

From Sta 44+80 to 101+16 and Sta 301+00 to 307+73 (the north bank improvement on either side of the Indian Bend Wash Channel) the levee will consist of a relatively low (20 foot high) fill embankment levee with 3 to 1 (H to V) side slopes, a 15 foot wide crest and rock rip-rap slope protection on the channel side. The rock rip-rap will extend down to the existing ground where it will terminate on the top of the cement stabilized alluvium bank protection. The 3 to 1 (H to V) channel slope will extend down from this point to the channel bottom. The cement stabilized alluvium bank protection will be buried below the channel slope and will extend down at 1.5 to 1 (H to V) to the channel thalweg.

From Sta 200+00 to 220+11 (the north bank improvement from Indian Bend Wash to McClintock) the levee will consist of a relatively low (20 foot high) fill embankment levee with a 3 to 1 (H to V) outside slope, a 15 foot wide crest and 1.5 to 1 (H to V) channel side. The channel side slope will extend down to the existing ground surface and will have cement stabilized alluvium bank protection. At this point a 24 foot wide horizontal bench extends from the upper bank protection to the crest of the lower channel slope. Erosion protection on this bench is provided by rock rip-rap. The channel slope extends down from the channel crest to the channel bottom at 3 to 1 (H to V) slope. The bank protection for the channel is provided by a buried cement stabilized alluvium at a 1.5 to 1 (H to V) slope which extends down to the channel thalweg.

Along the south bank of the channel the levee from Sta 400+00 to 435+12 will consist of 3 to 1 (H to V) channel side slopes from the crest of the embankment down to the channel bottom, a 15 foot wide crest and 3 to 1 (H to V) side slope on the outside. The upper portion of the levee will be fill. Channel bank protection will consist of buried cement stabilized alluvium extends from the crest of the levee and channel down at 1.5 to 1 (H to V) to the channel thalweg.

From Sta 435+12 to 469+30, the south bank levee will be of similiar design to the north bank levee from Sta 200+00 to 220+11 except the horizontal bench between the upper and lower slope protection will be a maximum of 80 feet wide and does not contain rip-rap.

From Sta 469+30 to 522+30, the south bank levee will be of similar design to the north bank levee from Sta 44+80 to 101+16 and Sta 301+00 to 307+73. Except the rip-rap bank protection will be a 15 foot high fill embankment instead of a 20 foot high fill embankment. The overbank levee will consist of a relatively low fill embankment with 3 to 1 (H to V) side slope and rip-rap slope protection on the channel side.

The crest of the levee will be at approximately Elevation 1174 feet at the east end of the project and at approximately Elevation 1158 feet at the west end. The channel thalweg will be at approximately Elevation 1142 feet at the east end and at approximately Elevation 1132 feet at the west end.

The embankment fills, and the channel slopes will be constructed with materials obtained from the channel improvement area. We recommend that the levee embankment fills be constructed with the sand, gravel and cobble soils which contain no to some silty and clayey fines. The channel slope materials which cover the buried bank protection can be constructed of any of the materials encountered within the project area which are free of organic material, garbage, debris and rubble.

Slope stability analysis of the proposed embankment configurations described above were performed. The following parameters were used in the evaluation.

1. Embankment fill and native alluvium
   Ø = 41 degree
   C = 0 psf
   Inplace density = 135 pcf
   Submerged = 72 pcf
2. Materials at submerged conditions
3. Rapid Draw-down

These embankment configurations (as described earlier) have results in calculated factors of safety for the various conditions and configuration in the range of 1.4 to 4.3. The analyses was conservative in that the strength of bank protection was ignored. Therefore, it is our opinion that embankments may be satisfactorily constructed as planned.

<u>Levee Contruction</u>: The proposed levee development will include the construction of various heights and widths of embankment fill. Foundation preparation in the embankment fill area should include, as a minimum, the complete removal of all sandy clay, sandy silt, sand and silty sand soils and debris and rubble laden materials. The landfill which is known to exist in the area of Sta 208+00 to 226+00 should be removed from below the levee area. After removal of the various materials, the foundation area should be scarified to a minimum depth of 8 inches and compacted and, the embankment fill material placed and compacted in horizontal lifts. Scarified soil and embankment fill materials should be compacted to at least 98 percent of the maximum dry density as determined by ASTM D698 at a moisture content in the range of 3 percent below to 3 percent above optimum.

Cement Stabilized Alluvium: Erosion bank protection along the proposed levee will include rip-rap placed on 3 to 1 (H to V) slopes and cement stabilized alluvium placed on a 1.5 to 1 (H to V) slope. The cement stabilized alluvium will be at least 8 feet wide (horizontally) and extend 4 to 28 feet below exsiting grade down to the channel thalweg. Depending on groundwater conditions at the time of construction, dewatering of this bank protection excavation area may be required. The design of the dewatering system should be accomplished by the contractor and approved by the project designer. During the design of the dewatering system, the effect of seepage on slope stability should be considered.

A preliminary mix design for cement stabilized alluvium is presently being accomplished and will be submitted to you upon completion. In general, the cement stabilized alluvium will consist of the clean sand, gravel and cobble deposit which have been processed to the following requirements.

Sieve Size	Percent Passing
3"	100
#4	40 - 60
#200	0 - 8

The aggregate should be blended with cement/fly ash and water in a mixing plant and transported to the construction area. The material should be uniformly spread so that compacted horizontial lift thickness do not exceed 9 inches. The materials should be compacted to at least 98 percent of the maximum dry density as determined by ASTM D698. All exposed portions of the compacted material should be kept moist for at least 7 days. Compaction of the cement stabilized alluvium should be completed within one hour after the water is added to the mix. The surface of the compacted materials which has not been worked for more than two hours should be scarified to a depth of 1 inch, and the loose material removed prior to placing additional materials.

Grade Control Structure: The two grade control structures planned for the project will be a four step structure at Indian Bend Wash and a two step structure on the Salt River at McClintock Drive. Each of the steps of the structures will drop the channel grade and will consist of a soil cement or cement stabilized alluvium dike section with an 8 foot wide crest; 1.5 to 1 (H to V) downstream slope and either 1.5 to 1 (H to V) or 0.75 to 1 (H to V) upstream slope which from the risers of the step. The step runners between the dike will be inlaid with precast reinforced concrete panels laid on end at a 0.75 to 1 (H to V) slope pointing downstream. The panels will be set on a 12 inch thick sand layer.

In order to provide uniform support of the grade control structure especially at the Indian Bend Wash site, we recommend that foundation preparation below the entire structure include:

1. The complete removal of sandy clay and sandy silt deposits.

2. All areas which have been over-excavated and which require the placement of fill should be scarified to a minimum depth of 8 inches and compacted.

3. All fill materials required below the dikes and sand bedding areas should be placed in horizontal lifts, moisture conditioned to a uniform moisture content in the range of 3 percent below to 3 percent above optimum and compacted to at least 98 percent of the maximum dry density as determined in accordance with ASTM D698.

4. Granular soils obtained during excavation of the grade control structure or from other parts of the channelization project may be used as fill materials provided these materials are free of organic matter, debris, rubble and garbage.

Storm Drain Relocation A 36 inch diameter storm drain will be relocated parallel to and on the west side of Rural Road. The storm drain will be founded 22 to 28 feet below existing grade. Three test borings (U-1, U-2 and U-3) supplemented by three test pits (U-1A, U-2A and U-3A) were accomplished along this alignment. In addition, the test borings for the Rural Road bridge accomplished by others were reviewed. The results of these field explorations indicate that the materials for the full depth of utility line excavation will consist of sand, gravel and cobbles containing no to some silty and clayey fines. Groundwater was not encountered at the time of our field exploration. However, fluctuations in the groundwater table will occur and either dewatering or working below groundwater levels may be required at the time of construction.

Excavation Conditions: The field exploration and sampling at the site was performed for design purposes. It is not possible to accurately correlate results of the various methods of field explorations 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 designer's information with the understanding that they are approximations based only on field exploration data. More accurate information regarding excavatability should be evaluated by contractors or other interested parties from test excavations using the intended equipment.

Excavations into the site soils should be possible with conventional excavating equipment. Due to the granular nature of these soils, the presence of relatively clean sand layers, possible shallow groundwater and the presence of cobble and boulder sized material excavations may be slow and difficult to accomplish. Excavations into the underlying, hard to very hard, bedrock west of the east end of Tempe Butte may be difficult and require the assistance of specialized equipment and/or blasting.

Temporary Construction Slopes: Temporary slopes required for the construction of various aspects of the project will be dependent on the materials encountered, groundwater conditions, seepage conditions and the location, type, extent and weight of surcharge loads. In general, the following temporary slopes may be used in design but flatter or steeper slopes may be accomplished or required in the field as dictated by specific conditions.

Material	*Temporary Slope Configuration (Horizontal to Vertical)
Sand, gravel & cobbles	1 to 1
Sand, silty sand, sandy silt	1.5 to 1
Sandy Clay	0.5 to 1

*These slopes are for soils at relatively low water contents not subjected to seepage forces or submerged. Flatter slopes will be required for these conditions.

Excess Material Usage: During the various development phases of the proposed improvements, it is anticipated that 6.5 million cubic yards of excess material may be generated. Based on the field exploration performed to date, a majority of the materials will be granular alluvial soils. These same materials have been mined in this area over the last 30 to 40 years. These materials may be processed to make mineral aggregate for use in portland cement concrete and asphalt concrete. The material may also be used as fill below buildings, parking areas and other developments.

Foundations for Future Development: Future phases of the improvement will probably include permanent structures within the channel area. These structures will be designed to withstand certain stream flow loadings and should be supported on foundation elements which extend below potential scour depths. Spread and continuous footings founded below these scour depths on the compacted fill or undisturbed channel materials at shallow depths should be appropriate for support of lightly loaded building (allowable bearing pressures on the order of 1500 to

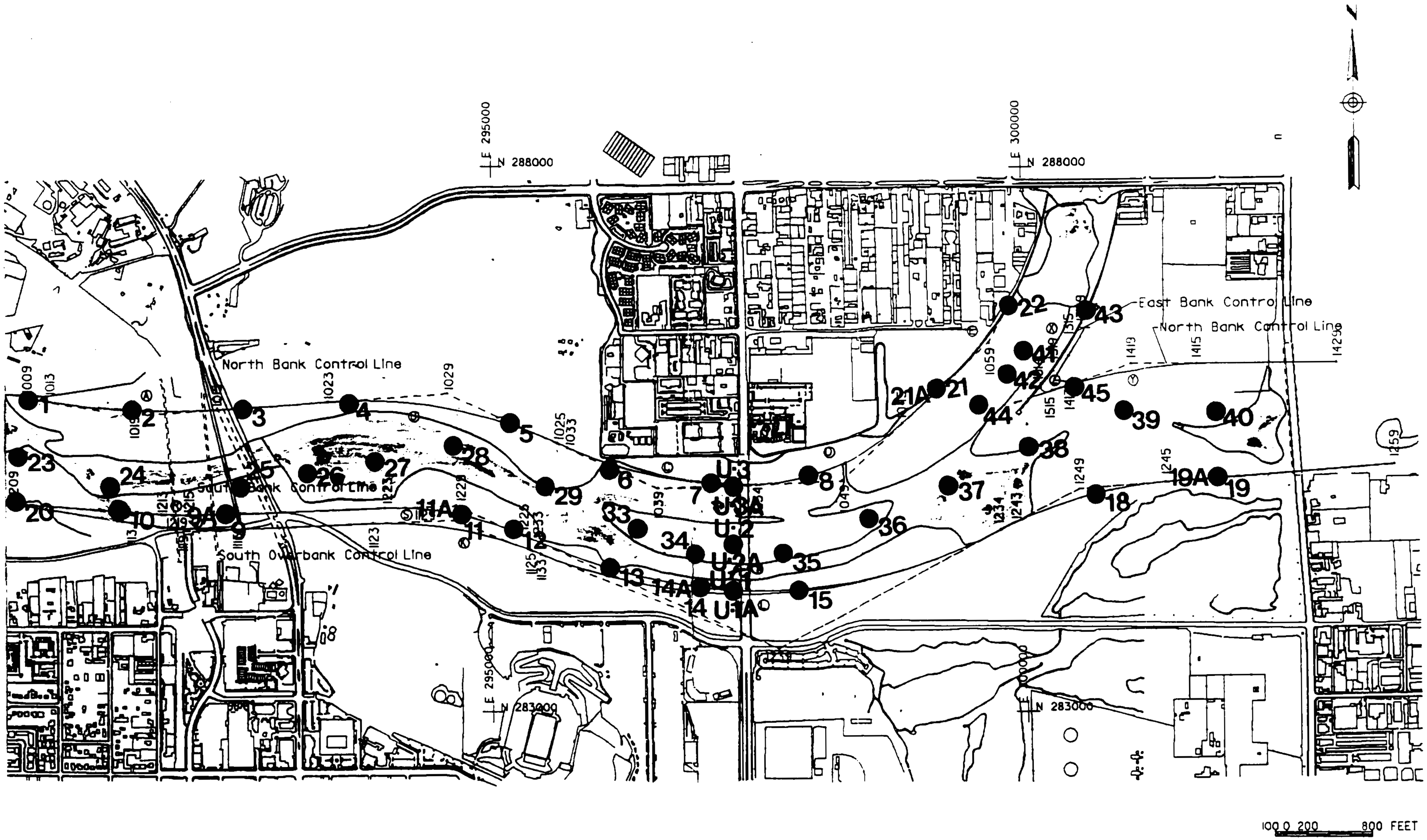
2500 psf). The settlement potential of these footings will be moderate under the existing low moisture conditions. Some additional settlement may occur if the bearing soils experience an increase in moisture content. Deeper spread and continuous footings or drilled piers founded on the dense natural sand, gravel, and cobble deposit at moderate depths should be appropriate for support of heavier structures (allowable bearing pressures on the order of 12,000 to 20,000 psf).

Lakes: Preliminary conceptual plans of the Rio Salado areas includes several interconnected and individual lakes. These lakes may be up to 40 feet deep. A majority of the future lake excavations will encounter the relatively clean sand, gravel and cobble deposits which have relatively high seepage characteristics. These seepage characteristics will be reduced over time by siltation but in general will not be reduced to a magnitude of losses which is generally considered acceptable. In addition, a shallow groundwater table exists at the west end of the project which will make lake construction more difficult. Water levels in these lakes would reflect the level and flucuations of the groundwater table. Shallow bedrock also exists in the channel area west of the east end of Tempe Butte. The shallow bedrock, if encountered, will reduce the seepage losses through the bottom; however, it may be very difficult to excavate the lakes in this area.

Since limiting the amount of water lost by seepage is a main criterion for most lake construction, the proposed future lakes may require the installation of some type of liner. The most common types of liners used in the Phoenix area are compacted clay soil, compacted soil with a permeability control emulsion added, cement stabilized alluvium, soil cement, and polyvinyl chloride (PVC) membrane.

The type of liner selected would be dependent upon the initial cost, maintenance cost and adaptability to the variable groundwater conditions and granular alluvial soils present. For example, a clay liner would require importation of clay; a permeability control emulsion does not work in granular soils; and the other liner types have high initial costs.

APPENDIX A
FIELD RESULTS



Field Exploration Location

Site Plan

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

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	

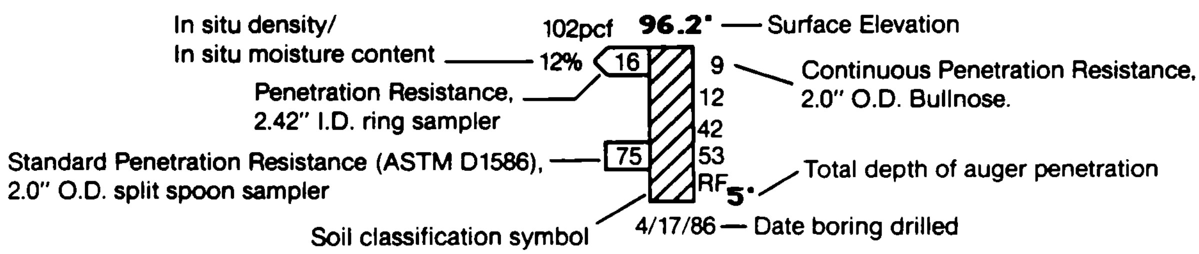
FINE-GRAINED SOIL

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		
(finer than 200)	200–40	40–10	10–4	4–3/4"	3/4"–3"	3"–12"	over 12"

MOISTURE CONDITION (INCREASING MOISTURE →)

DRY	SLIGHTLY DAMP	DAMP	MOIST	VERY MOIST	WET (SATURATED)
		(Plastic Limit)			(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. 89-0919

LEGEND OF SOIL TYPES

<u>SAND, GRAVEL AND COBBLES</u> (SM-GM); brown; dense to very dense; slightly damp; non-plastic; contains trace to some silt; 25 to 35 percent cobbles; some boulders

<u>SAND, GRAVEL AND COBBLES</u> (SP-GP); brown; dense to very dense; slightly damp; contains occasional layers or lenses of sand; 25 to 35 percent cobbles; some boulders.

1 <u>SILTY SAND</u> (SM); brown; dense; slightly damp; non-plastic; trace to some gravel and cobbles

<u>CLAYEY SAND WITH GRAVEL</u> (SC); brown; dense; slightly damp; low to medium plasticity; trace to some cobbles

<u>SAND AND GRAVEL</u> (GP-SP); brown; medium dense; slightly damp; non-plastic; trace silt

2 <u>SILTY SAND WITH GRAVEL</u> (SM); brown; dense; slightly damp; non-plastic

<u>SAND AND GRAVEL</u> (GC); brown; dense; slightly damp; some clay; medium plasticity; 25 to 35 percent cobbles; some boulders

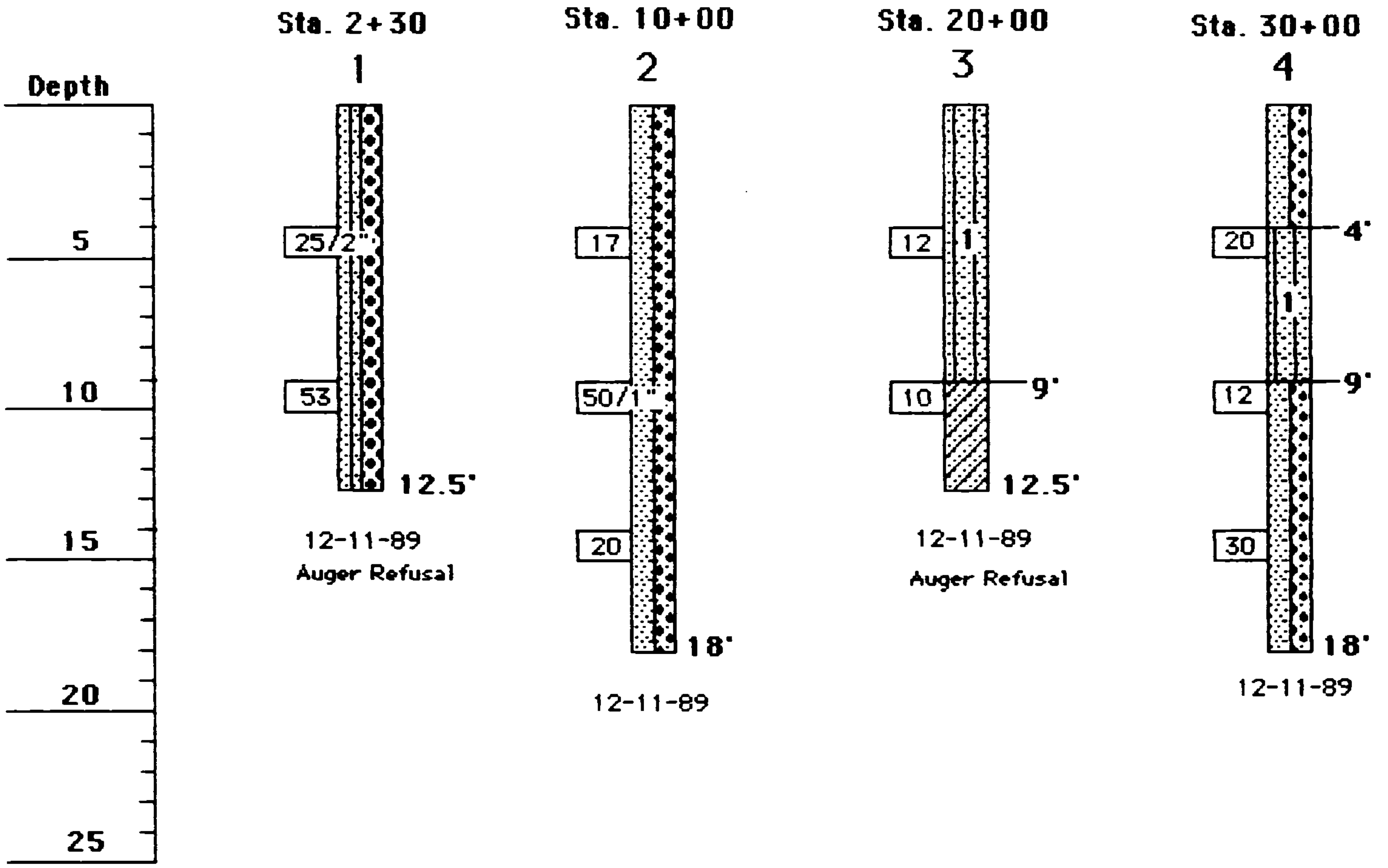
<u>SANDY SILT</u> (ML); brown; stiff; slightly damp; non-plastic

<u>SANDY CLAY</u> (CL); brown; stiff; slightly damp; low to medium plasticity

<u>SAND</u> (SP); brown; medium dense; slightly damp; trace to some gravel

<u>SAND WITH SILT</u> (SP-SM); brown; medium dense; slightly damp; non-plastic

Project No. 89-0919
Thomas-Hartig & Associates, Inc.



No free groundwater was encountered in any of the field exploration at the time of our field work unless noted otherwise on the boring logs.

All field explorations drilled with a CME 55 drill rig using a 7" diameter hollow stem auger unless otherwise noted.

Project No. 89-0919

Thomas - Hartig & Associates

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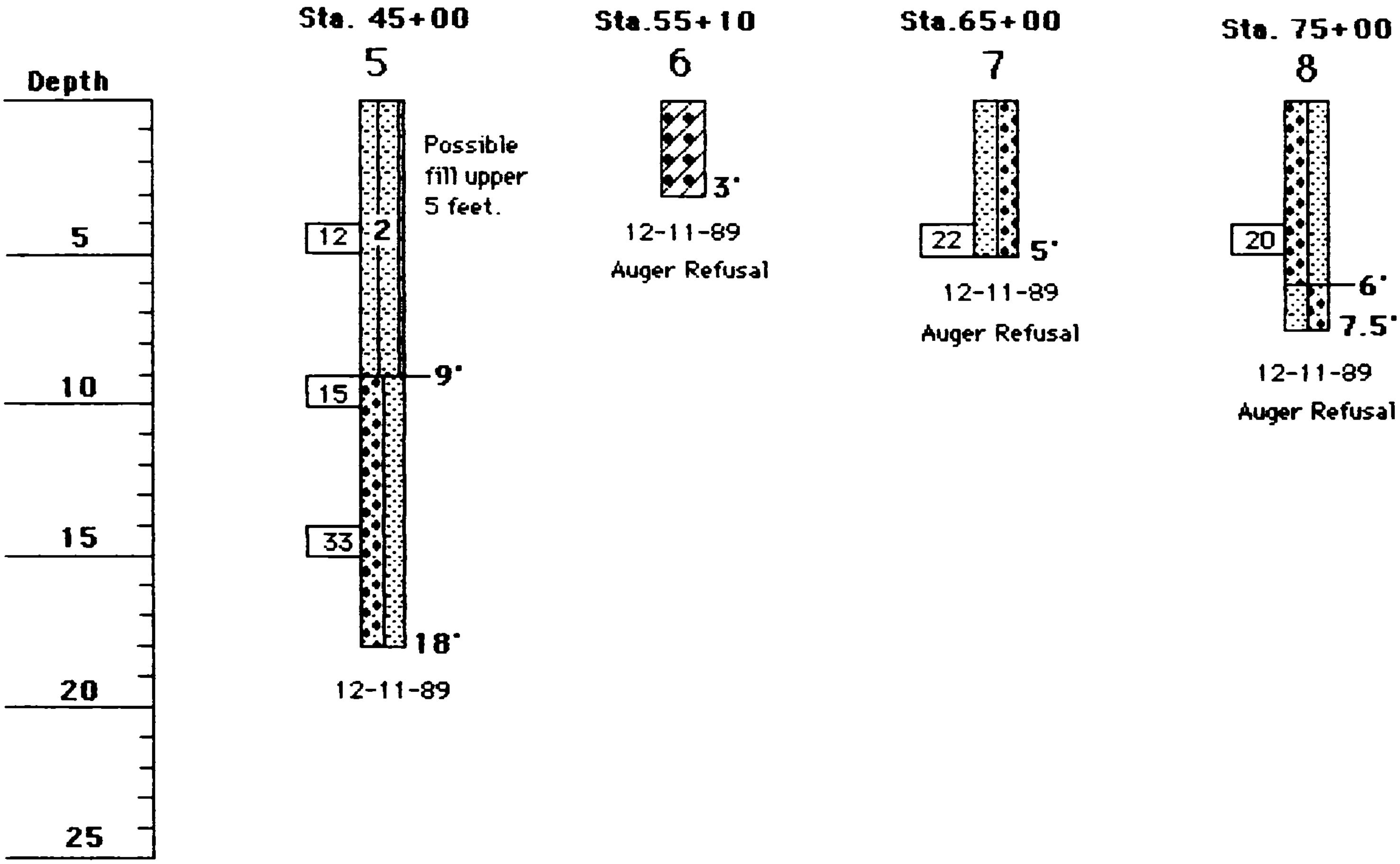

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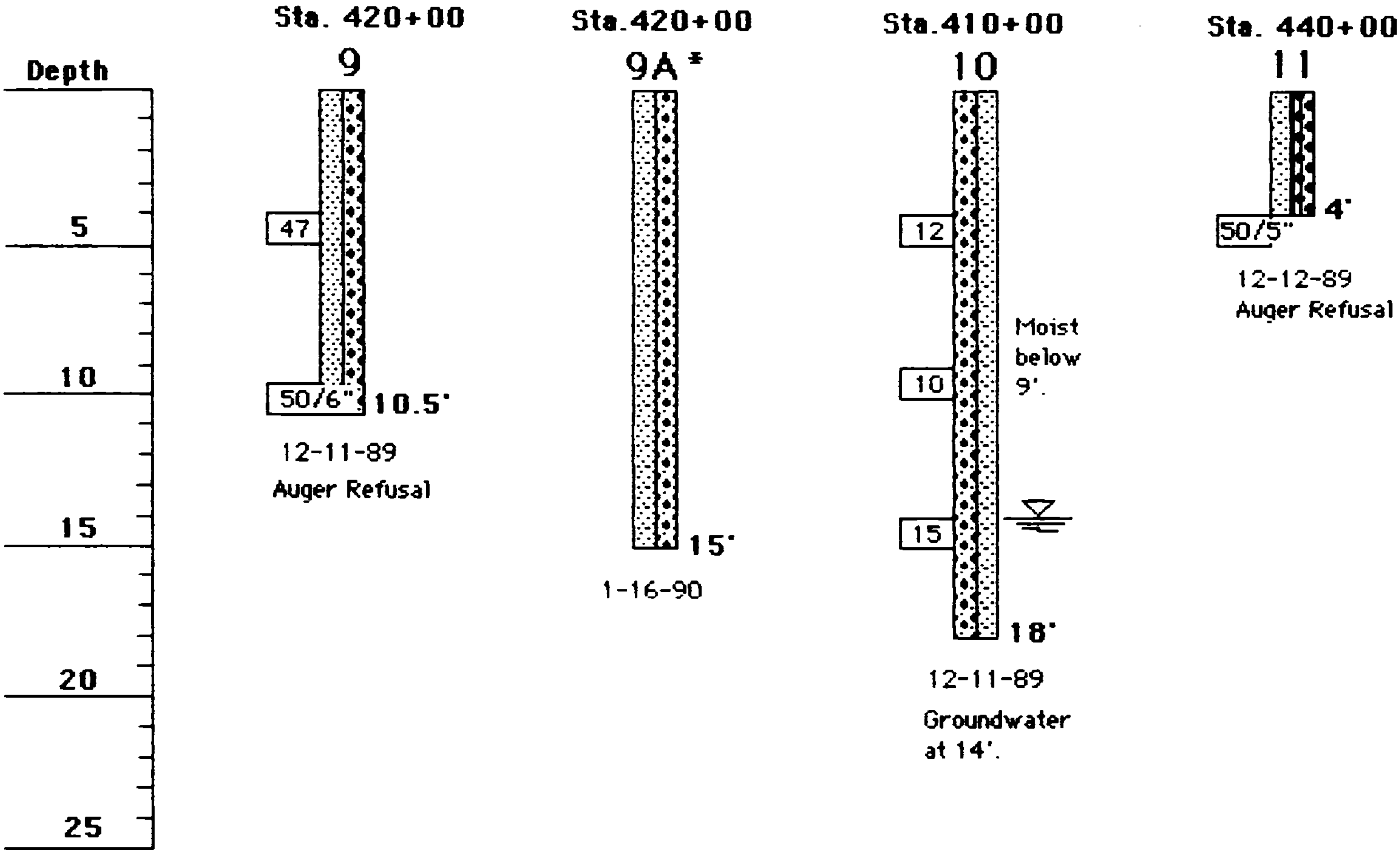

*Field exploration excavated with a Case 780 extend-a-hoe backhoe using a 24 inch wide bucket.

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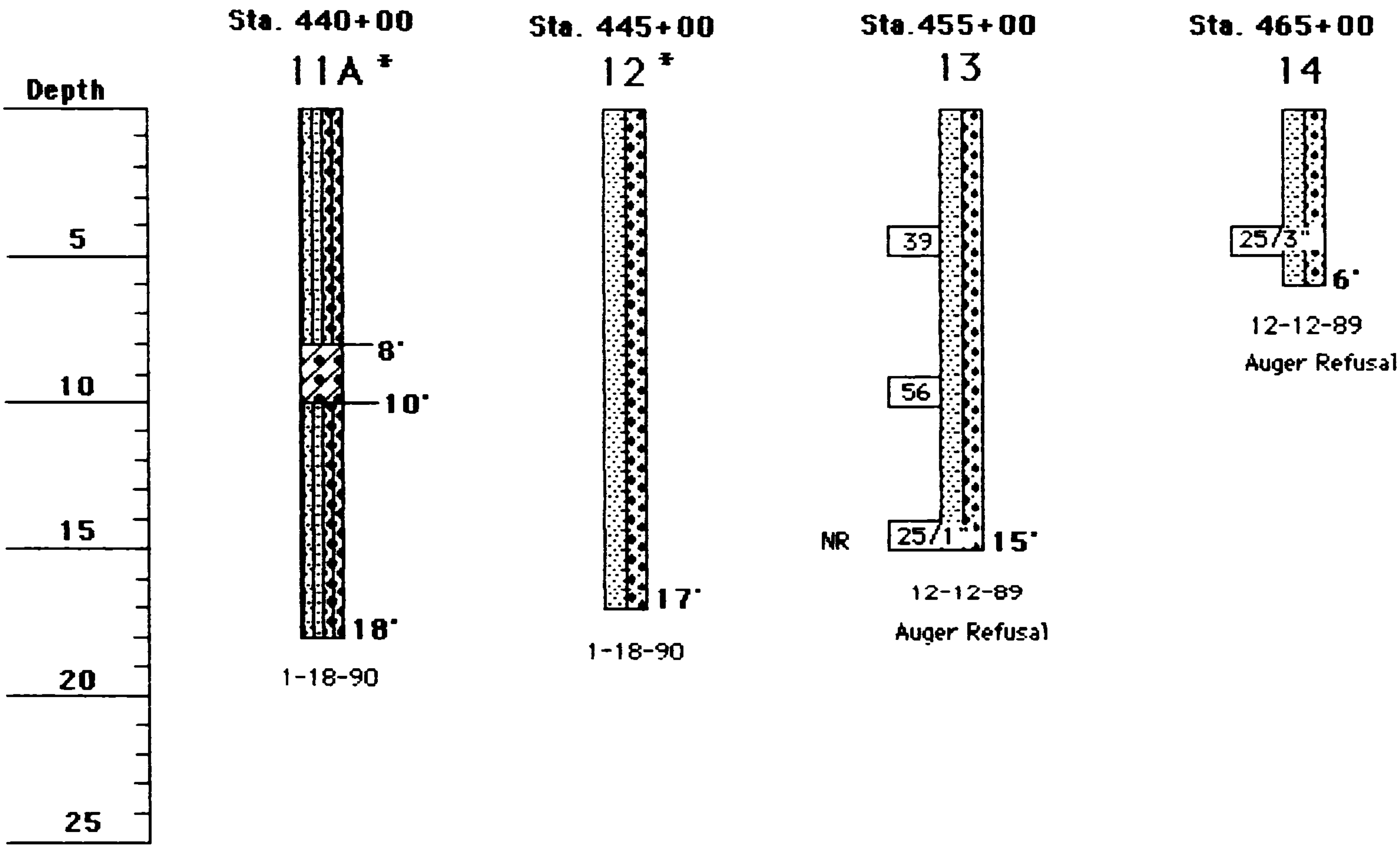

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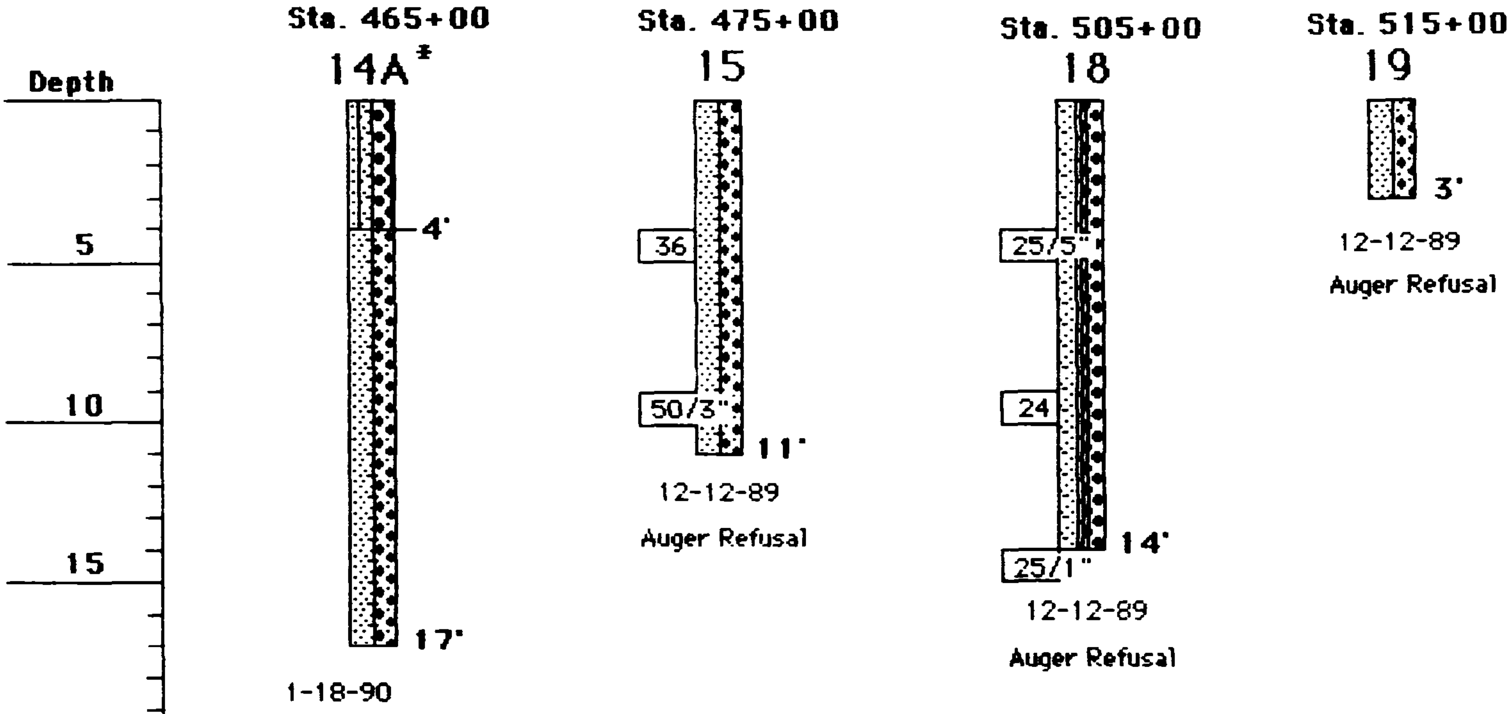

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Note: Field exploration 16 and 17 not accomplished due to restricted site access.

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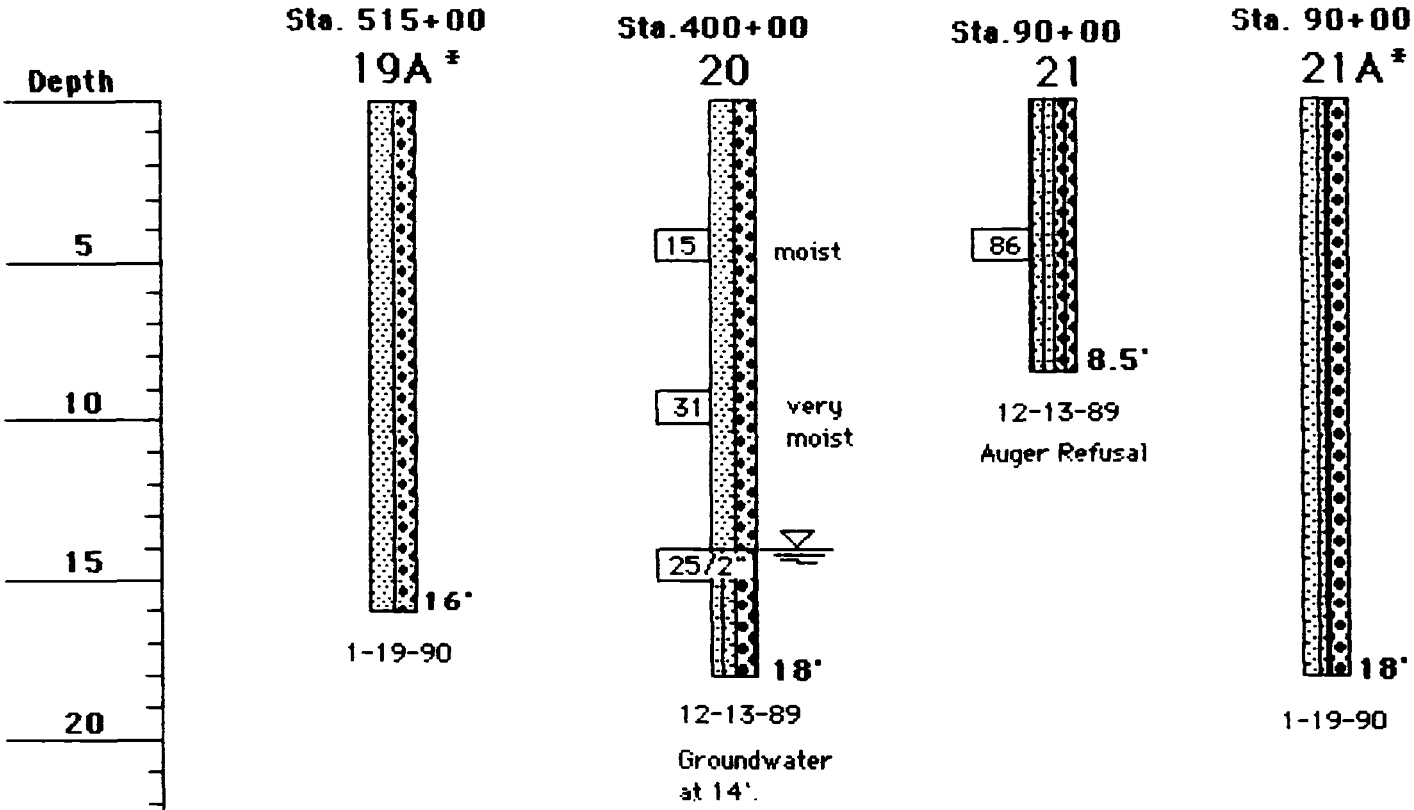

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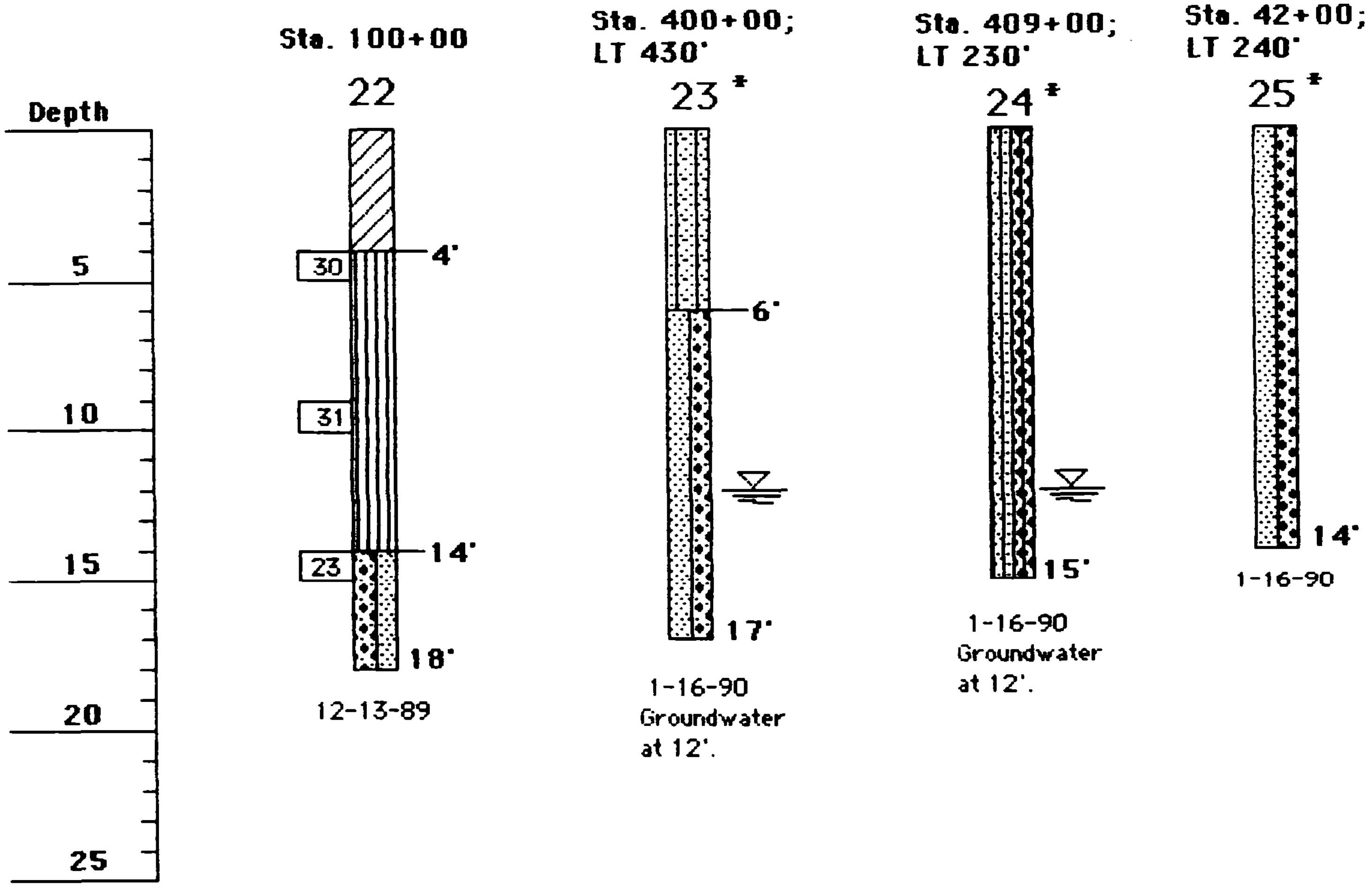

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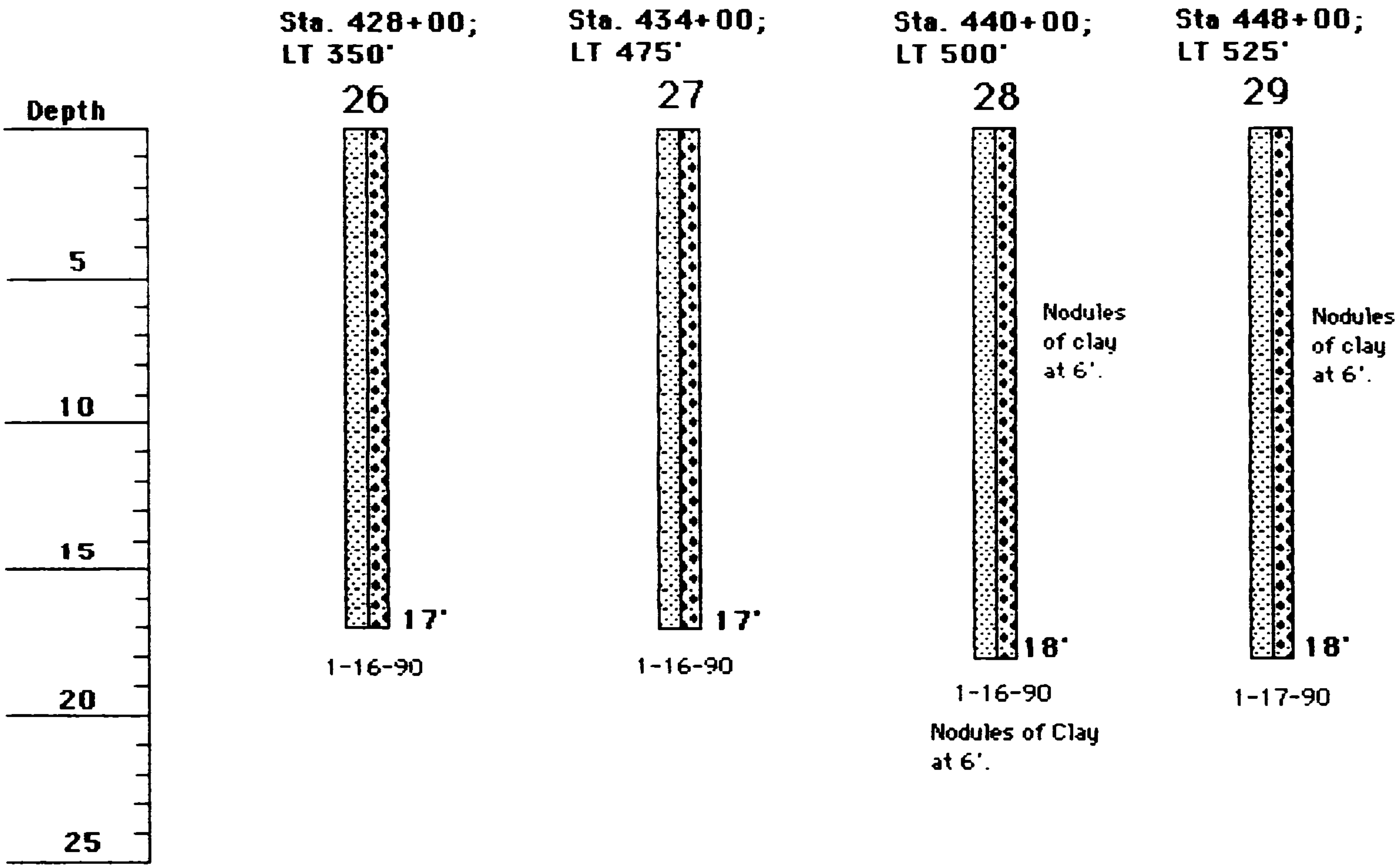

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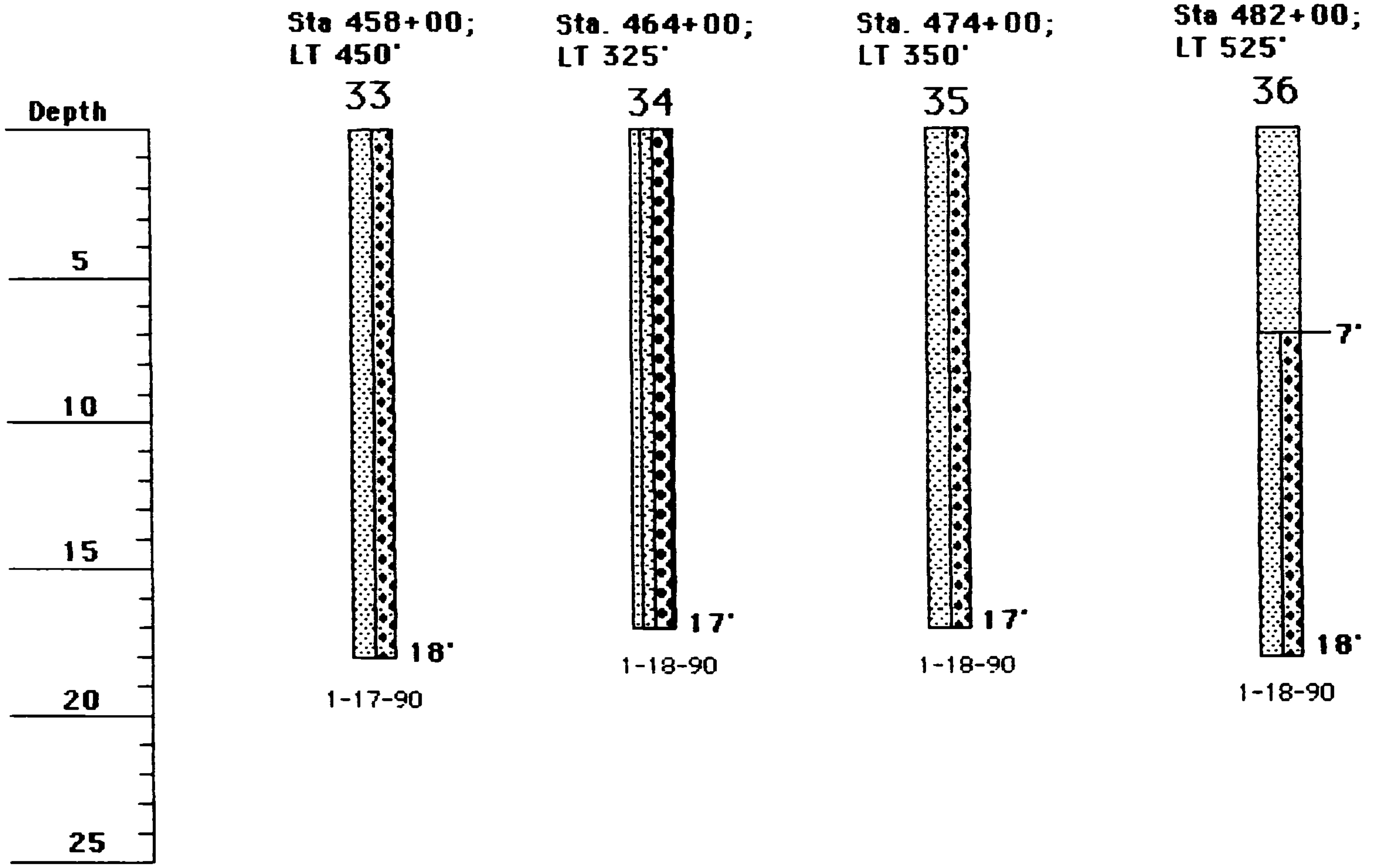

Note: Field explorations 30, 31 and 32 not accomplished due to restricted access.

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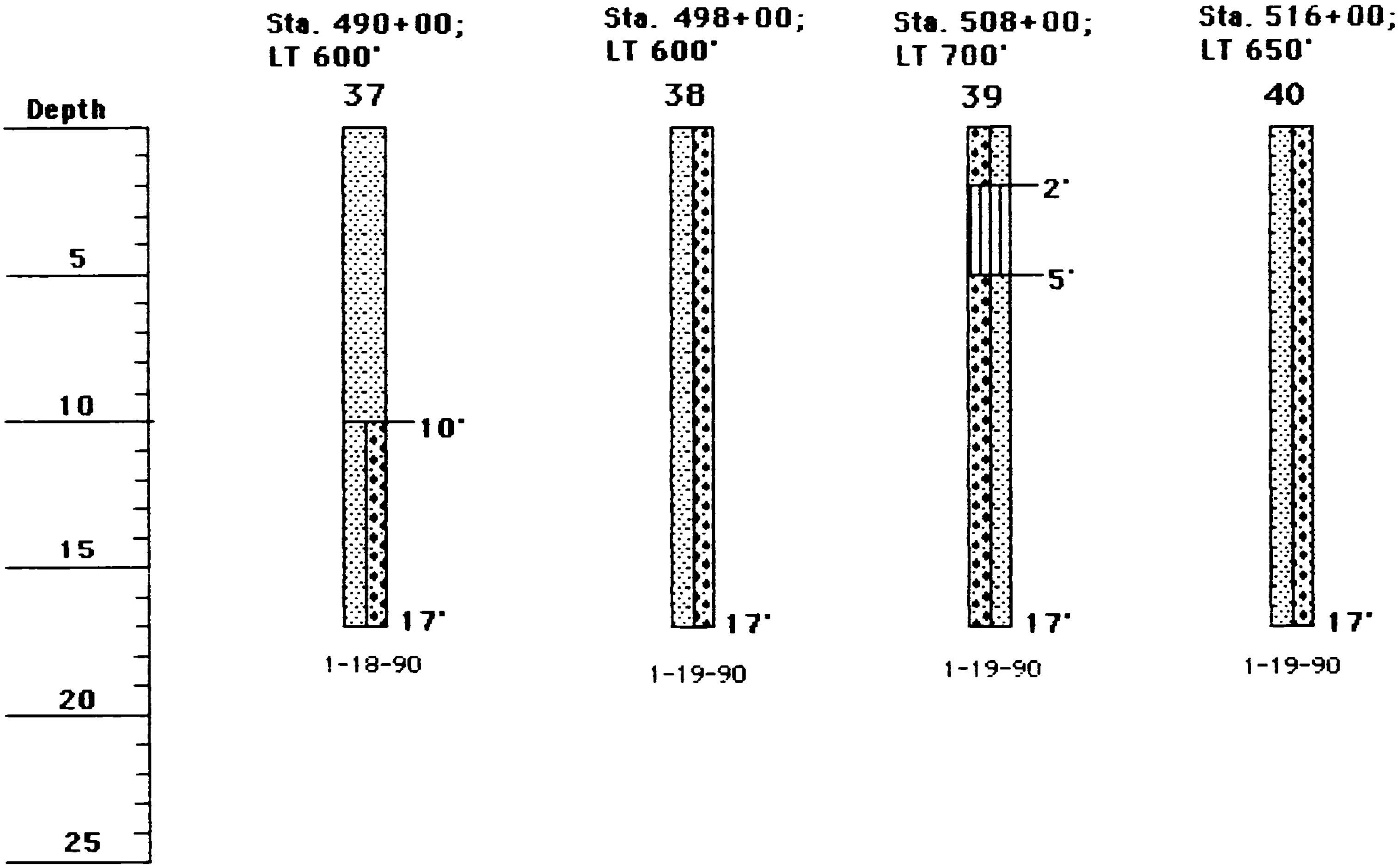

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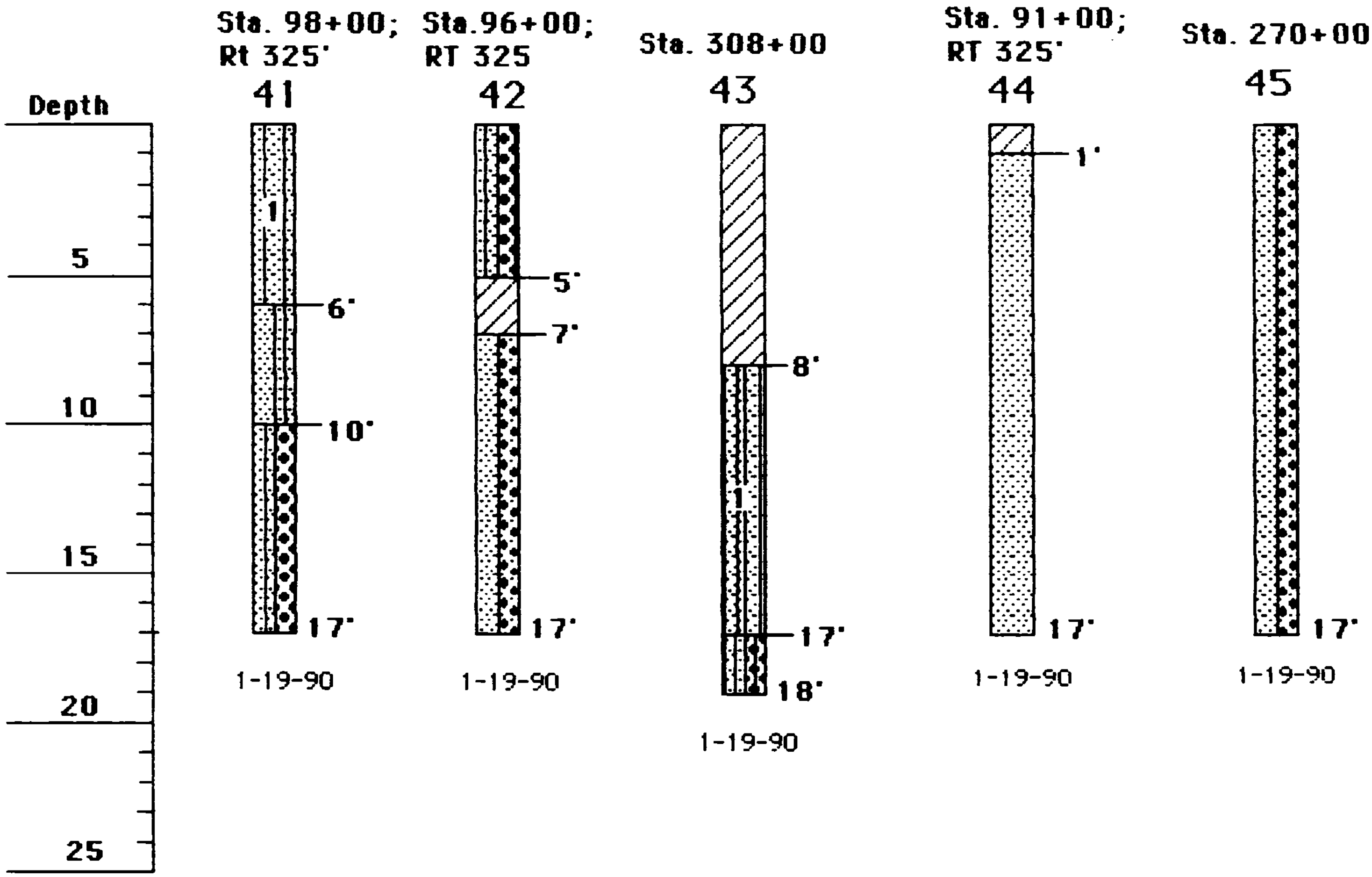

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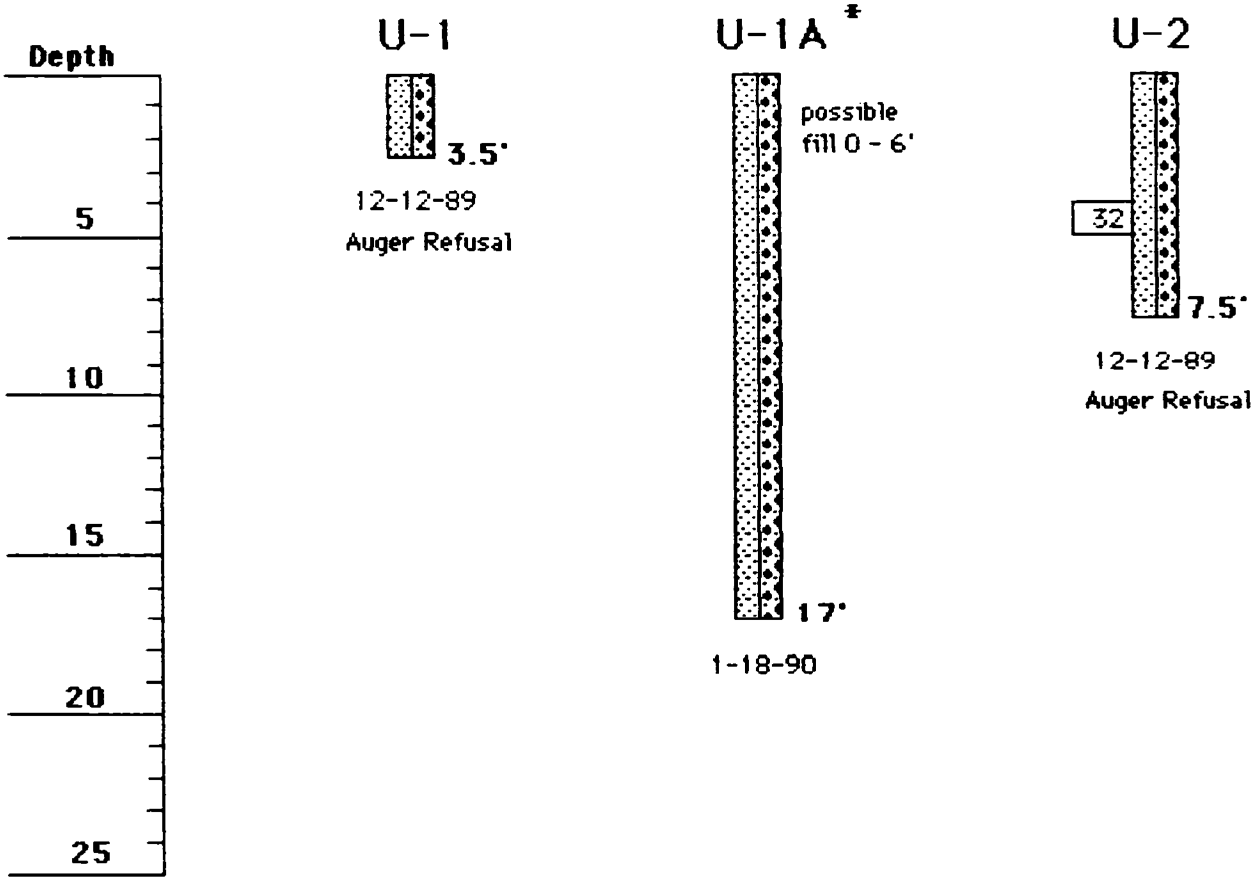

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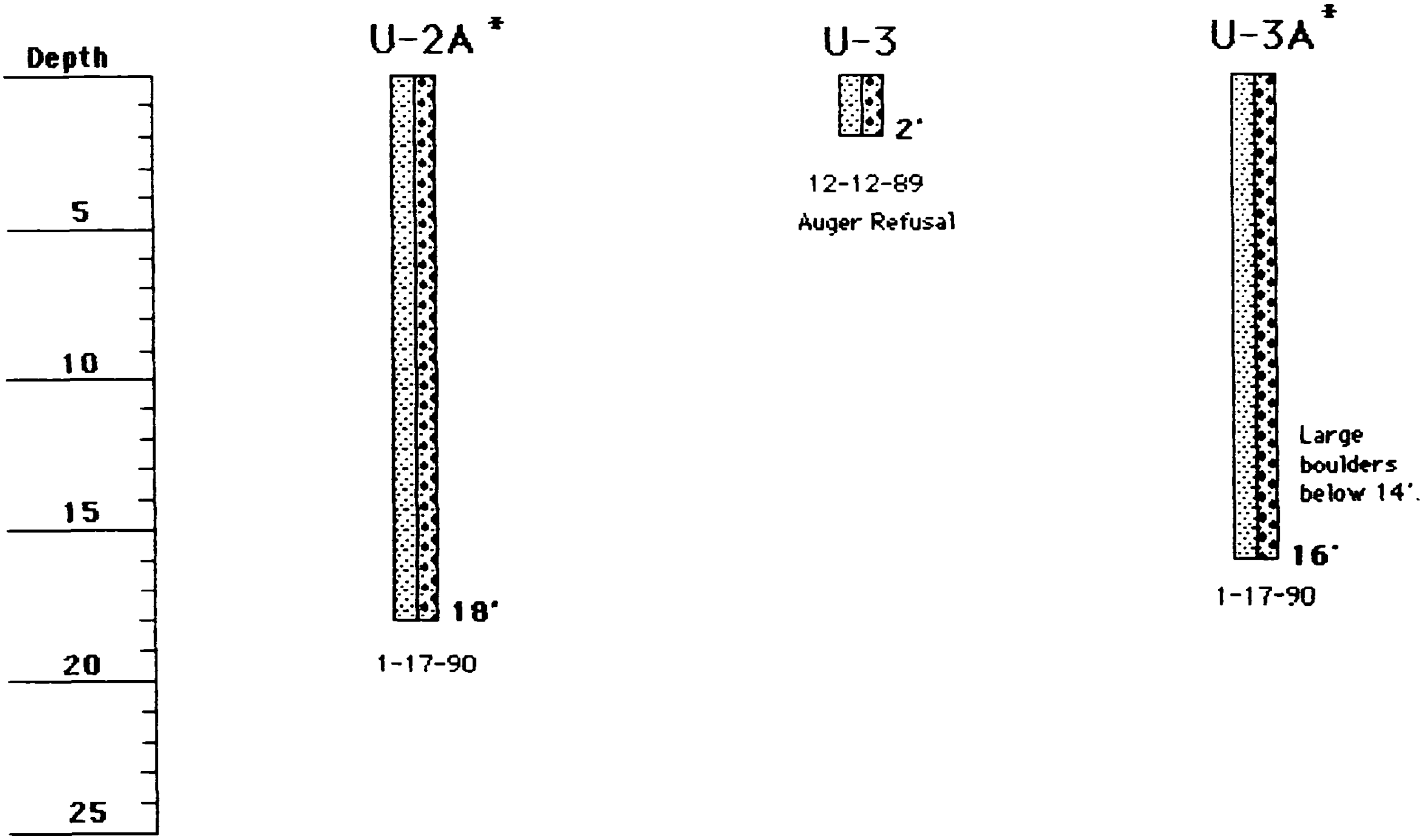


*Field exploration excavated with a Case 780 extend-a-hoe backhoe using a 24 inch wide bucket.

No free groundwater was encountered in any of the field exploration at the time of our field work unless noted otherwise on the boring logs.

All field explorations drilled with a CME 55 drill rig using a 7" diameter hollow stem auger unless otherwise noted.

Project No. 89-0919

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 FIELD TESTS

DESCRIPTION: Date: 2/13/90

Location: As Noted Below

Material: Soil

Performed By: TH/Perry & Thompson

TESTED: Visual estimates of material greater than 3 inches in size

RESULTS:

Location	Depth (feet)	Estimated Percent of Material Greater than 3 Inches
1	0-12.5	27-33
2	0-18	30-35
3	0-9	0-5
3	9-12.5	0-4
4	0-4	29-34
4	4-18	0
5	0-18	0
6	0-3	25-30
7	0-5	31-36
8	0-6	0-5
8	6-7.5	30-35
9	0-10.5	30-35
9A	0-15	40-50
10	0-18	0
11	0-4	28-33
11A	0-8	30-35
11A	8-10	0
11A	10-18	30-40
12	0-9	0
12	9-17	30-40
13	0-15	30-35
14	0-6	29-34
14A	0-4	0
14A	4-17	30-40
15	0-11	30-35

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON FIELD TESTS

DESCRIPTION: Date: 2/13/90

Location: As Noted Below

Material: Soil

Performed By: TH/Perry & Thompson

TESTED: Visual estimates of material greater than 3 inches in size

RESULTS:

Location	Depth (feet)	Estimated Percent of Material Greater than 3 Inches
18	0-14	30-35
19	0-3	30-35
19A	0-16	60-70
20	0-14	30-35
20	14-18	22-27
21	0-8.5	27-32
21A	0-18	0-20
22	0-18	0
23	0-17	30-40
24	0-15	60-70
25	0-14	50-60
26	0-17	30-40
27	0-17	30-40
28	0-8	20-30
28	8-12	30-40
28	12-18	40-50
29	0-18	40-50
33	0-18	40-50
34	0-7	0
34	7-17	30-40
35	0-8	30-40
35	8-17	40-50
36	0-7	0
36	7-18	30-40
37	0-10	0

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON FIELD TESTS

DESCRIPTION: Date: 2/13/90

Location: As Noted Below

Material: Soil

Performed By: TH/Perry & Thompson

TESTED: Visual estimates of material greater than 3 inches in size

RESULTS:

Location	Depth (feet)	Estimated Percent of Material Greater than 3 Inches
37	10-17	20-30
38	0-17	50-60
39	0-5	0
39	5-17	30-40
40	0-4	20-30
40	4-17	40-50
41	0-10	0
41	10-17	60-70
42	0-5	20-30
42	5-7	0
42	7-17	30-40
43	0-17	0
43	17-18	30-40
44	0-1	0
44	1-17	30-40
45	0-8	20-30
45	8-17	40-50
U-1	0-3.5	30-35
U-1A	0-5	60-70
U-1A	5-17	30-40
U-2	0-7	30-35
U-2A	0-9	20-30
U-2A	9-18	40-50
U-3	0-2	38-42
U-3A	0-13	30-40
U-3A	13-16	60-70

Project No. 89-0919

REPORT ON LABORATORY TESTS

SAMPLE: Date 2/7/90

Source As Noted Below

Type Bulk Samples

Material River Bed

Sampled By TH/Thompson

TESTED: Sieve Analysis & Plasticity Index - minus 3 inch materal

RESULTS: NOTE: Samples from test explorations 1 to 22 may not be representative of the amount of gravel & cobble size material present due to sorting by the auger.

Sample	LL	PI	Sieve Size -					Accum. % Passing						*
			200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class
1; 0 - 4'	--	NP	30	56	73	76	79	81	83	91	93	100		SM
2; 10-15'	--	NP	1	2	3	4	6	8	10	37	59	94	100	GP
3; 4 - 9'	--	NP	46	70	84	88	91	94	96	98	100			SM
4; 4 - 9'	--	NP	38	64	94	98	99	99	100					SM
5; 0 - 4'	--	NP	24	38	44	52	58	63	70	92	98	100		SM
6; 0 - 3'	33	12	13	14	14	15	17	18	19	40	54	95	100	GC
7; 0 - 4'	--	NP	2	4	13	39	52	58	63	80	86	97	100	SP
8; 4 - 7'	--	NP	3	6	24	58	69	75	78	91	96	100		SP
9; 4 - 9'	--	NP	2	4	6	16	31	43	54	84	92	100		SP-GP
10; 9 - 14'	--	NP	1	5	22	50	59	63	65	83	89	100		SP
11; 0 - 4'	--	NP	25	43	52	55	58	61	63	79	85	95	100	GM
12; 0 - 4'	--	NP	4	8	17	29	36	41	45	71	82	98	100	GP
13; 9 - 14'	--	NP	1	4	10	17	26	34	42	75	87	100		GP
15; 4 - 9'	--	NP	5	10	22	42	61	69	74	92	97	100		SP

NP = Non-Plastic

* Unified Soil Classification

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 2/7/90

Source As Noted Below

Type Bulk Samples

Material River Bed

Sampled By TH/Thompson

TESTED: Sieve Analysis & Plasticity Index - minus 3 inch material

RESULTS: NOTE: Samples from test explorations 1 to 22 may not be representative of the amount of gravel & cobble size material present due to sorting by the auger.

Sample	LL	PI	Sieve Size - 200	100	50	30	16	Accum. % Passing 8	4	3/4"	1"	2"	3"	* Class
18; 9 - 14'	--	NP	13	23	39	51	61	68	74	93	98	100		SM
19; 0 - 3'	--	NP	1	1	3	6	9	11	13	31	46	92	100	GP
20; 14 - 18'	26	4	2	20	40	50	60	70	81	100				SP
21; 4 - 9'	--	NP	7	12	22	34	42	45	47	61	68	97	100	GP-GM
22; 4 - 9'	--	NP	79	97	99	100								ML
23; 0 - 6'	--	NP	5	31	71	82	87	89	90	94	95	100		SP-SM
24; 0 - 16'	--	NP	12	24	33	35	36	36	37	42	47	73	** 95	GP-GM
25; 0 - 11'	--	NP	1	2	5	13	25	34	41	61	68	87	100	GP
26; 8 - 17'	--	NP	2	3	5	14	27	36	43	67	74	87	100	GP
27; 0 - 8'	--	NP	1	2	5	17	36	44	51	71	77	94	100	SP-GP
28; 8 - 18'	--	NP	2	3	6	11	18	26	34	59	66	83	100	GP
29; 0 - 8'	--	NP	1	2	4	12	24	31	37	53	59	71	100	GP
33; 0 - 8'	--	NP	1	1	4	11	25	34	42	63	69	85	100	GP
34; 9 - 17'	--	NP	5	7	12	22	35	43	49	68	74	91	100	GP-GM

NP = Non-Plastic

* Unified Soil Classification
**100 passing 3½" sieve

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 2/7/90

Source As Noted Below

Type Bulk Samples

Material River Bed

Sampled By TH/Thompson

TESTED: Sieve Analysis & Plasticity Index -minus 3 inch material

RESULTS: NOTE:

Sample	LL	PI	Sieve Size - 200	100	50	30	16	Accum. % Passing 8	4	3/4"	1"	2"	3"	* Class
35; 0 - 8'	--	NP	1	3	7	17	29	37	44	69	77	93	100	GP
36; 0 - 7'	--	NP	1	4	20	68	91	95	96	98	98	100		SP
37; 0 - 10'	--	NP	0	2	14	41	68	80	84	94	95	98	100	SP
38; 8 - 17'	--	NP	2	3	7	13	23	32	39	60	68	90	100	GP
39; 0 - 8'	--	NP	1	3	12	27	39	46	52	70	77	91	100	SP
40; 8 - 17	--	NP	1	2	6	15	27	36	43	63	70	90	100	GP
41; 0 - 6'	--	NP	36	64	93	97	98	99	99	100				SM
41; 6 - 10'	--	NP	7	15	52	85	94	96	97	100				SP-SM
42; 0 - 5'	--	NP	9	17	27	36	45	51	56	79	84	98	100	SP-GM
43; 0 - 8'	29	12	68	79	84	86	89	92	95	100				CL
44; 0 - 8'	--	NP	3	4	11	25	44	53	59	74	81	97	100	SP
45; 0 - 8'	--	NP	4	8	15	23	32	40	48	69	76	92	**95	GP
U1; 0 - 4'	--	NP	1	2	6	17	29	34	37	54	62	89	**94	GP
U2; 4 - 7'	--	NP	3	5	9	17	28	38	45	71	83	99	100	GP

NP = Non Plastic

* Unified Soil Classification

**100 passing 3½" sieve

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 2/7/90

Source As Noted Below

Type Bulk Samples

Material River Bed

Sampled By TH/Thompson

TESTED: Sieve Analysis & Plasticity Index - minus 3 inch material

RESULTS:

Sample	LL	PI	Sieve Size - 200	100	50	30	16	Accum. 8	4	% Passing 3/4"	1"	2"	3"	* Class
U3; 0 - 2'	--	NP	3	5	6	7	9	10	12	31	42	77	** 87	GP

NP = Non Plastic

* Unified Soil Classification
**100 passing 3½" sieve

Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 12/28/89

Source Test Boring 10;9 - 14'

Type Driven ring sample; 116 pcf dry density; 4% field moisture

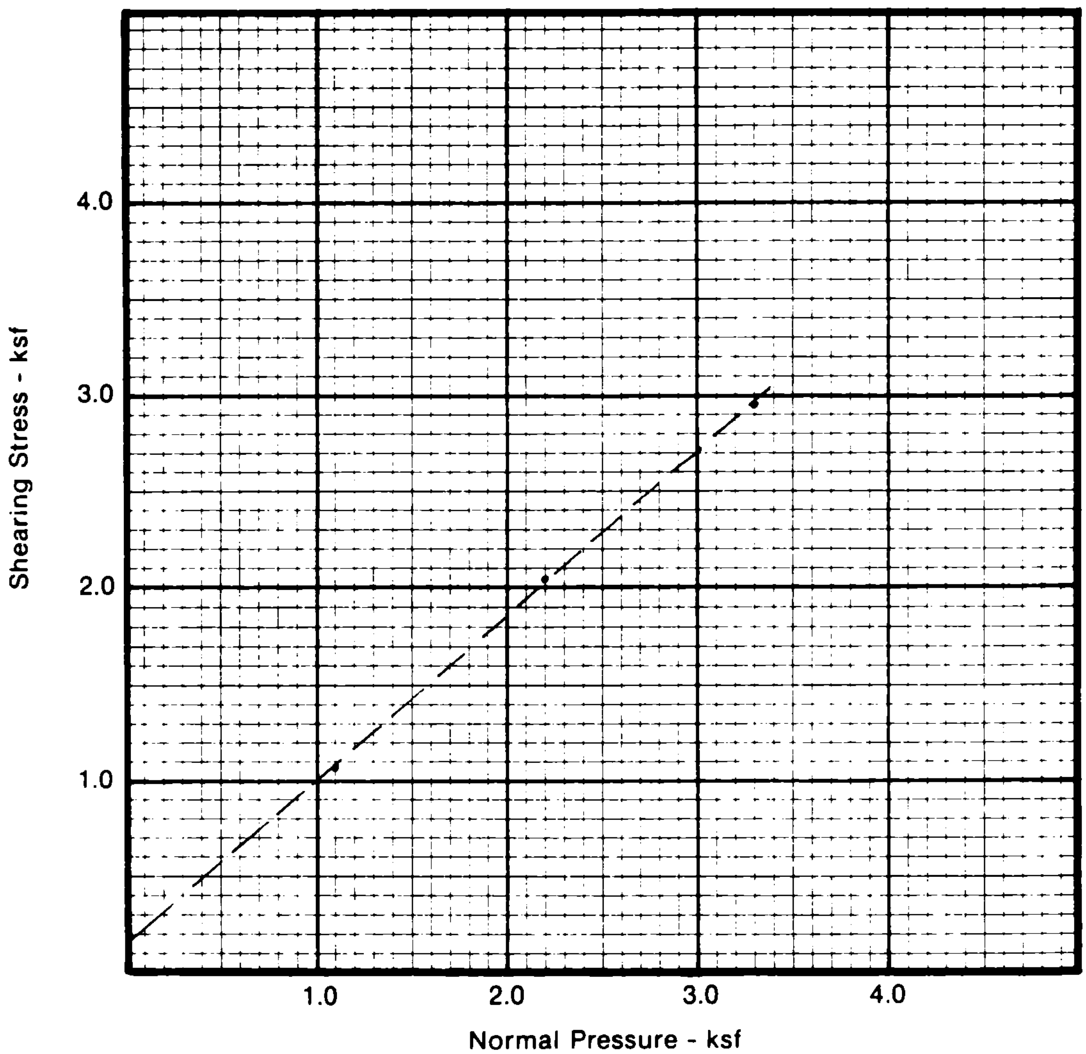
Material Sand & Gravel

Sampled By TH/Thompson

TESTED: Direct Shear - samples submerged during testing. Samples compacted to approximately 95% of ASTM D698 Dry Density.

RESULTS:

Friction Angle (Ø) = 41^{0} Cohesion (c) = 180 psf



Project No. 89-0919

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE:

Date 12/29/89

Source Test Boring 22; 4 - 9'

Type Driven ring sample; 101 pcf dry density; 18% field moisture

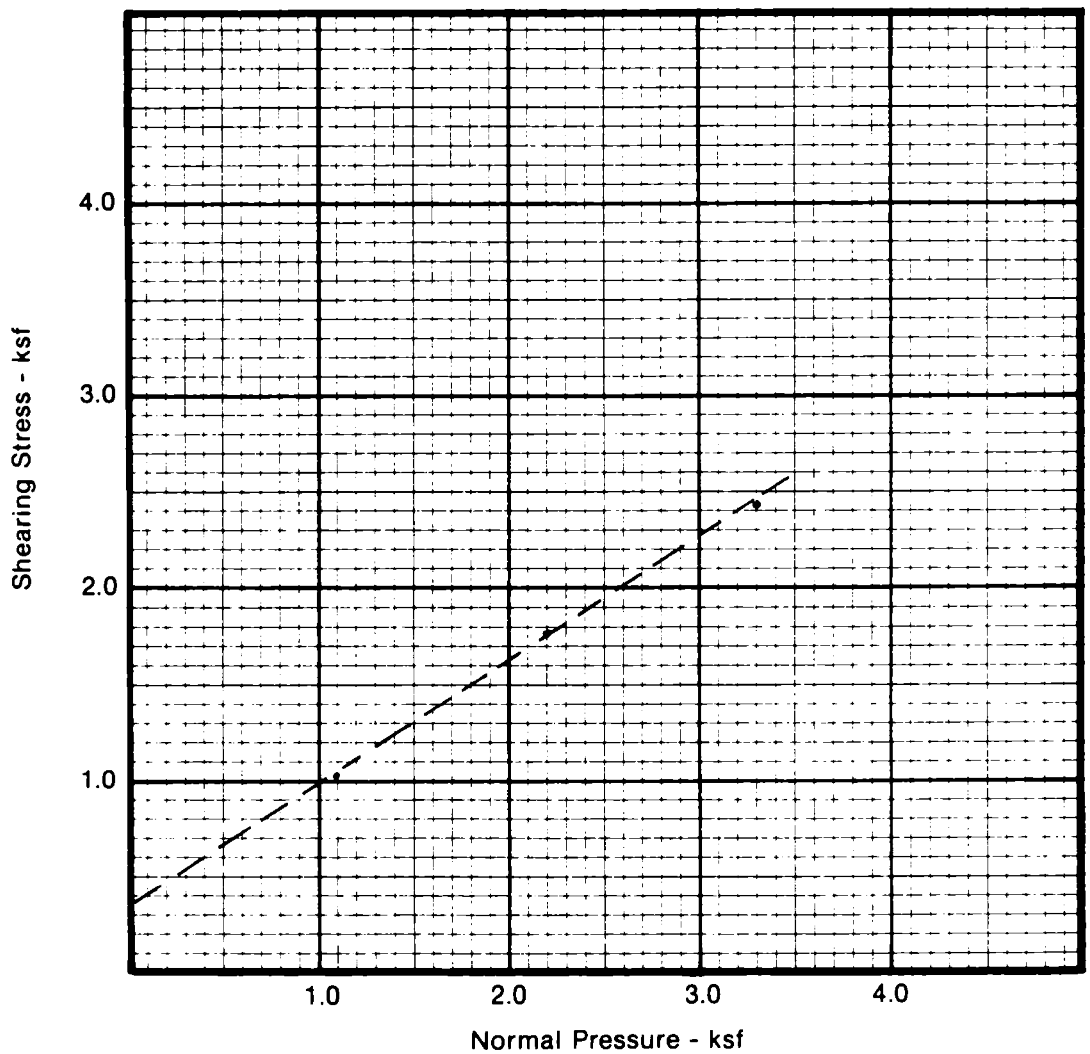
Material Sandy Clay

Sampled By TH/Thompson

TESTED: Direct Shear - samples submerged during testing. Samples compacted to approximately 95% of ASTM D698 Dry Density.

RESULTS:

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



Project No. 89-0919

TH-105

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/90

Source Test Boring 26; 8 - 17'

Type Driven ring sample; 113 pcf dry density;

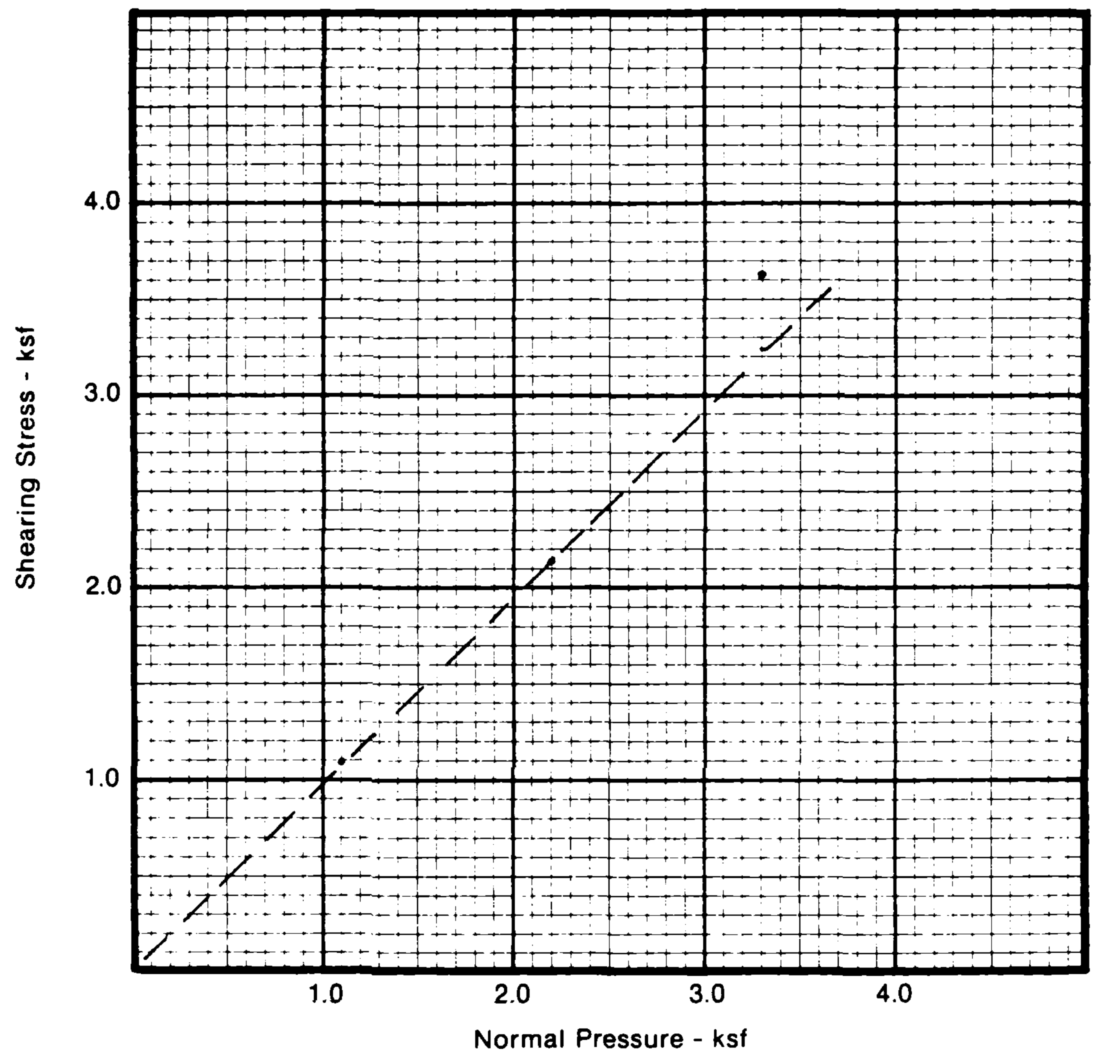
Material Sand & Gravel

Sampled By TH/Thompson

TESTED: Direct Shear - samples submerged during testing. Samples compacted to approximately 95% of ASTM D698 Dry Density.

RESULTS:

Friction Angle (Ø) = 44^{0} Cohesion (c) = 0



Project No. 89-0919

REPORT ON LABORATORY TESTS

SAMPLE:

Date 2/5/90

Source Test Boring 41; 0 - 6'

Type Driven ring sample; 105 pcf dry density; 15% field moisture

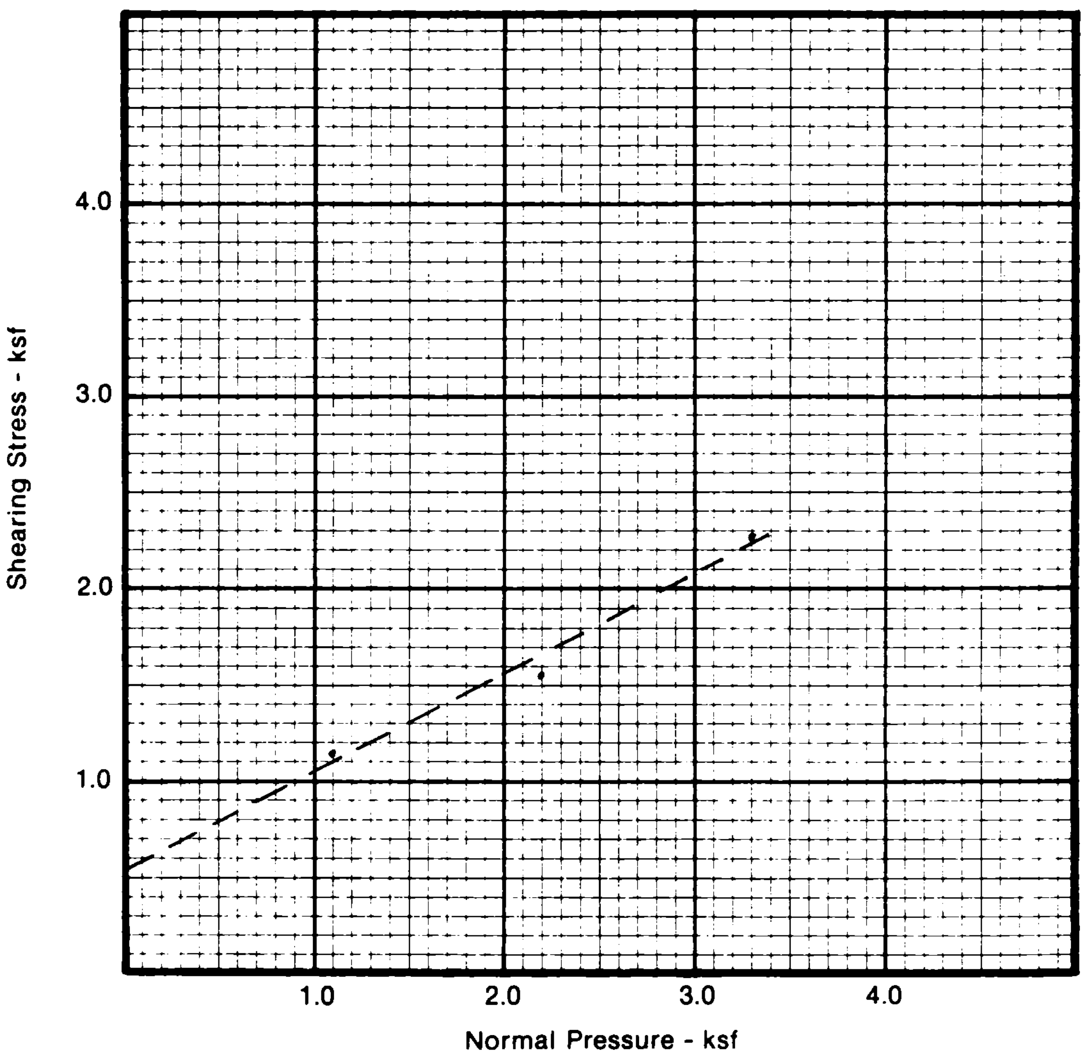
Material Silty Sand

Sampled By TH/Thompson

TESTED: Direct Shear - samples submerged during testing. Samples compacted to approximately 95% of ASTM D698 Dry Density.

RESULTS:

Friction Angle (Ø) = 27^{0} Cohesion (c) = 550 psf



Project No. 89-0919

REPORT ON LABORATORY TESTS

Date 2/5/90

SAMPLE:

Source Test Boring 41; 6 - 10'

Type Driven ring sample; 103 pcf dry density; 15% field moisture

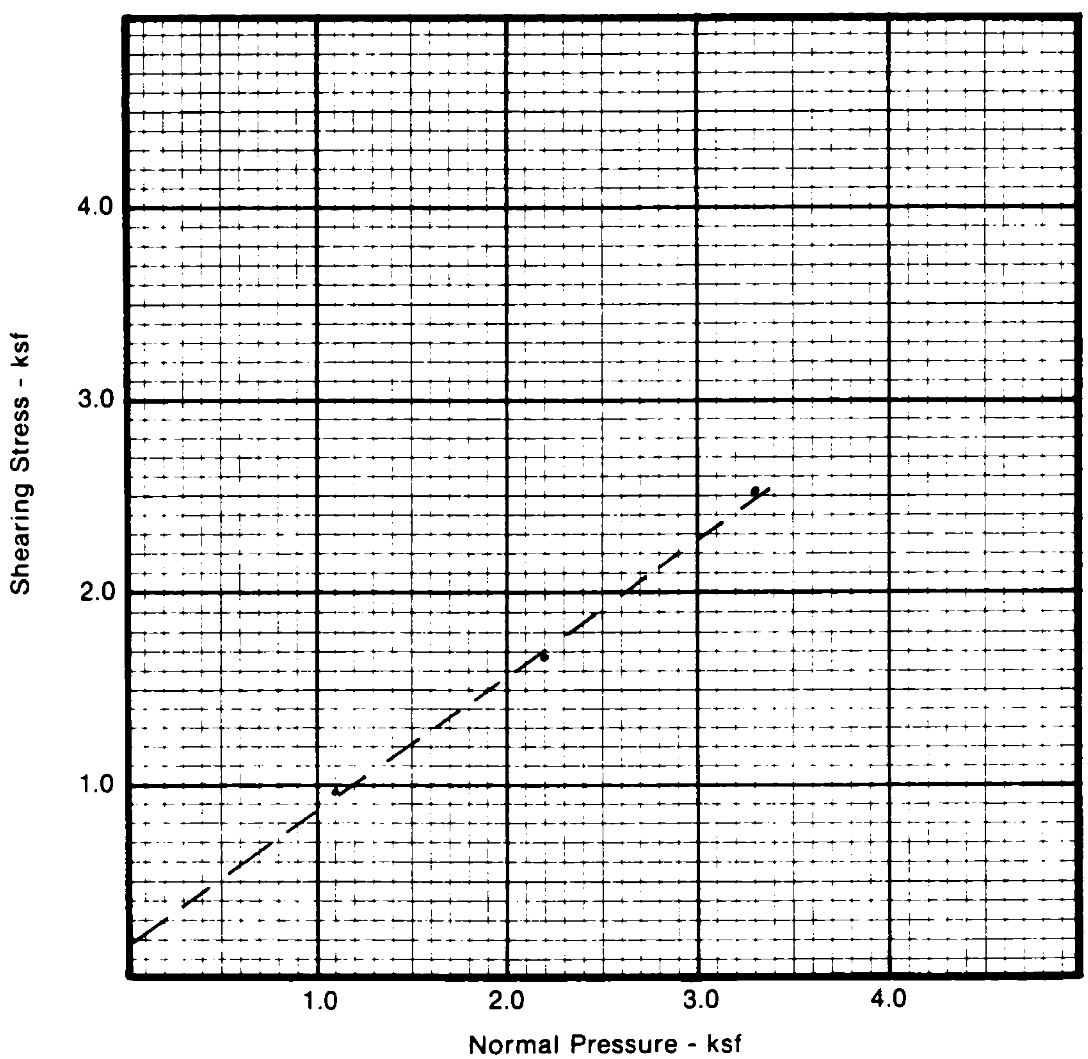
Material Sand with Silt

Sampled By TH/Thompson

TESTED: Direct Shear - samples submerged during testing. Samples compacted to approximately 95% of ASTM D698 Dry Density.

RESULTS:

Friction Angle (Ø) = 35^0 Cohesion (c) = 190 psf



Project No. 89-0919

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/25/90

Source Test Boring 26; 8 - 17'

Type Grab Sample

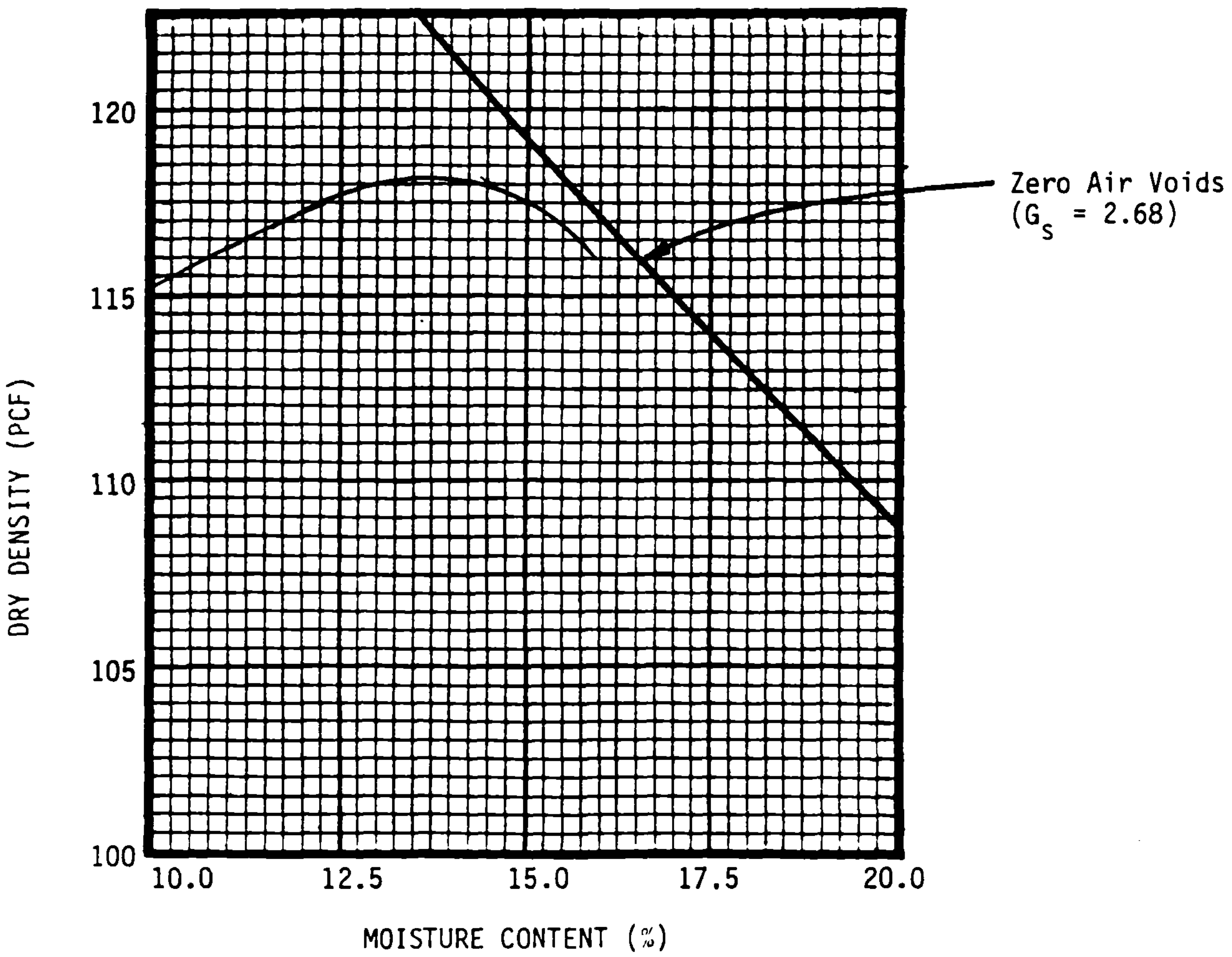
Material Soil

Sampled By TH/Thompson

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

RESULTS:

Max. Dry Density (pcf) 118.3 Optimum Moisture Content (%) 13.6



Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/29/90

Source Test Boring 33; 0 - 8'

Type Grab Sample

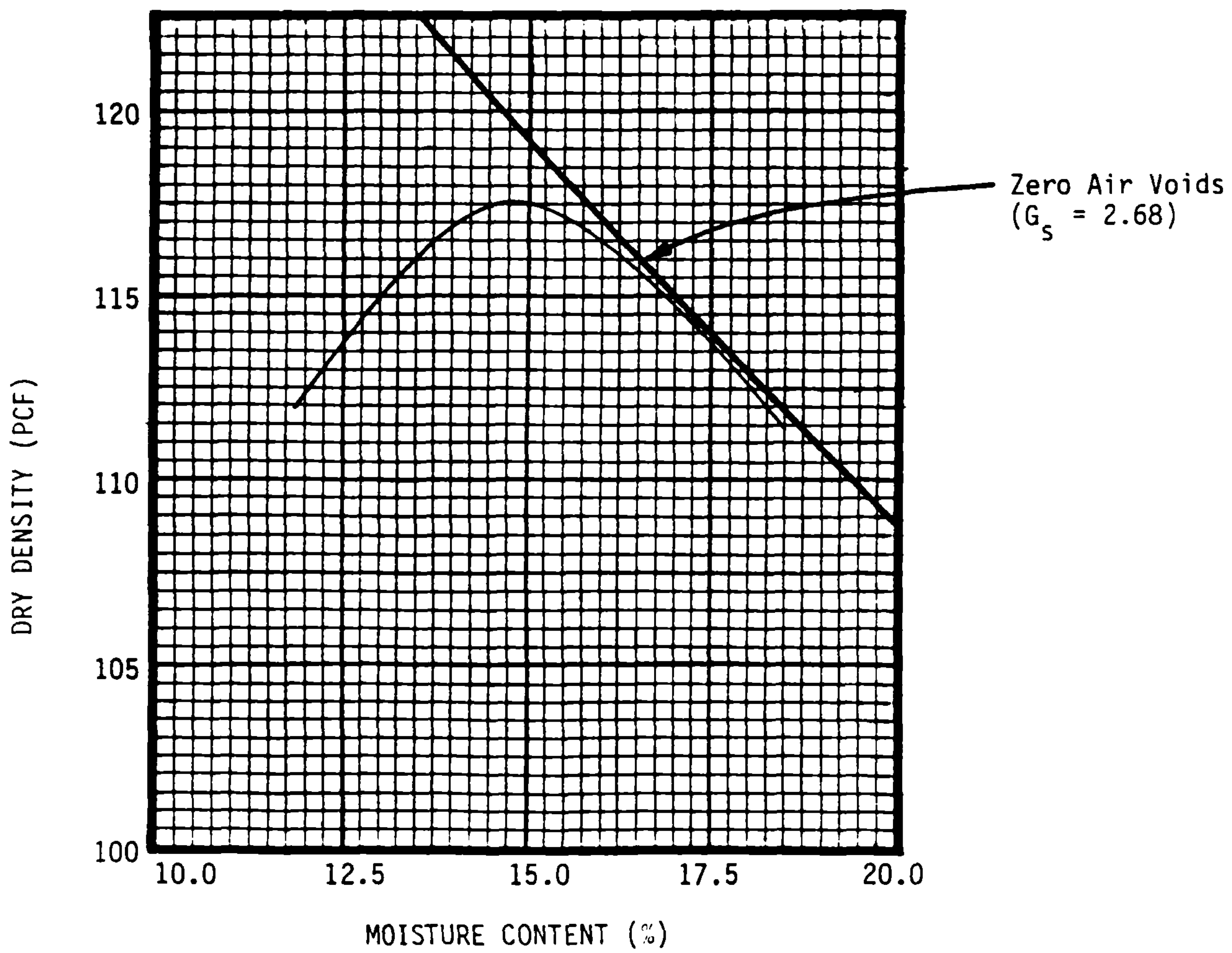
Material Soil

Sampled By TH/Thompson

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

RESULTS:

Max. Dry Density (pcf) 117.6 Optimum Moisture Content (%) 14.8



Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/31/90

Source Test Boring 41; 0 - 6'

Type Grab Sample

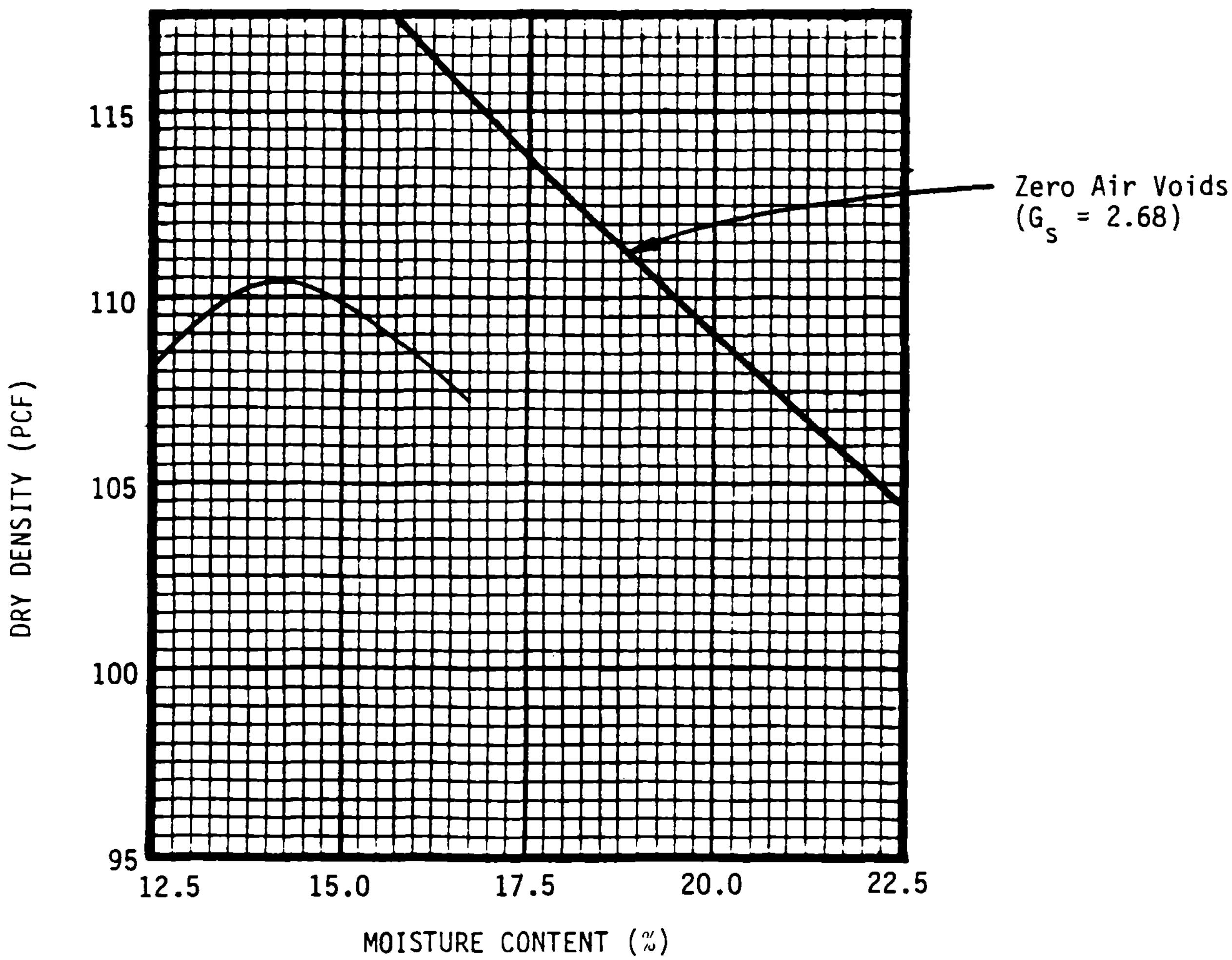
Material Soil

Sampled By TH/Thompson

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

RESULTS:

Max. Dry Density (pcf) 110.4 Optimum Moisture Content (%) 14.2



Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

SAMPLE: Date 1/30/90

Source Test Boring 41; 6 - 10'

Type Grab Sample

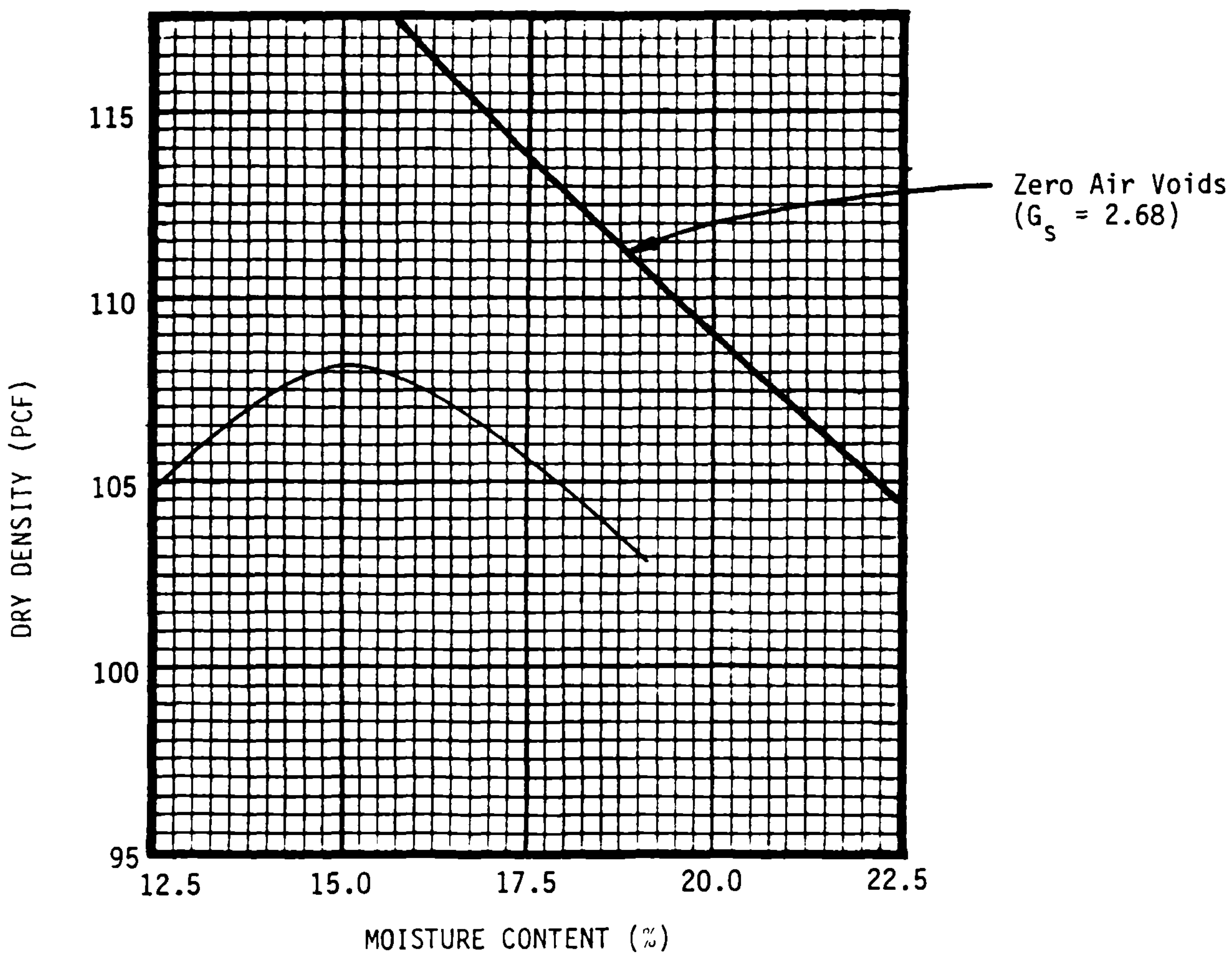
Material Soil

Sampled By TH/Thompson

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

RESULTS:

Max. Dry Density (pcf) 108.2 Optimum Moisture Content (%) 15.1



Project No. 89-0919

THOMAS-HARTIG & ASSOCIATES, INC.