Upper East Fork Cave Creek
Detention Basin #3A & 3B Design
Preliminary Design Submittal

Index No. ST-896829, P-901801

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

NBS LOWRY

ENGINEERS & PLANNERS

Upper East Fork Cave Creek
Detention Basin #3A & 3B Design
Preliminary Design Submittal

Index No. ST-896829, P-901801

Prepared For:

City of Phoenix, Arizona

Prepared By:

NBS/Lowry, Engineers & Planners
2600 North 44th Street
Phoenix, Arizona 85008

September 9, 1991

Detention Basin #3A & 3B
Preliminary Design Calculations

Table of Contents

I. Project Description

II. Design Criteria

III. Hydrology

- A. Off-site
  1. Subarea map
  2. TR-20 output (2 yr & 100 yr)
- B. On-site
  1. Subarea map
  2. Rational eqn. worksheet
  3. Inlet sizing calcs.
- C. Basin Routing
  1. Plots (WSEL, Q, 3A Rating curve)
  2. Storage - Indication routing calcs.

IV. Hydraulics

- A. Inlet Rating curves
  1. Basin 3A outlet
  2. Low flow pipes
  3. Basin 3B outlet
- B. Basin 3A - Pepperidge inlet
  1. Hydraulic Profile (Q100)
  2. Spillway Sizing
  3. Stability

Detention Basin #3A & 3B
Preliminary Design Calculations

Table of Contents
(Continued)

C. Basin 3A - Grovers Inlet
1. Hydraulic Profile (Q100)

D. Basin 3B - Outlet
1. Tailwater rating curve

V. Preliminary Sediment Analysis

VI. Structural Analysis

A. Pipes
1. D-Load (RCP)
2. Min. gage thickness (CMP)

VII. Preliminary Construction Cost Estimate.

VIII Geotechnical Report

Upper East Fork Cave Creek Detention Basin #3A & 3B
Project Description

The Upper East Fork Cave Creek Detention Basin #3A & 3B is being designed as part of the implementation of the Drainage Master Plan for the Upper East Fork Cave Creek watershed adopted by the City of Phoenix and the Flood Control District of Maricopa County and presented in the Upper East Fork Cave Creek Area Drainage Master Study, October 1987.

The project is located in the area north and south of Grovers Avenue between 20th Street and Cave Creek Road. The purpose of the basin is to reduce the 100 year peak discharge to the historic 2 year flow to allow the basin to drain into the City of Phoenix 2 year storm drain system in 20th Street. Upon completion of project construction the basin is to be incorporated into the City of Phoenix Park system for routine maintenance.

The project consists of providing all professional engineering services necessary for the design and preparation of plans, construction special provisions and construction cost estimates for the construction of detention basin #3A & 3B. Major project elements include park master planning for the ultimate park improvements, design plans for landscaping and irrigation, site area lighting, grading plans, inlet and outlet structure design, low flow drainage bypass system design, water and sanitary sewer relocations, and half street paving improvements around the perimeter of the basin.

Design calculations and descriptions of the major project elements follow:

Design Criteria

A. Basin Grading

Sideslopes	4:1	Granite areas
	5:1	Turf areas
Walking grades	12:1	max. (8.3%)
Equest. grades	6.7:1	max. (15%)
Bottom slope	1	percent min.

B. Pipes

Manning's n	0.013	(Concrete)
Min. Velocity	5	fps (desirable)
	2	fps (Absolute)
Diameter	18	inch min.

C. Basin 3B outfall

Peak Outlet Discharge	
Desired	462 cfs
Allowed	522 cfs
Attained	500 cfs

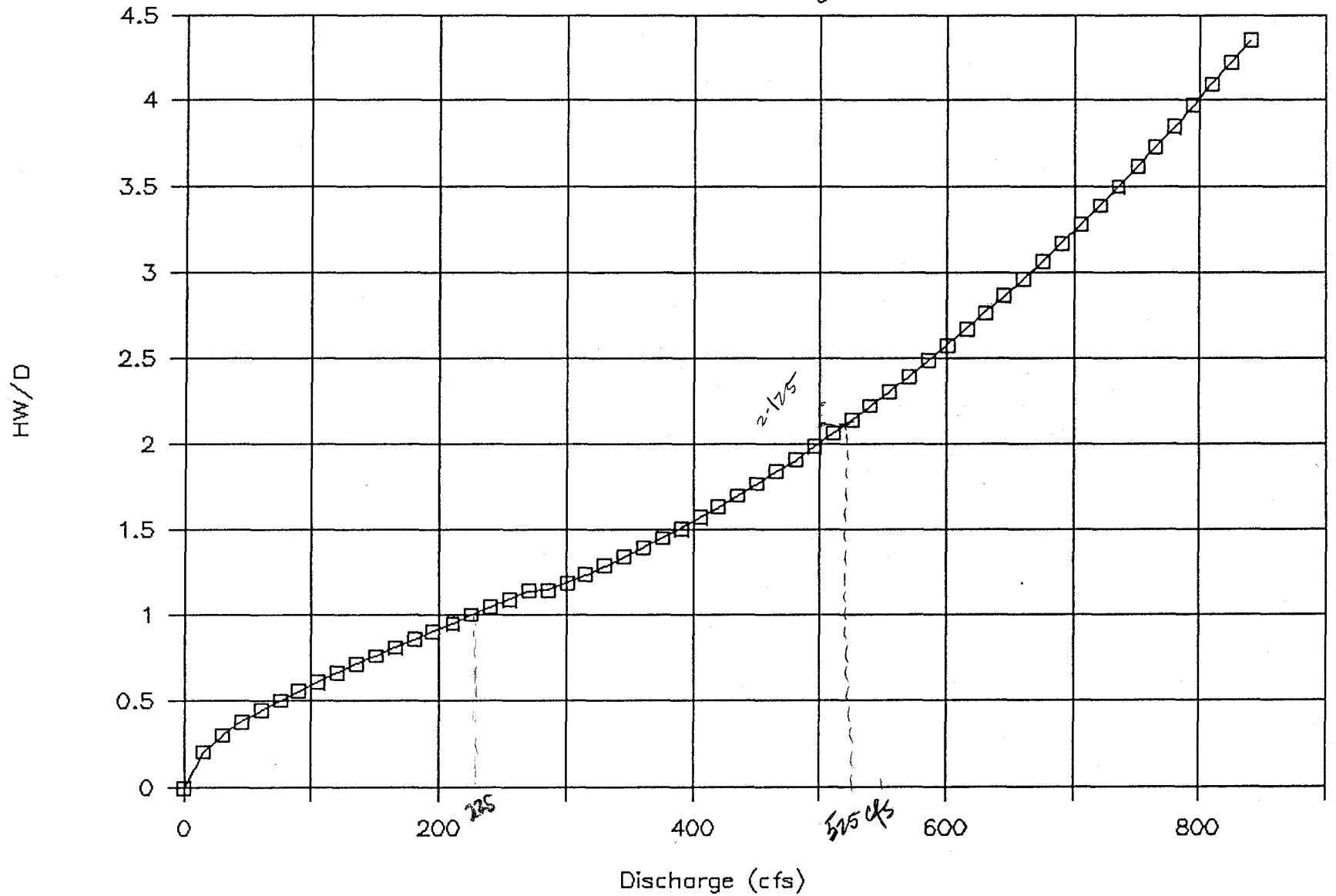
The basin outfall discharges into a proposed 120 inch stormdrain in 20th Street.

D. Storage

Freeboard	0.5	ft. min.
Drain Time	36	hrs. max.

DETENTION BASIN NO. 3 DESIGN

Basin 3A – Outlet Rating Curve



Culvert from A to B Analysis:

***** INLET CONTROL *****

No. Bbls= 3 K = 0.0078
Shape =circ, concrete M = 2
Hgt/Diam= 4 ft. C = 0.0292
Slope = 0.0224 ft/ft Y = 0.74
Q inc = 15 cfs Area = 12.57

--------- Reach 1 ----------

No. Bbls= 3 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 5.5 ft. Kb = 0
Length = 72 ft
[1+Kb+(29n^2L/Rh^1.33)]= 1.231033
Area = 23.75829

--------- Reach 2 ----------

No. Bbls= 3 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 4 ft. Kb = 0
Length = 174 ft. Ke = 0.2
[Ke+Kb+(29n^2L/Rh^1.33)]= 1.052774
Area = 12.56637

***** OUTLET CONTROL *****

US Inv. = 1423.00
DS Inv. = 1416.00
L*So = 7.00

Q	Q/bbl	Q/AD^.5	Hc(ft.)	HWi/D	Q/bbl	H1 (ft.)	Dc(ft)	Ho(ft)	Q/bbl	H2 (ft.)	HWo/D	Hl (ft.)	Control HW/D	Q	Control
0.00	0.00	0.000	0	0.00	0	0.00	0	2.75	0	0.00	-1.06	0.00	0.000	0	Inlet
15.00	5.00	0.199	0.87	0.21	5	0.00	0.6	3.05	5	0.00	-0.99	0.00	0.207	15	Inlet
30.00	10.00	0.398	1.25	0.30	10	0.00	0.85	3.18	10	0.01	-0.95	0.01	0.303	30	Inlet
45.00	15.00	0.597	1.54	0.38	15	0.01	1.04	3.27	15	0.02	-0.92	0.03	0.377	45	Inlet
60.00	20.00	0.796	1.79	0.44	20	0.01	1.21	3.36	20	0.04	-0.90	0.05	0.441	60	Inlet
75.00	25.00	0.995	2.02	0.50	25	0.02	1.35	3.43	25	0.06	-0.87	0.09	0.502	75	Inlet
90.00	30.00	1.194	2.23	0.56	30	0.03	1.48	3.49	30	0.09	-0.85	0.12	0.557	90	Inlet
105.00	35.00	1.393	2.43	0.61	35	0.04	1.61	3.56	35	0.13	-0.82	0.17	0.611	105	Inlet
120.00	40.00	1.592	2.62	0.66	40	0.05	1.72	3.61	40	0.17	-0.79	0.22	0.664	120	Inlet
135.00	45.00	1.790	2.8	0.71	45	0.07	1.83	3.67	45	0.21	-0.76	0.28	0.714	135	Inlet
150.00	50.00	1.989	2.97	0.76	50	0.08	1.93	3.72	50	0.26	-0.74	0.34	0.762	150	Inlet
165.00	55.00	2.188	3.14	0.81	55	0.10	2.03	3.77	55	0.31	-0.70	0.42	0.811	165	Inlet
180.00	60.00	2.387	3.3	0.86	60	0.12	2.12	3.81	60	0.37	-0.67	0.49	0.858	180	Inlet
195.00	65.00	2.586	3.46	0.91	65	0.14	2.21	3.86	65	0.44	-0.64	0.58	0.906	195	Inlet
210.00	70.00	2.785	3.61	0.95	70	0.17	2.3	3.90	70	0.51	-0.61	0.67	0.952	210	Inlet
225.00	75.00	2.984	3.77	1.00	75	0.19	2.38	3.94	75	0.58	-0.57	0.77	1.001	225	Inlet
240.00	80.00	3.183	3.92	1.05	80	0.22	2.46	3.98	80	0.66	-0.54	0.88	1.048	240	Inlet
255.00	85.00	3.382	4.07	1.10	85	0.24	2.54	4.02	85	0.75	-0.50	0.99	1.096	255	Inlet
270.00	90.00	3.581	4.22	1.14	90	0.27	2.62	4.06	90	0.84	-0.46	1.11	1.144	270	Inlet
285.00	95.00	3.780	4.37	1.15	95	0.31	2.7	4.10	95	0.93	-0.42	1.24	1.146	285	Inlet
300.00	100.00	3.979	0	1.19	100	0.34	2.77	4.14	100	1.04	-0.37	1.37	1.191	300	Inlet
315.00	105.00	4.178	0	1.24	105	0.37	2.84	4.17	105	1.14	-0.33	1.51	1.238	315	Inlet
330.00	110.00	4.377	0	1.29	110	0.41	2.91	4.21	110	1.25	-0.28	1.66	1.288	330	Inlet
345.00	115.00	4.576	0	1.34	115	0.45	2.98	4.24	115	1.37	-0.24	1.82	1.340	345	Inlet
360.00	120.00	4.775	0	1.39	120	0.49	3.05	4.28	120	1.49	-0.19	1.98	1.394	360	Inlet
375.00	125.00	4.974	0	1.45	125	0.53	3.11	4.31	125	1.62	-0.14	2.15	1.451	375	Inlet
390.00	130.00	5.173	0	1.51	130	0.57	3.18	4.34	130	1.75	-0.08	2.32	1.510	390	Inlet
405.00	135.00	5.371	0	1.57	135	0.62	3.24	4.37	135	1.89	-0.03	2.50	1.571	405	Inlet
420.00	140.00	5.570	0	1.63	140	0.66	3.3	4.40	140	2.03	0.02	2.69	1.635	420	Inlet

Culvert from A to B Analysis:

***** INLET CONTROL *****

No. Bbls= 3 K = 0.0078
Shape =circ, concrete M = 2
Hgt/Diam= 4 ft. C = 0.0292
Slope = 0.0224 ft/ft Y = 0.74
Q inc = 15 cfs Area = 12.57

--------- Reach 1 ----------

No. Bbls= 3 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 5.5 ft. Kb = 0
Length = 72 ft
[1+Kb+(29n^2L/Rh^1.33)]= 1.231033
Area = 23.75829

--------- Reach 2 ----------

No. Bbls= 3 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 4 ft. Kb = 0
Length = 174 ft. Ke = 0.2
[Ke+Kb+(29n^2L/Rh^1.33)]= 1.052774
Area = 12.56637

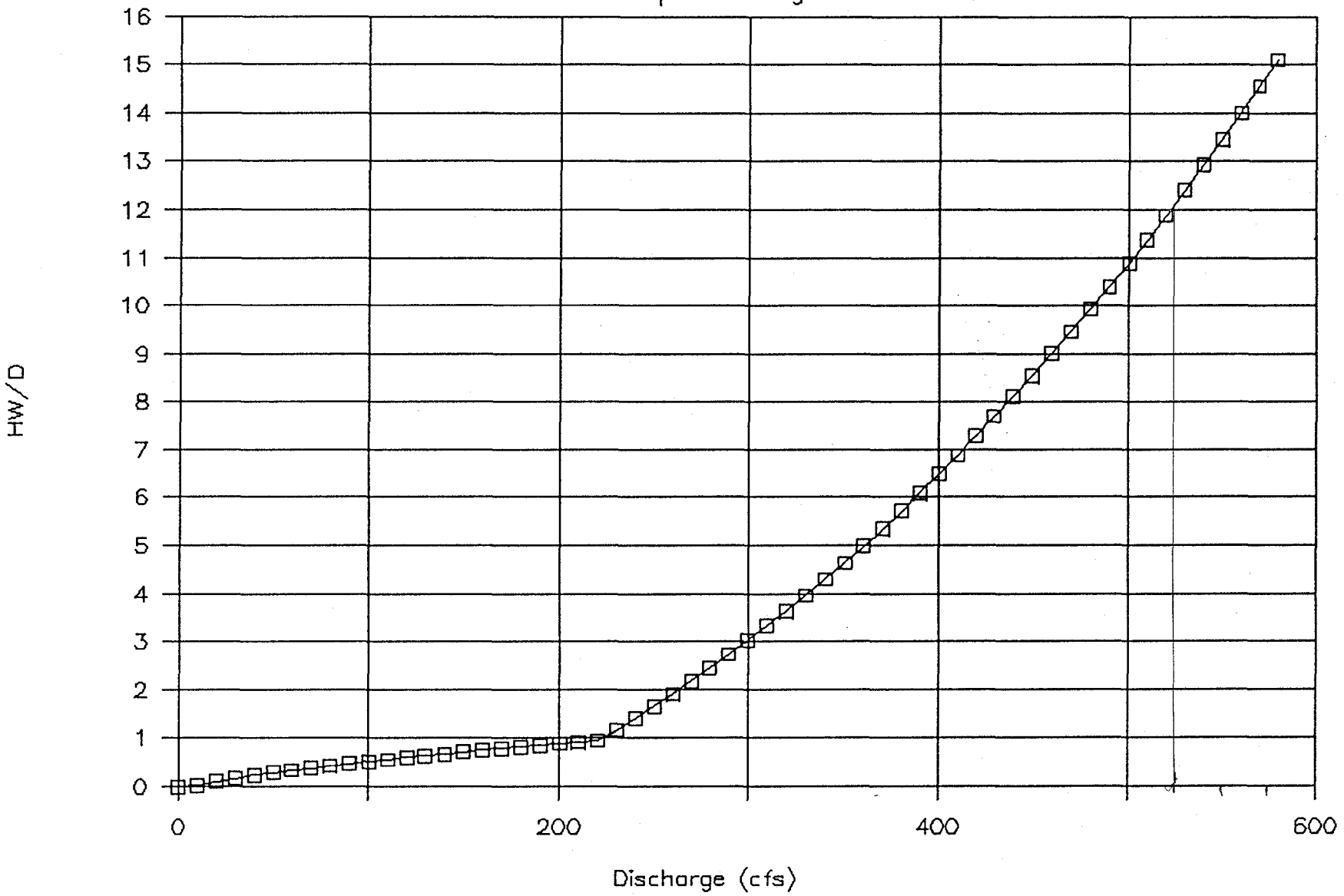
***** OUTLET CONTROL *****

US Inv. = 1423.00
DS Inv. = 1416.00
L*So = 7.00

Q	Q/bbl	Q/AD^.5	Hc(ft.)	HWi/D	Q/bbl	H1 (ft.)	Dc(ft)	Ho(ft)	Q/bbl	H2 (ft.)	HWo/D	Hl (ft.)	Control HW/D	Q	Control
435.00	145.00	5.769	0	1.70	145	0.71	3.36	4.43	145	2.18	0.08	2.89	1.701	435	Inlet
450.00	150.00	5.968	0	1.77	150	0.76	3.42	4.46	150	2.33	0.14	3.09	1.769	450	Inlet
465.00	155.00	6.167	0	1.84	155	0.81	3.48	4.49	155	2.49	0.20	3.30	1.839	465	Inlet
480.00	160.00	6.366	0	1.91	160	0.87	3.54	4.52	160	2.65	0.26	3.52	1.912	480	Inlet
495.00	165.00	6.565	0	1.99	165	0.92	3.59	4.55	165	2.82	0.32	3.74	1.987	495	Inlet
510.00	170.00	6.764	0	2.06	170	0.98	3.65	4.58	170	2.99	0.39	3.97	2.065	510	Inlet
525.00	175.00	6.963	0	2.14	175	1.04	3.7	4.60	175	3.17	0.45	4.21	2.145	525	Inlet
540.00	180.00	7.162	0	2.23	180	1.10	3.76	4.63	180	3.35	0.52	4.45	2.227	540	Inlet
555.00	185.00	7.361	0	2.31	185	1.16	3.81	4.66	185	3.54	0.59	4.70	2.311	555	Inlet
570.00	190.00	7.560	0	2.40	190	1.22	3.86	4.68	190	3.74	0.66	4.96	2.398	570	Inlet
585.00	195.00	7.759	0	2.49	195	1.29	3.91	4.71	195	3.94	0.73	5.22	2.487	585	Inlet
600.00	200.00	7.958	0	2.58	200	1.35	3.96	4.73	200	4.14	0.81	5.50	2.578	600	Inlet
615.00	205.00	8.157	0	2.67	205	1.42	4.01	4.76	205	4.35	0.88	5.77	2.672	615	Inlet
630.00	210.00	8.356	0	2.77	210	1.49	4.06	4.78	210	4.57	0.96	6.06	2.767	630	Inlet
645.00	215.00	8.555	0	2.87	215	1.57	4.11	4.81	215	4.79	1.04	6.35	2.866	645	Inlet
660.00	220.00	8.754	0	2.97	220	1.64	4.16	4.83	220	5.01	1.12	6.65	2.966	660	Inlet
675.00	225.00	8.952	0	3.07	225	1.71	4.16	4.83	225	5.24	1.20	6.96	3.069	675	Inlet
690.00	230.00	9.151	0	3.17	230	1.79	4.25	4.88	230	5.48	1.29	7.27	3.174	690	Inlet
705.00	235.00	9.350	0	3.28	235	1.87	4.29	4.90	235	5.72	1.37	7.59	3.282	705	Inlet
720.00	240.00	9.549	0	3.39	240	1.95	4.33	4.92	240	5.96	1.46	7.91	3.392	720	Inlet
735.00	245.00	9.748	0	3.50	245	2.03	4.37	4.94	245	6.21	1.55	8.25	3.504	735	Inlet
750.00	250.00	9.947	0	3.62	250	2.12	4.41	4.96	250	6.47	1.64	8.59	3.618	750	Inlet
765.00	255.00	10.146	0	3.73	255	2.20	4.45	4.98	255	6.73	1.73	8.93	3.735	765	Inlet
780.00	260.00	10.345	0	3.85	260	2.29	4.49	5.00	260	7.00	1.82	9.29	3.854	780	Inlet
795.00	265.00	10.544	0	3.98	265	2.38	4.53	5.02	265	7.27	1.92	9.65	3.975	795	Inlet
810.00	270.00	10.743	0	4.10	270	2.47	4.57	5.04	270	7.55	2.01	10.02	4.099	810	Inlet
825.00	275.00	10.942	0	4.22	275	2.56	4.57	5.04	275	7.83	2.11	10.39	4.225	825	Inlet
840.00	280.00	11.141	0	4.35	280	2.66	4.64	5.07	280	8.12	2.21	10.77	4.353	840	Inlet
840.00	280.00	11.141	11.7	5	280	2.66	4.64	5.07	280	8.12	2.21	10.77	5.000	840	Inlet

DETENTION BASIN NO. 3 DESIGN

Lowflow Pipe – Rating Curve



Low Flow Pipe Analysis:

***** INLET CONTROL *****

No. Bbls= 1 K = 0.061
Shape =Rect. Box M = 0.75
Hgt/Diam= 4 ft. C = 0.04
Width = 10 ft. Y = 0.8
Slope = 0.2 ft/ft
Q inc = 10 cfs Area = 40.00

--------- Reach 1 ----------

No. Bbls= 2 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 4.5 ft. Kb = 0.14
Length = 1500 ft
[1+Kb+(29n^2L/Rh^1.33)]= 7.425547
Area = 15.90431

--------- Reach 2 ----------

No. Bbls= 1 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 6 ft. Kb = 0.16
Length = 1320 ft. Ke = 0.2
[Ke+Kb+(29n^2L/Rh^1.33)]= 4.132743
Area = 28.27433

***** OUTLET CONTROL *****

US Inv. = 1420.93
DS Inv. = 1411.50
L*So = 9.43

Q	Q/bbl	Q/AD^.5	Dc(ft.)	Hc(ft.)	HWi/D	Q/bbl	H1 (ft.)	Dc(ft)	Ho(ft)	Q/bbl	H2 (ft.)	HWo/D	Hl (ft.)	Control HW/D	Q	Control
0.00	0.00	0.000	0.000	0	0.00	0	0.00	0	2.25	0	0.00	-1.80	0.00	0.000	0	Inlet
10.00	10.00	0.125	0.315	0.47	0.03	5	0.01	0.63	2.57	10	0.01	-1.71	0.02	0.031	10	Inlet
20.00	20.00	0.250	0.500	0.75	0.11	10	0.05	0.9	2.70	20	0.03	-1.66	0.08	0.109	20	Inlet
30.00	30.00	0.375	0.655	0.98	0.17	15	0.10	1.1	2.80	30	0.07	-1.61	0.17	0.174	30	Inlet
40.00	40.00	0.500	0.794	1.19	0.23	20	0.18	1.28	2.89	40	0.13	-1.56	0.31	0.233	40	Inlet
50.00	50.00	0.625	0.921	1.38	0.29	25	0.28	1.43	2.97	50	0.20	-1.49	0.49	0.288	50	Inlet
60.00	60.00	0.750	1.040	1.56	0.34	30	0.41	1.57	3.04	60	0.29	-1.42	0.70	0.338	60	Inlet
70.00	70.00	0.875	1.153	1.73	0.39	35	0.56	1.7	3.10	70	0.39	-1.34	0.95	0.387	70	Inlet
80.00	80.00	1.000	1.260	1.89	0.43	40	0.73	1.83	3.17	80	0.51	-1.26	1.24	0.432	80	Inlet
90.00	90.00	1.125	1.363	2.04	0.48	45	0.92	1.94	3.22	90	0.65	-1.16	1.57	0.477	90	Inlet
100.00	100.00	1.250	1.462	2.19	0.52	50	1.14	2.05	3.28	100	0.80	-1.05	1.94	0.519	100	Inlet
110.00	110.00	1.375	1.558	2.33	0.56	55	1.38	2.16	3.33	110	0.97	-0.94	2.35	0.560	110	Inlet
120.00	120.00	1.500	1.651	2.47	0.60	60	1.64	2.26	3.38	120	1.16	-0.81	2.80	0.601	120	Inlet
130.00	130.00	1.625	1.742	2.61	0.64	65	1.93	2.35	3.43	130	1.36	-0.68	3.28	0.639	130	Inlet
140.00	140.00	1.750	1.830	2.74	0.68	70	2.23	2.44	3.47	140	1.57	-0.54	3.81	0.678	140	Inlet
150.00	150.00	1.875	1.916	2.87	0.71	75	2.56	2.53	3.52	150	1.81	-0.39	4.37	0.715	150	Inlet
160.00	160.00	2.000	2.000	2.99	0.75	80	2.92	2.62	3.56	160	2.05	-0.22	4.97	0.751	160	Inlet
170.00	170.00	2.125	2.083	3.12	0.79	85	3.29	2.71	3.61	170	2.32	-0.05	5.61	0.787	170	Inlet
180.00	180.00	2.250	2.164	3.24	0.82	90	3.69	2.79	3.65	180	2.60	0.13	6.29	0.822	180	Inlet
190.00	190.00	2.375	2.243	3.36	0.86	95	4.11	2.87	3.69	190	2.90	0.32	7.01	0.856	190	Inlet
200.00	200.00	2.500	2.321	3.47	0.89	100	4.56	2.94	3.72	200	3.21	0.51	7.77	0.890	200	Inlet
210.00	210.00	2.625	2.398	3.59	0.92	105	5.03	3.02	3.76	210	3.54	0.72	8.57	0.923	210	Inlet
220.00	220.00	2.750	2.473	3.70	0.96	110	5.52	3.09	3.80	220	3.89	0.94	9.40	0.956	220	Inlet
230.00	230.00	2.875	2.548	3.81	0.99	115	6.03	3.16	3.83	230	4.25	1.17	10.27	1.169	230	Outlet
240.00	240.00	3.000	2.621	3.92	1.02	120	6.56	3.23	3.87	240	4.62	1.41	11.19	1.406	240	Outlet
250.00	250.00	3.125	2.693	4.03	1.05	125	7.12	3.3	3.90	250	5.02	1.65	12.14	1.652	250	Outlet
260.00	260.00	3.250	2.765	4.14	1.08	130	7.70	3.36	3.93	260	5.43	1.91	13.13	1.908	260	Outlet
270.00	270.00	3.375	2.835	4.24	1.11	135	8.31	3.42	3.96	270	5.85	2.17	14.16	2.172	270	Outlet
280.00	280.00	3.500	2.905	4.35	1.14	140	8.93	3.48	3.99	280	6.29	2.45	15.23	2.447	280	Outlet

Low Flow Pipe Analysis:

***** INLET CONTROL *****

No. Bbls= 1 K = 0.061
Shape =Rect. Box M = 0.75
Hgt/Diam= 4 ft. C = 0.04
Width = 10 ft. Y = 0.8
Slope = 0.2 ft/ft
Q inc = 10 cfs Area = 40.00

Reach 1

No. Bbls= 2 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 4.5 ft. Kb = 0.14
Length = 1500 ft
[1+Kb+(29n^2L/Rh^1.33)]= 7.425547
Area = 15.90431

Reach 2

No. Bbls= 1 n = 0.013
Shape =Circ. Concrete
Hgt/Diam= 6 ft. Kb = 0.16
Length = 1320 ft. Ke = 0.2
[Ke+Kb+(29n^2L/Rh^1.33)]= 4.132743
Area = 28.27433

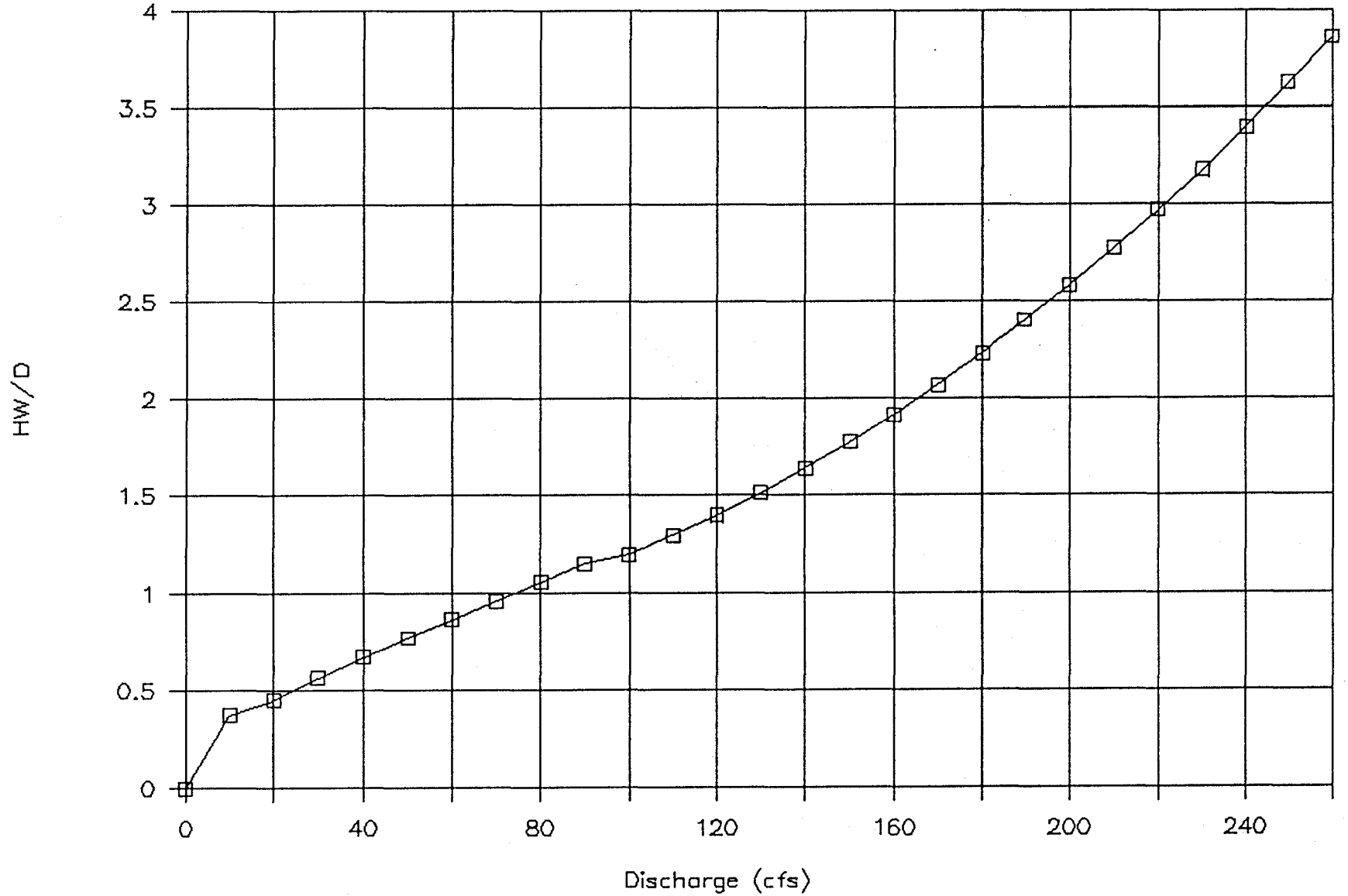
***** OUTLET CONTROL *****

US Inv. = 1420.93
DS Inv. = 1411.50
L*So = 9.43

Q	Q/bbl	Q/AD^.5	Dc(ft.)	Hc(ft.)	HWi/D	Q/bbl	H1 (ft.)	Dc(ft)	Ho(ft)	Q/bbl	H2 (ft.)	HWo/D	Hl (ft.)	Control HW/D	Q	Control
290.00	290.00	3.625	2.973	4.45	1.17	145	9.58	3.54	4.02	290	6.75	2.73	16.33	2.731	290	Outlet
300.00	300.00	3.750	3.041	4.55	1.26	150	10.26	3.6	4.05	300	7.22	3.03	17.48	3.025	300	Outlet
310.00	310.00	3.875	3.109	4.65	1.30	155	10.95	3.65	4.08	310	7.71	3.33	18.67	3.328	310	Outlet
320.00	320.00	4.000	3.175	4.75	1.34	160	11.67	3.7	4.10	320	8.22	3.64	19.89	3.640	320	Outlet
330.00	330.00	4.125	3.241	4.85	1.38	165	12.41	3.75	4.13	330	8.74	3.96	21.15	3.962	330	Outlet
340.00	340.00	4.250	3.306	4.95	1.42	170	13.17	3.8	4.15	340	9.28	4.29	22.45	4.293	340	Outlet
350.00	350.00	4.375	3.370	5.04	1.47	175	13.96	3.85	4.18	350	9.83	4.63	23.79	4.635	350	Outlet
360.00	360.00	4.500	3.434	5.14	1.51	180	14.77	3.89	4.20	360	10.40	4.98	25.17	4.984	360	Outlet
370.00	370.00	4.625	3.498	5.24	1.56	185	15.60	3.93	4.22	370	10.99	5.34	26.59	5.344	370	Outlet
380.00	380.00	4.750	3.560	5.33	1.60	190	16.46	3.97	4.24	380	11.59	5.71	28.05	5.713	380	Outlet
390.00	390.00	4.875	3.623	5.42	1.65	195	17.33	4.01	4.26	390	12.21	6.09	29.54	6.092	390	Outlet
400.00	400.00	5.000	3.684	5.51	1.70	200	18.23	4.04	4.27	400	12.84	6.48	31.08	6.479	400	Outlet
410.00	410.00	5.125	3.745	5.61	1.75	205	19.16	4.07	4.29	410	13.49	6.88	32.65	6.876	410	Outlet
420.00	420.00	5.250	3.806	5.70	1.80	210	20.10	4.07	4.29	420	14.16	7.28	34.26	7.279	420	Outlet
430.00	430.00	5.375	3.866	5.79	1.86	215	21.07	4.13	4.32	430	14.84	7.70	35.91	7.700	430	Outlet
440.00	440.00	5.500	3.926	5.88	1.91	220	22.06	4.16	4.33	440	15.54	8.13	37.60	8.126	440	Outlet
450.00	450.00	5.625	3.985	5.97	1.97	225	23.08	4.18	4.34	450	16.26	8.56	39.33	8.561	450	Outlet
460.00	460.00	5.750	4.044	6.05	2.02	230	24.11	4.21	4.36	460	16.99	9.01	41.10	9.006	460	Outlet
470.00	470.00	5.875	4.102	6.14	2.08	235	25.17	4.23	4.37	470	17.73	9.46	42.91	9.460	470	Outlet
480.00	480.00	6.000	4.160	6.23	2.14	240	26.26	4.25	4.38	480	18.49	9.92	44.75	9.924	480	Outlet
490.00	490.00	6.125	4.218	6.31	2.20	245	27.36	4.26	4.38	490	19.27	10.40	46.64	10.396	490	Outlet
500.00	500.00	6.250	4.275	6.40	2.26	250	28.49	4.28	4.39	500	20.07	10.88	48.56	10.880	500	Outlet
510.00	510.00	6.375	4.332	6.48	2.33	255	29.64	4.28	4.39	510	20.88	11.37	50.52	11.370	510	Outlet
520.00	520.00	6.500	4.388	6.57	2.39	260	30.81	4.31	4.41	520	21.71	11.87	52.52	11.874	520	Outlet
530.00	530.00	6.625	4.445	6.65	2.46	265	32.01	4.5	4.50	530	22.55	12.41	54.56	12.407	530	Outlet
540.00	540.00	6.750	4.500	6.74	2.52	270	33.23	4.5	4.50	540	23.41	12.93	56.64	12.927	540	Outlet
550.00	550.00	6.875	4.556	6.82	2.59	275	34.47	4.5	4.50	550	24.28	13.46	58.76	13.456	550	Outlet
560.00	560.00	7.000	4.611	6.90	2.66	280	35.74	4.5	4.50	560	25.17	14.00	60.91	13.995	560	Outlet
570.00	570.00	7.125	4.665	6.98	2.73	285	37.03	4.5	4.50	570	26.08	14.54	63.11	14.544	570	Outlet

DETENTION BASIN NO. 3 DESIGN

Basin 3B – Outlet Rating Curve



Basin 3B Outlet Pipe Analysis:

***** INLET CONTROL *****				***** OUTLET CONTROL *****		
No. Bbls=	1	K =	0.0078	Mann. n = 0.013	US Inv. =	1417.00
Shape	=circ, concrete	M =	2	Inlet Ke= 0.2	DS Inv. =	1416.00
Hgt/Diam=	4 ft.	C =	0.0292	Length = 85 ft	L*So =	1
Slope =	0.0118 ft/ft	Y =	0.74	[1+Ke+(29n^2L/Rh^1.33)]=		1.616585
Q inc =	10 cfs	Area =	12.57	Avg Slope 0.0118		

Q	Q/bbl	Q/AD^.5	Hc(ft.)	HWi/D	H(ft)	Dc(ft)	Ho(ft)	HWo/D	Control HW/D	Q	Control
0.00	0.00	0.000	0	0.00	0.00	0	2.00	0.00	0.000	0	Outlet
10.00	10.00	0.398	1.25	0.31	0.02	0.93	2.47	0.37	0.370	10	Outlet
20.00	20.00	0.796	1.79	0.45	0.06	1.32	2.66	0.43	0.447	20	Inlet
30.00	30.00	1.194	2.23	0.56	0.14	1.63	2.82	0.49	0.563	30	Inlet
40.00	40.00	1.592	2.62	0.67	0.25	1.89	2.95	0.55	0.669	40	Inlet
50.00	50.00	1.989	2.97	0.77	0.40	2.13	3.07	0.62	0.767	50	Inlet
60.00	60.00	2.387	3.3	0.86	0.57	2.34	3.17	0.69	0.864	60	Inlet
70.00	70.00	2.785	3.61	0.96	0.78	2.53	3.27	0.76	0.957	70	Inlet
80.00	80.00	3.183	3.92	1.05	1.02	2.71	3.36	0.84	1.053	80	Inlet
90.00	90.00	3.581	4.22	1.15	1.29	2.88	3.44	0.93	1.149	90	Inlet
100.00	100.00	3.979	0	1.20	1.59	3.03	3.52	1.03	1.196	100	Inlet
110.00	110.00	4.377	0	1.29	1.92	3.18	3.59	1.13	1.293	110	Inlet
120.00	120.00	4.775	0	1.40	2.29	3.3	3.65	1.23	1.400	120	Inlet
130.00	130.00	5.173	0	1.52	2.69	3.42	3.71	1.35	1.515	130	Inlet
140.00	140.00	5.570	0	1.64	3.12	3.52	3.76	1.47	1.640	140	Inlet
150.00	150.00	5.968	0	1.77	3.58	3.6	3.80	1.59	1.774	150	Inlet
160.00	160.00	6.366	0	1.92	4.07	3.67	3.84	1.73	1.918	160	Inlet
170.00	170.00	6.764	0	2.07	4.59	3.73	3.87	1.86	2.070	170	Inlet
180.00	180.00	7.162	0	2.23	5.15	3.78	3.89	2.01	2.232	180	Inlet
190.00	190.00	7.560	0	2.40	5.74	3.82	3.91	2.16	2.403	190	Inlet
200.00	200.00	7.958	0	2.58	6.36	4	4.00	2.34	2.583	200	Inlet
210.00	210.00	8.356	0	2.77	7.01	4	4.00	2.50	2.773	210	Inlet
220.00	220.00	8.754	0	2.97	7.69	4	4.00	2.67	2.972	220	Inlet
230.00	230.00	9.151	0	3.18	8.41	4	4.00	2.85	3.180	230	Inlet
240.00	240.00	9.549	0	3.40	9.16	4	4.00	3.04	3.397	240	Inlet
250.00	250.00	9.947	0	3.62	9.94	4	4.00	3.23	3.623	250	Inlet
260.00	260.00	10.345	0	3.86	10.75	4	4.00	3.44	3.859	260	Inlet

INPUT FILE LISTING

```
T1  DETENTION BASIN #3A & 3B DESIGN
T2  FILENAME: PEPPER.DAT
T3  2 - 10' X 10' BOX & PEPPERIDGE TRANSITION PROFILE  -  100 YEAR FLOWS
SO   3881.34  36.89  1                          44.6
TS   3932.60  37.12  2      .013
WX   3932.60  37.12  3      .013
R    4088.96  37.89  3      .013
WE   4088.96  37.89  2      .013
TS   4169.37  44.46  4      .013
R    4189.37  44.47  4      .030
SH   4189.37  44.47  4
```

SP

WATER SURFACE PROFILE - CHANNEL DEFINITION LISTING

CARD CODE	SECT NO	CHN TYPE	NO OF PIERS	AVE PIER WIDTH	HEIGHT 1 DIAMETER	BASE WIDTH	ZL	ZR	INV DROP	Y(1)	Y(2)	Y(3)	Y(4)	Y(5)	Y(6)	Y(7)	Y(8)	Y(9)	Y(10)
CD	1	2	0	.00	7.00	40.00			.00										
CD	2	2	0	.00	10.00	21.00			.00										
CD	3	3	1	.92	10.00	20.92	.00	.00	.00										
CD	4	1	0	.00	4.00	70.00	4.00	4.00	.00										

WATER SURFACE PROFILE - TITLE CARD LISTING

HEADING LINE NO 1 IS -

DETENTION BASIN #3A & 3B DESIGN

HEADING LINE NO 2 IS -

FILENAME: PEPPER.DAT

HEADING LINE NO 3 IS -

2 - 10' X 10' BOX & PEPPERIDGE TRANSITION PROFILE - 100 YEAR FLOWS

WATER SURFACE PROFILE - ELEMENT CARD LISTING

```
ELEMENT NO   1 IS A SYSTEM OUTLET     *         *     *
                U/S DATA   STATION    INVERT  SECT                                W S ELEV
                            3881.34   36.89    1                                   44.60

ELEMENT NO   2 IS A TRANSITION        *         *     *
                U/S DATA   STATION    INVERT  SECT              N
                            3932.60   37.12    2              .013

ELEMENT NO   3 IS A  WALL  EXIT                       *
                U/S DATA   STATION    INVERT  SECT
                            3932.60   37.12    3

ELEMENT NO   4 IS A REACH             *         *     *
                U/S DATA   STATION    INVERT  SECT              N                                RADIUS  ANGLE  ANG PT  MAN H
                            4088.96   37.89    3              .013                                 .00    .00     .00     0

ELEMENT NO   5 IS A  WALL  ENTRANCE                   *
                U/S DATA   STATION    INVERT  SECT             FP
                            4088.96   37.89    2              .013

ELEMENT NO   6 IS A TRANSITION        *         *     *
                U/S DATA   STATION    INVERT  SECT              N
                            4169.37   44.46    4              .013

ELEMENT NO   7 IS A REACH             *         *     *
                U/S DATA   STATION    INVERT  SECT              N                                RADIUS  ANGLE  ANG PT  MAN H
                            4189.37   44.47    4              .030                                 .00    .00     .00     0

ELEMENT NO   8 IS A SYSTEM HEADWORKS                  *                       *
                U/S DATA   STATION    INVERT  SECT                                W S ELEV
                            4189.37   44.47    4                                    .00
NO EDIT ERRORS ENCOUNTERED-COMPUTATION IS NOW BEGINNING

** WARNING NO. 2 ** - WATER SURFACE ELEVATION GIVEN IS LESS THAN OR EQUALS INVERT ELEVATION IN HDWKDS, W.S.ELEV = INV + DC
```

WATER SURFACE PROFILE LISTING
DETENTION BASIN #3A & 3B DESIGN
FILENAME: PEPPER.DAT
2 - 10' X 10' BOX & PEPPERIDGE TRANSITION PROFILE - 100 YEAR FLOWS

STATION / L/ELEM	INVERT ELEV / SO	DEPTH OF FLOW	W.S. ELEV	Q	VEL (ft/sec)	VEL HEAD / SF AVE	ENERGY GRD.EL. / HF	SUPER ELEV	CRITICAL DEPTH / NORM DEPTH	HGT/ DIA	BASE/ ID NO.	ZL / ZR	NO PIER	AVBPR
3881.34	36.89	7.71	44.60	40' 1590.0	~~5.16~~ = 1590/308.4	41	45.01	.00	3.66	7.00	40.00	.00	0	.00
TRANS STR	.00449				14.56							.00		
3932.60	37.12	4.85	41.97	20' 1590.0	~~15.63~~	3.79	45.76	.00	5.63	10.00	21.00	.00	0	.00
WALL EXIT					(7.6 ft/sec							.00		
3932.60	37.12	5.22	42.34	20' 1590.0	15.24	3.61	45.95	.00	5.81	10.00	20.92	.00	1	.92
WALL EXIT												.00		
3932.60	37.12	5.22	42.34	20' 1590.0	15.24	3.61	45.95	.00	5.81	10.00	20.92	.00	1	.92
156.36	.00493					.00526	.82		5.28			.00		
4088.96	37.89	5.10	42.99	20' 1590.0	15.60	3.78	46.77	.00	5.81	10.00	20.92	.00	1	.92
WALL ENTRANCE												.00		
4088.96	37.89	4.15	42.04	72' 1590.0	18.23	5.17	47.21	.00	5.63	10.00	21.00	.00	0	.00
TRANS STR	.08171					.00394	.32					.00		
4169.37	44.46	~~2.40~~	46.86	~~80'~~ 72 1590.0	~~8.31~~	1.07	47.94	.00	2.40	4.00	70.00	4.00	0	.00
.74	.00050	1.60			12.77 ft/sec	.00956	.01		5.68			4.00		
4170.11	44.46	2.51	46.97	1590.0	7.92	.98	47.94	.00	2.40	4.00	70.00	4.00	0	.00
2.53	.00050					.00826	.02		5.68			4.00		
4172.64	44.46	2.62	47.08	1590.0	7.55	.89	47.96	.00	2.40	4.00	70.00	4.00	0	.00
4.80	.00050					.00714	.03		5.68			4.00		
4177.43	44.46	2.73	47.19	1590.0	7.20	.81	48.00	.00	2.40	4.00	70.00	4.00	0	.00
7.66	.00050					.00618	.05		5.68			4.00		
4185.10	44.47	2.85	47.31	1590.0	6.87	.73	48.05	.00	2.40	4.00	70.00	4.00	0	.00
4.27	.00050					.00556	.02		5.68			4.00		

How are these W.S.E. calculated

V = Q/A

= 308

WATER SURFACE PROFILE LISTING
DETENTION BASIN #3A & 3B DESIGN
FILENAME: PEPPER.DAT
2 - 10' X 10' BOX & PEPPERIDGE TRANSITION PROFILE - 100 YEAR FLOWS

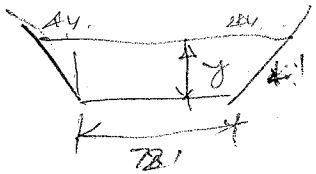
STATION	INVERT ELEV	DEPTH OF FLOW	W.S. ELEV	Q	VEL	VEL HEAD	ENERGY GRD.EL.	SUPER ELEV	CRITICAL DEPTH		HGT/ DIA	BASE/ ID NO.	ZL	NO PIER	AVBPR
L/ELEM	SO					SF AVE	HF			NORM DEPTH			ZR		
4189.37	44.47	2.90	47.37	1590.0	6.73	.70	48.07	.00	2.40		4.00	70.00	4.00	0	.00
.00	.00050					.00270	.00			5.68			4.00		
4189.37	44.47	2.90	47.37	1590.0	6.73	.70	48.07	.00	2.40		4.00	70.00	4.00	0	.00

$Q = V \cdot A$

$V = \frac{Q}{A}$ $A = \frac{1590}{6.73} = 236.25$

$\frac{1590}{(70 \times 2.9 + \frac{1}{2}(2.9 \times 4) \times 2)}$

$AR^{2/3} = \frac{Q \times n}{1.486 \times \sqrt{S}} = \frac{1590 \times .013}{1.486 \times \sqrt{.0820}} = 48.6$ OR 93.4

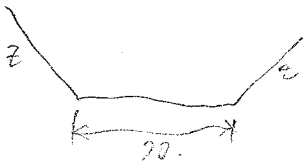


$\frac{A^{1.67}}{P^{0.67}} = \frac{(72y + 4y^2)^{1.67}}{(72 + 8y)^{0.67}} =$

$y = 2.9$, $x = 453.53$

Velocity may be higher than this because, steep slope.

$A = (72 \times y + 4y^2)$
$P = (72 + 8 \times y)$



$72xy + 4y^2 = 236.26$

$70 \times d + \frac{1}{2}(2 \times d) \times 2$

$70 \times d + \frac{4}{2} \times d$

$(70 + 4) d = 236.25$

$d =$

$A = 236.26$
$D = 81.46$

$4y^2 = 236.26 - 72 \times 2.9$

$x = \frac{27.46}{(2.9)^2} = 3.26$

STEPPED SPILLWAY ANALYSIS:

INPUT DATA:

Q =	1590 cfs	Cf =	0.18
Width,W =	40 ft.		
Step ht. h =	1.5 ft.		
Step length,l =	3.75 ft.		

CALCULATED DATA:

Unit Disch, q =	39.75 cfs/ft	Yc =	3.67 f
So = sin alpha=	0.3714	Yc/h =	2.45
Depth, yo =	2.28 ft.	Regime =	Skimming
Vel. Vo =	17.42 fps		
Energy E =	6.99 ft	Froude #=	2.03

Hydraulic Jump:

Conj. Depth,Y2=	5.51 ft.	Length,L=	26.80 f

Appendix. References

Gill, M. (1977). "Routing of floods in river channels." *Nordic Hydr.*, 8, 163–170.

Heggen, R. (1984). "Univariate least squares Muskingum flood routing." *Water Res. Bull.*, 20(1), 103–107.

O'Donnell, T. (1985). "A direct three-parameter Muskingum procedure incorporating lateral inflow." *Hydro. Sci. J.*, 30(4), 479–496.

Skimming Flow in Stepped Spillways

By N. Rajaratnam,[1] Member, ASCE

Introduction

In a stepped spillway, the spillway face is provided with a series of steps, from near the crest to the toe. The energy dissipation caused by the steps reduces the size of the energy dissipator, generally provided at the toe of the spillway. Stepped spillways have been built in the past but there appears to be renewed interest in them because of significant cost savings. A recent example is the Monksville dam (Sorensen 1985).

Based on the experimental observations of Essery and Horner (1971) and Sorensen (1985), the flow over stepped spillways can be divided into nappe flow and skimming flow regimes [Figs. 1(*a*) and (*b*)]. In the nappe flow regime, the flow from each step hits the step below as a falling jet, with the energy dissipation occurring by jet breakup in air, jet mixing on the step, with or without the formation of a partial hydraulic jump on the step. In the skimming flow regime, the water flows down the stepped face as a coherent stream, skimming over the steps and cushioned by the recirculating fluid trapped between them. The energy dissipation in the flow appears to be enhanced by the momentum transfer to the recirculating fluid. In this note, a method for predicting the shear stress, thus frictional energy loss of the skimming flow is presented.

Analysis of Skimming Flow

For a stepped spillway of constant slope, $S_0 = \sin\alpha$, with a large number of identical steps of height, h, and (horizontal) length, l (note that $\sin\alpha = h/\sqrt{l^2 + h^2}$), the flow is assumed to become fully developed after the first few steps. For such a fully developed flow, with a constant mean velocity of V_0 and normal depth of y_0, considering unit width of the spillway, we write

$$y_0\gamma \sin\alpha = \tau \qquad (1)$$

where γ = the weight per unit volume of the fluid (water); and τ = the average Reynolds shear stress that exists between the skimming stream and the recirculating fluid underneath. Let us assume

$$\tau = c_f \frac{\rho V_0^2}{2} \qquad (2)$$

where c_f = the coefficient of fluid friction and would be equal to $f/4$, where f = the Darcy-Weisbach friction factor; and ρ = the mass density of the

[1]Prof., Dept. of Civ. Engrg., Univ. of Alberta, Edmonton, Alberta, Canada, T6G 2G7.

Note. Discussion open until September 1, 1990. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on July 11, 1988. This paper is part of the *Journal of Hydraulic Engineering*, Vol. 116, No. 4, April, 1990. ©ASCE, ISSN 0733-9429/90/0004-0587/$1.00 + $.15 per page. Paper No. 24515.

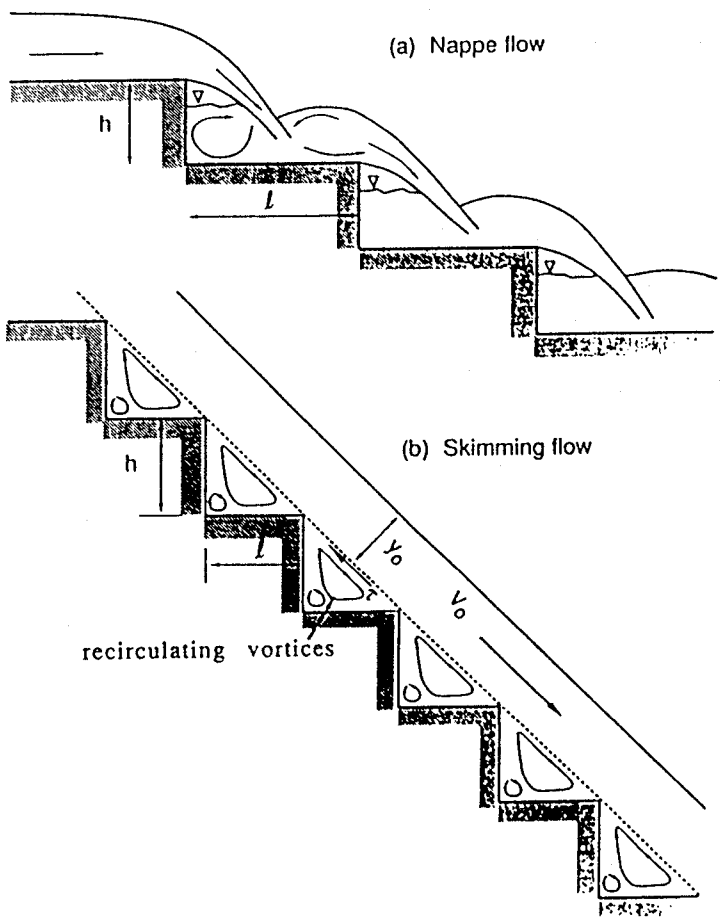


FIG. 1. Definition Sketches for Flow Over Stepped Spillways

fluid. Combining Eqs. 1 and 2, we obtain

$$c_f = \frac{2y_0^3 g \sin \alpha}{q^2} \quad (3)$$

where g = the acceleration due to gravity; and q = discharge per unit width of the spillway.

The idea of Eq. 2 is to represent the turbulent shear stress between the main stream and the recirculating fluid, trapped between the steps of the spillway. For example, this method has been used in developing flow equations for Denil, Vertical Slot, and other fishways (Rajaratnam and Katopodis 1984; Rajaratnam et al. 1986; Rajaratnam et al. 1988). For a Denil fishway, with the depth of flow much larger than its width, so that the shear comes mainly from the sides and for subcritical flows, c_f has been found to be equal to about 0.09 and for relatively smaller depths, values of c_f have been found

to be as large as 0.6. The Reynolds number $R = V_0 y_0/\nu$, with ν being the kinematic viscosity of the fluid, was in the range of 10^5–10^6 for the prototype and equal to about 10^5 for the 1/3 scale model of the Denil fishway. For pool and weir fishways, c_f has been found to be about 0.09. Ervine and Baird (1982) used the same idea to express the apparent shear between the main channel and flood-plain channel flows and found a value of 0.05 for c_f. The slopes for Denil fishways had a maximum value of about 0.3 whereas for the compound channel of Ervine and Baird, they were very small. The point is that the idea of Eq. 2 appears to be generally valid for the range of slopes studied so far and is used for stepped spillway only to attempt to estimate its flow characteristics. It should be pointed out that for smooth boundaries, the coefficient of skin friction decreases from about 0.0035 to 0.0025 as R increases from 10^5 to 10^6.

To evaluate c_f for skimming flow in a stepped spillway, the experimental observations of Sorensen on a 1:25 scale model of the new Monksville dam spillway of approximate height of 32 m are used. Steps at the crest were fitted to the standard Waterways Experiment Station (WES) spillway profile. But for the small number of steps near the crest, the height h of the remaining steps was 0.6 m with a spillway slope of 1 (vertical) on 0.78 (horizontal). The horizontal length, l, of the steps was 0.47 m, thus giving a value of 1.28 for h/l.

For experiments (C1–C8), c_f was found to vary from 0.11 to 0.2 with an average value of 0.18. For experiments C9 and C10, with very small flow rates, c_f was found to be 0.25 and 0.28, respectively, and it is possible that in these experiments the flow was in a transition state between nappe and skimming regimes. It is realized that for the C-series, Sorensen measured the flow depths downstream of the toe and on the stepped spillway the flow would have been aerated. Sorensen also admits to 10–15% error in depth measurements between continuity calculations and stagnation tube measurements. Aeration aspects are also important. Hence, calculations of c_f and energy loss on the spillway (presented later) would have to be considered very approximate. The average value of 0.18 for the skimming flow in the stepped spillway studied by Sorensen, is about twice as large as the value for a Denil fishway with a relatively large depth of flow. For a vertical slot fishway, c_f was found to be equal to about 0.14 for a 1/16 scale model with R equal to about 5,000. For a pool and weir fishway, $c_f \simeq 0.09$ in the streaming flow regime (Rajaratnam et al. 1988). Considering all these values including 0.05 (Ervine and Baird 1982), c_f is the range of 0.05–0.18 with the Reynolds number in the approximate range of 5,000–10^6. It would be interesting and useful to find c_f for skimming flow stepped spillways for a range of slopes or h/l.

For convenience of use, rewrite Eq. 3 as

$$V_0 = \sqrt{\frac{2}{c_f}} \sqrt{g y_0 S_0} \quad (4)$$

For the discharge per unit width q, write

$$q = \sqrt{\frac{2}{c_f}}\, y_0^{3/2} \sqrt{g S_0} \quad (5)$$

Energy Loss in a Stepped Spillway

An estimate of the energy loss for skimming flow in a stepped spillway is presented using the analysis presented in the previous sections. If E is the energy in the flow at the toe of the stepped spillway

$$E = y_0 + \frac{V_0^2}{2g} \quad (6)$$

$$E = \left(\frac{c_f q^2}{2g \sin \alpha}\right)^{1/3} + \left(\frac{q \sin \alpha}{c_f \sqrt{2g}}\right)^{2/3} \quad (7)$$

If y_0' and V_0' = the corresponding depth and velocity at the toe of a smooth spillway without steps

$$E' = y_0' + \frac{V_0'^2}{2g} \quad (8)$$

and one can write an equation similar to Eq. 7 with c_f' replacing c_f, where c_f' = the coefficient of skin friction for the smooth spillway. Sorensen's tests (B series) on a smooth spillway give a value of 0.0065 for c_f'. If ΔE is defined as

$$\Delta E = E' - E \quad (9)$$

ΔE gives the energy loss caused by the steps over that caused by the smooth spillway face. If the relative energy loss is defined as $\Delta E/E'$, it can be shown that

$$\frac{\Delta E}{E'} = \frac{(1 - A) + \frac{F_0'^2}{2}\frac{(A^2 - 1)}{A^2}}{1 + \frac{F_0'^2}{2}} \quad (10)$$

where $A = (c_f/c_f')^{1/3}$; and F_0' = the Froude number at the toe of the smooth spillway. Taking $c_f \simeq 0.18$ and $c_f' \simeq 0.0065$, $A \simeq 3$ and for a relatively large value of F_0', $\Delta E/E'$ is approximately equal to $(A^2 - 1)/A^2$, which further reduces to 8/9. This indicates the considerable amount of energy loss that can be produced by steps, as was found by Sorensen.

Onset of Skimming Flow

Based on their experimental observations, Essery and Horner (1971) found that for horizontal steps, for the onset of skimming flow, the ratio y_c/l, wherein y_c is the critical depth, increased with h/l from about 0.32 for $h/l = 0.4$ to about 0.69 for $h/l = 0.9$. By a reanalysis of their data, it was found that for the whole range of h/l from 0.4 to 0.9, at the onset of skimming flow, y_c/h was approximately equal to 0.8. This means that for y_c/h greater than 0.8, skimming flow occurs. The observations of Sorensen for $h/l = 1.28$ support this criteria. For y_c/h less than 0.8, one would expect to get nappe flow and in Sorensen's experiments, nappe flow occurred for $y_c/h = 0.16$.

Conclusions

This technical note presents a method of predicting the characteristics of skimming flow on stepped spillways. For a stepped spillway with a slope of 1 vertical on 0.78 horizontal, the fluid friction coefficient c_f was evaluated using the experimental results of Sorensen and found to be about 0.18. An estimate has been made of the energy loss on stepped spillways for skimming flow.

Appendix. References

Ervine, D. A., and Baird, J. I. (1982). "Rating Curves for Rivers with Overbank Flow," *Proc. Instn. of Civ. Engrs., Part 2*, Institute of Civil Engineers, 465–472.

Essery, I. T. S., and Horner, M. W. (1971). "The hydraulic design of stepped spillways." *Report 33*, Constr. Industry Res. and Information Assoc., London, England.

Rajaratnam, N., and Katopodis, C. (1984). "Hydraulics of Denil fishways." *Proc.*, ASCE, 110(9), 1219–1233.

Rajaratnam, N., Katopodis, C., and Mainali, A. (1988). "Plunging and streaming flows in pool and weir fishways." *Proc.*, ASCE, 114(8), 939–944.

Rajaratnam, N., Van der Vinne, G., and Katopodis, C. (1986). "Hydraulics of Vertical Slot fishways." *Proc.*, ASCE, 112(10), 909–927.

Sorensen, R. M. (1985). "Stepped spillway hydraulic model investigation." *Proc.*, ASCE, 111(12), 1461–1472.

Flow Routing," *Journal of the Environmental Engineering Division*, ASCE, Vol. 104, No. 331, Feb., 1978.
21. Muzik, I., "State Variable Model of Overland Flow," *Journal of Hydrology*, Vol. 22, No. 3/4, 1974.
22. Napiorkowski, J. J., Strupczewski, W. G., and Dooge, J. C. I., "Lumped Non-Linear Flood Routing Model and Its Simplication to the Muskingum Model," Rainfall-Runoff Relationship, V. P. Singh, Ed., *Proceedings* of International Symposium on Rainfall-Runoff Modeling, at Mississippi State Univ., May 18–21, 1981.
23. Nash, J. E., and Farell, J. P., "A Graphical Solution of the Flood Routing Equation for Linear Storage Discharge Relations," *Transactions*, American Geophysical Union, Vol. 36, 1955, pp. 319–320.
24. Nash, J. E., "A Note on the Muskingum Method of Flood Routing," *Journal of Geophysical Research*, Vol. 64, 1959, pp. 1053–1056.
25. Overton, D. E., "Muskingum Flood Routing of Upland Streamflow," *Journal of Hydrology*, Vol. 4, 1966, pp. 185–200.
26. Ponce, V. M., and Yevjevich, V., "Muskingum-Cunge Method with Variable Parameters," *Journal of the Hydraulic Division*, ASCE, Vol. 104, No. HY12, Dec., 1978, pp. 1663–1667.
27. Ponce, V. M., "Simplified Muskingum Routing Equation," *Journal of Hydraulic Division*, ASCE, Vol. 105, No. HY1, Jan., 1979, pp. 85–91.
28. Prasad, R., "A Nonlinear Hydrologic Systems Response Model," *Journal of Hydraulic Division*, ASCE, Vol. 93, No. HY4, 1967, pp. 201–221.
29. Singh, V. P., and McCann, R. C., "Some Notes on Muskingum Method of Flood Routing," *Journal of Hydrology*, Vol. 48, 1980, pp. 343–361.
30. Stephenson, D., "Direct Optimization of Muskingum Routing Coefficients," *Journal of Hydrology*, Vol. 41, 1979.
31. Strupczewski, W., and Kundzewicz, Z., "Muskingum Method Revisited," *Journal of Hydrology*, Vol. 48, 1980, pp. 327–342.
32. Strupczewski, W., and Kundzewicz, Z., "Translatory Characteristics of the Muskingum Method of Flood Routing—A Comment," *Journal of Hydrology*, Vol. 48, 1980, pp. 363–368.
33. Tung, Y. K., and Mays, L. W., "State Variable Model of Urban Rainfall-Runoff Process," *Water Resources Bulletin*, AWRA, Vol. 17, Apr., 1981.
34. Venetia, C., "The IUH of Muskingum Channel Reach," *Journal of Hydrology*, Vol. 4, 1969, pp. 185–200.
35. Wilson, E. M., *Engineering Hydrology*, MacMillan, London, England, 1974.
36. Young, P., and Beck, B., "The Modeling and Control of Water Quality in River System," *Automatica*, Vol. 10, No. 5, 1974.

Stepped Spillway Hydraulic Model Investigation

By Robert M. Sorensen,[1] F. ASCE

Abstract: A physical hydraulic model investigation was conducted to evaluate the performance of a stepped overflow spillway. The spillway has a standard ogee profile with continuous steps cut into the spillway face from just below the crest, to the toe. The steps significantly increase the rate of energy dissipation on the spillway face, thus eliminating or greatly reducing the need for a large energy dissipation basin at the spillway toe. Primary objectives of the investigation were to evaluate the effectiveness of the flow transition from the smooth crest profile to the steps, to quantify the energy dissipation on the spillway face, and to define the flow characteristics on the steps. The investigation demonstrated that this stepped spillway is quite effective at dissipating energy and that smooth flow transition from the spillway crest to the stepped face is easily achieved.

Introduction

A stepped spillway has been designed for the new Monksville Dam, which is to be part of the Wanaque South Project being developed by the North Jersey District Water Supply Commission and the Hackensack Water Company. This spillway will be a modification of the Waterways Experiment Station (WES) standard profile for an uncontrolled ogee spillway. At a point just downstream of the spillway crest, steps are designed into the profile so that the envelope of their tips follows the standard profile down to the toe of the spillway. The steps significantly increase the rate of energy dissipation taking place on the spillway face and eliminate or greatly reduce the need for a large energy dissipation basin at the toe of the spillway. The performance of this type of spillway has not been extensively investigated so the designers, O'Brien and Gere Engineers, Inc., contracted with the Hydraulics Division, Fritz Engineering Laboratory, Lehigh University, to conduct hydraulic model investigations of the proposed design.

The stepped spillway concept is not new; a stepped spillway was used on the New Croton Dam built in 1892–1906 (8). However, the writer is not aware of any model or prototypical hydraulic investigations done on this type of spillway prior to 1982. In 1982, results of a Bureau of Reclamation model study of a stepped spillway for the Upper Stillwater Dam (to be constructed in Utah) were published (9). The design cross section for the Upper Stillwater Dam spillway is significantly different from that proposed for the Monksville Dam spillway. Results from the Bureau of Reclamation model study are generally useful for the design of stepped spillways and provided incentive for the use of this concept at Monksville Dam.

[1]Prof. of Civ. Engrg., Lehigh Univ., Bethlehem, PA 18015.

Note.—Discussion open until May 1, 1986. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 16, 1984. This paper is part of the *Journal of Hydraulic Engineering*, Vol. 111, No. 12, December, 1985. ©ASCE, ISSN 0733-9429/85/0012-1461/$01.00. Paper No. 20220.

The Monksville Dam spillway will have a crest elevation of 400 ft (121.9 m) and a toe elevation that will vary irregularly between 280 ft (85.3 m) and 310 ft (94.5 m). Its crest width will be 200 ft (61 m). The design discharge per foot of spillway crest for energy dissipation considerations is 65 sq ft/sec (6 m^2/s) and the probable maximum flood discharge, used to determine the spillway profile, is 100 sq ft/sec (9.3 m^2/s). For these two flow rates, the reservoir heads above the spillway crests are 6.3 ft (1.9 m) and 8.6 ft (2.6 m), respectively.

During the preliminary design phase, alternate means of providing energy dissipation for these spillway flows were considered. They include:

1. A standard hydraulic jump stilling basin, which would dissipate a sufficient percentage of the flow energy. However, since the water surface elevation at the spillway toe commonly will fluctuate by 20 ft (6.1 m) or more, extensive rock excavation would be required to set the basin floor at the proper elevation or a large basin end sill would have to be constructed.
2. A 50 ft (15.2 m) wide flip bucket at the toe to deflect the flow sufficiently far from the toe. A resulting jet velocity of about 75 ft/sec (22.9 m/s) would develop a significant scour hole. Design of a successful spillway chute that converges from 200 ft (61 m) to 50 ft (15.2 m) in width and conveys highly super-critical flow would be difficult.
3. A properly designed and constructed stepped ogee spillway should dissipate sufficient energy to not require a stilling basin. It would allow full aeration of spillway flows and should be aesthetically pleasing for typical flow rates over the spillway. The estimated construction cost for this third alternative offers substantial savings over the first two alternatives.

The objective of this study was to investigate the specific stepped spillway design proposed by the design engineers. Primary concerns included:

1. The effectiveness of flow transition from the smooth surface profile at and below the spillway crest, to the first few steps on the spillway face.
2. The amount of energy dissipation in the flow over the stepped spillway and resulting flow velocities at the spillway toe.
3. Training wall heights required to contain the flow along the stepped spillway face.

The specific spillway design being investigated is basic in that it is a straightforward modification of a standard ogee spillway profile. Results of this investigation, which addrtess the aforementioned concerns and indicate that this stepped spillway design is effective, are thus of general value to spillway designers.

Spillway Design Profile

The WES standard spillway profile (see Ref. 2, p. 364) was determined for the design flow conditions (probable maximum flood) and then steps

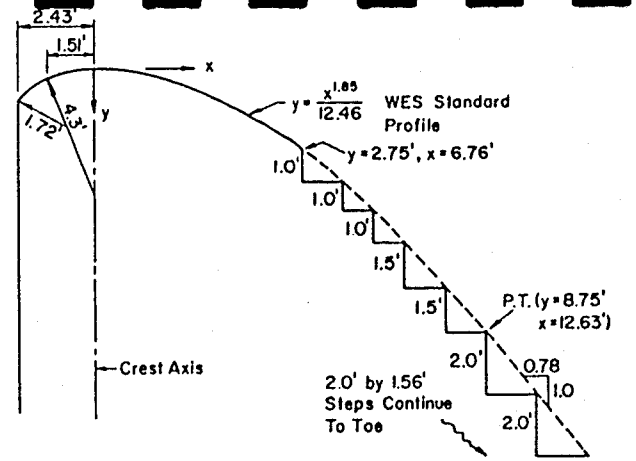


FIG. 1.—WES Standard Profile and Stepped Profile, Upper Portion of Spillway (Prototypical Dimensions)

were fitted to this profile. The standard profile from the upstream face to the point of tangency on the spillway face, which depends on both the design head and the upstream face slope, is shown in Fig. 1. Below the point of tangency the spillway profile has a 0.78 H:1V slope.

Owing to construction techniques for the concrete dam and spillway, the designers decided to use 2 ft (0.61 m) vertical steps below the initial few transition steps. Fig. 1 also shows the resulting step geometry on the face of the upper portion of the spillway. Below the point of tangency, 2 ft by 1.56 ft (0.61 m by 0.48 m) steps continue down to the toe. Above the point of tangency, step sizes decrease in transition to the standard nonstepped ogee profile.

Experimental Setup

The experimental phase of this study consisted of tests on three two-dimensional sectional models of the proposed spillway. Model scales of 1:10 and 1:25 were used.

Similitude Requirements.—Flow over a spillway involves significant horizontal and vertical components of velocity and acceleration. Thus, a spillway model should be built to undistorted scale. Since gravity forces dominate, Froude similarity criteria define model/prototype scale relationships.

Bureau of Reclamation (3) spillway models for large dams have typically been constructed to scale ratios between 1:30 and 1:100. The Bureau recommends that medium-size spillway models not be smaller than a 1:60 scale. Sharp (7) concurs with this recommendation. Thus, the 1:10 and 1:25 scale models used in this study quite well satisfy these recommended minimum scale requirements. The surface roughness of a concrete spillway is relatively small so the models were built of plexiglas.

The Bureau of Reclamation recommends that the flow depth over the crest of a spillway model be at least 75 mm (0.246 ft) at the design normal operating range to reduce effects of viscosity and surface tension. They also recommend, for two-dimensional spillway models, that the model crest width be at least 150 mm (0.492 ft). These recommendations have been satisfied in this study.

For Froude similarity, the following scale relationships must hold:

$$q_r = L_r^{3/2} \quad (1)$$

$$v_r = L_r^{1/2} \quad (2)$$

where L_r is the model/prototype length ratio; q_r is the discharge per unit width of spillway ratio; and v_r is the velocity ratio. For the 1:10 and 1:25 scale ratios used in this study, this yields discharge ratios of 1:31.6 and 1:125, respectively, and velocity ratios of 1:3.16 and 1:5, respectively.

Flume.—Fig. 2 shows schematic plan and profile sections of the test flume. An 8-in. (20.3-cm) pipe with a butterfly valve for adjusting the flow rate provided flow to the head box. Flow from the tailbox dropped into a 5-ft (1.52-m) diam, 6-ft (1.83-m) high steel cylindrical volumetric tank on the floor below. A flexible pipe from the tailbox allowed direction of the flow into the tank for a flow rate measurement or directly into the sump for recirculation by the pump. The maximum discharge through the flume was limited by the capacity of the tailbox to just under 3 cfs (0.085 m^3/s). Flow in the head box passed two baffles before entering the narrower (1-ft wide) test section. The first head box baffle, made of plywood drilled with 1-1/2-in. (38.1-cm) diam holes, smoothed the flow. The second and shorter baffle consisting of a thin aluminum plate with fine holes was installed primarily to dampen surface waves.

There were two model test bays, each having a plywood back wall painted white and a clear plexiglas front wall. The clear viewing sections of each were 45 in. by 70 in. (114 cm by 178 cm) and 42 in. by 52

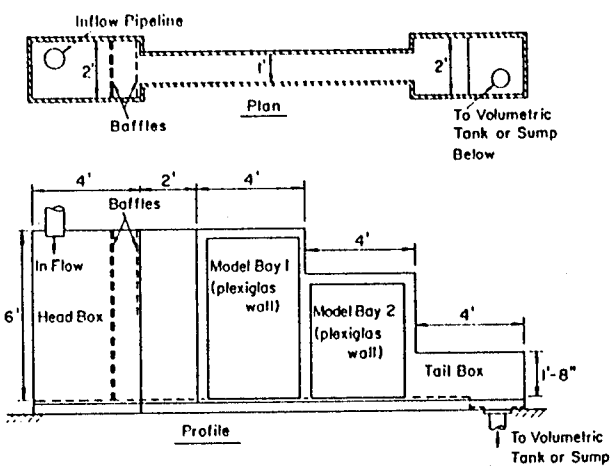


FIG. 2.—Schematic Plan and Profile of Model Test Flume

in. (107 cm by 132 cm), respectively. The first and smaller model was installed in model bay 2; the larger second and third models were placed primarily in bay 1 and extended into bay 2.

Spillway Models.—Three cross-sectional plexiglas spillway models were tested. Specifically, they were:

1. Model A—A 1:10 scale model of the upper 22.75 ft (6.9 m) of the spillway. The model extended down to seven steps below the point of tangency. Model A was used to evaluate the flow transition from the spillway crest to the first several steps and the nature of flow over the steps, for the range of spillway discharges of interest.
2. Model B—A 1:25 scale model of the standard WES ogee spillway profile used as the basis for the stepped spillway. It was a model of the entire 120 ft (36.6 m) spillway profile. This model was briefly tested to provide comparison data for the full stepped spillway profile model (model C).
3. Model C—A 1:25 scale model of the entire stepped spillway profile. Tests with model C were primarily to evaluate energy dissipation in the flow over the stepped spillway and anticipated flow depths along the spillway, to establish training wall dimensions.

Measurements.—The water surface elevation upstream of the spillway crest was measured by a point gage that could be read to the nearest thousandth of a foot. The gage was located 1 ft (0.30 m) upstream of the crest for model A and 0.4 ft (0.12 m) upstream of the crest for models B and C.

The volumetric tank used to measure the flow rate in the system had a capacity of over 115 cu ft (3.26 m^3) and could be read to the nearest half cubic foot. Flow rate measurements were probably accurate to within ±2%. A few repeat flow rate measurements made during the testing program were within these limits.

During each test run with models A and C, color photographs of flow conditions were taken. Also, for models A and C, flow conditions between zero flow and the peak flow at which the model was run, were recorded on video tape.

A scale was used to measure the vertical water depth on the spillway in model C, at the spillway crest and at the tips of steps 6, 15, 24, 33, 42, 51 and 59 (toe). When air-entrained flow was encountered, a conservative flow depth that included the bulk of the flow was recorded.

During the tests with models B and C, at least six depth measurements were made at equally spaced intervals across the toe of the structure. In model B, these measurements were made right at the toe while in model C they were made on the horizontal slope downstream of the toe where air entrainment had significantly diminished. With the measured flow rate and the depth measurements, average flow velocities could be calculated from continuity. As a check on this method of determining flow velocities, some velocity measurements were made with a stagnation tube. These measurements yielded results within 10–15% of the values calculated from continuity.

Tests and Results

Test Conditions

Table 1 lists the discharge and related upstream head (upstream surface elevation minus spillway crest elevation) for each test run for the three spillway section models. The head-discharge data are plotted in Fig. 3. The rating curve for the line fit by eye to model A (the larger scale model) data is

TABLE 1.—Model Test Data—Discharge, Upstream Head and Toe Velocity

Model A			Model B				Model C			
Run number (1)	Discharge (cfs/ft) (2)	Head (ft) (3)	Run number (4)	Discharge (cfs/ft) (5)	Head (ft) (6)	Toe velocity (fps) (7)	Run number (8)	Discharge (cfs/ft) (9)	Head (ft) (10)	Toe velocity (fps) (11)
A-1	2.53	0.710	B-1	1.56	0.432	21.3	C-1	1.20	0.413	7.5
A-2	2.27	0.658	B-2	1.14	0.398	17.6	C-2	1.06	0.377	6.6
A-3	1.96	0.626	B-3	0.78	0.321	16.7	C-3	0.89	0.333	7.4
A-4	1.90	0.621	B-4	0.77	0.315	16.9	C-4	0.65	0.283	6.7
A-5	1.83	0.586	B-5	0.70	0.300	16.5	C-5	0.55	0.255	5.9
A-6	1.30	0.476	B-6	0.57	0.267	15.7	C-6	0.40	0.213	4.7
A-7	0.96	0.399	B-7	0.47	0.240	17.9	C-7	0.28	0.168	4.4
A-8	0.55	0.275	B-8	0.33	0.195	15.5	C-8	0.20	0.137	3.8
A-9	0.38	0.217	B-9	0.16	0.115	15.2	C-9	0.097	0.080	2.7
A-10	0.056	0.063	B-10	0.071	0.071	10.3	C-10	0.067	0.068	2.5

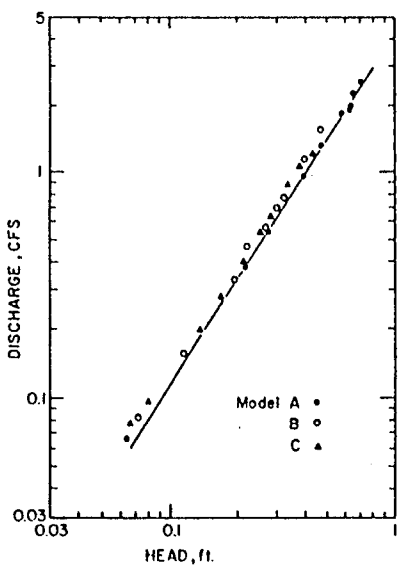


FIG. 3.—Head versus Discharge for each Model

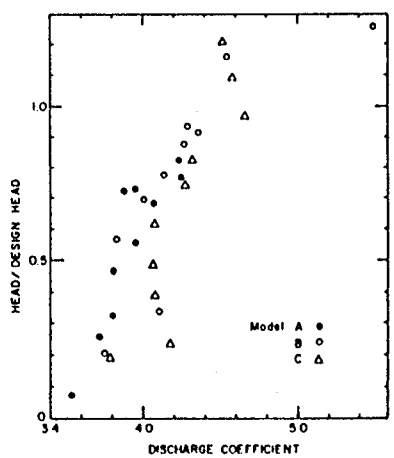


FIG. 4.—Dimensionless Head versus Discharge Coefficient for each Model

$$Q = 4.1\,LH^{1.54} \quad (3)$$

where Q is the discharge in cubic feet per second, for a crest width L and upstream head H in feet. The smaller scale models (B and C) have a head-discharge relationship that yields a slightly higher discharge for a given head.

Fig. 4 is a plot of the discharge coefficient, C (from $Q = CLH^{1.5}$), evaluated for each test run, versus the dimensionless head (head/spillway design discharge head). The data very approximately fit a straight-line relationship with a discharge coefficient that varies from 3.5 to over 5. At the design head the discharge coefficient is approximately 4.4. This range of discharge coefficient values is typical (see Ref. 2, Fig. 14-4), but commonly the discharge coefficient increases at a decreasing rate rather than increasing linearly with increasing head. This suggests that the head measuring station might not have been located far enough from the spillway crest.

Results

Test results are presented in three sections as they address, in order, the three study objectives listed in the introduction.

Flow Transition from Crest to Initial Steps.—The 10 test runs with model A were conducted specifically to determine whether any undesirable disturbance of flow developed in the flow transition from the crest to the stepped portion of the spillway profile. For the range of discharges from the highest model discharge of 2.53 cfs/ft (0.233 $m^3/s/m$) down to Run A-9 with a discharge of 0.38 cfs/ft (0.035 $m^3/s/m$), there was a smooth transition of flow onto the steps. The free surface was smooth down to the point of initiation of air entrainment. This point of air entrainment was located past the end of the spillway section for Runs A-1 to A-6 and moved progressively up from step 10 in Run A-7 to step 4 in Run A-9. For flow rates less than the flow rate in A-9,

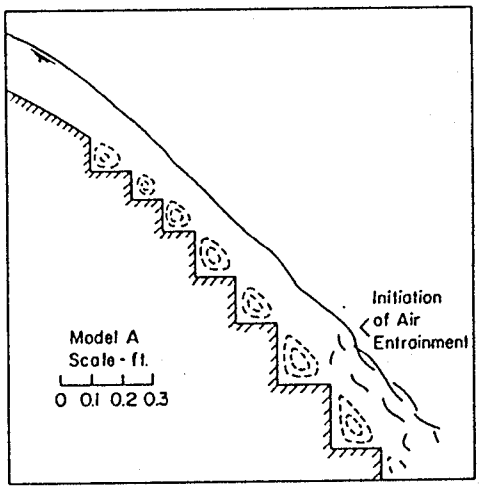


FIG. 5.—Flow Conditions, Run A-8

the air entrainment worked its way up to step 1.

For test runs A-1 to A-9, the typical flow conditions over the steps were as demonstrated in Fig. 5 for Run A-8. At each step, whether air entrainment was occurring or not, a stable rolling vortex developed in the step. The overlying flow moved down the spillway supported by the vortices and tips of the steps. Injection of dye indicated that flow enters a vortex, rotates in the vortex for a brief period and then returns to the main flow to proceed on down the spillway face. At the steps where air entrainment occurs, air bubbles penetrated the vortices and could be seen rotating with the vortex flow.

For Run A-10, which had a very low flow rate of 0.056 cfs/ft (0.0052 $m^3/s/m$), a thin film of water approximately 0.01-ft (0.3-cm) thick flowed off the spillway crest, hit the top of the first step, and deflected outward/downward hitting the spillway face again several steps further down the spillway. The model flow rate in Run A-10 represents a prototypical flow rate of 1.8 cfs/ft (0.16 $m^3/s/m$), which is typical of normal daily summertime flows at the site.

The spillway designers wanted to eliminate this deflecting jet of water so that modifications in the model A profile were made and tested for low model rates of 0.2 cfs/ft (0.019 $m^3/s/m$) and less. These tests indicated that the best way to eliminate the deflecting jet of water was to add a few smaller steps on up the face of the spillway. Fig. 6 shows the

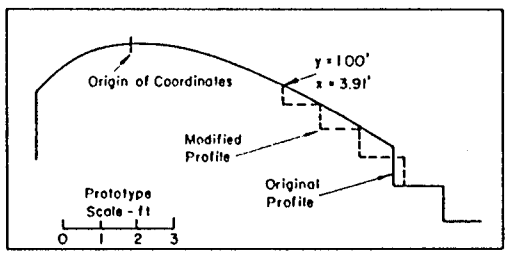


FIG. 6.—Original and Modified Spillway Crest Profiles

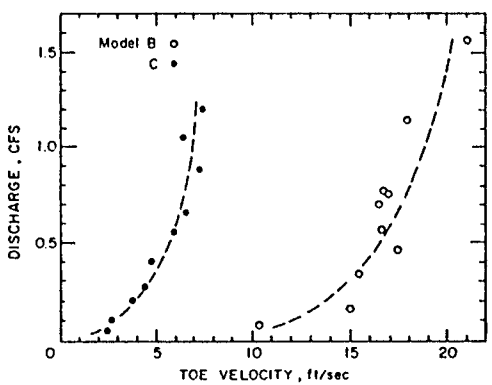


FIG. 7.—Toe Velocities for Models B and C

original and final modified spillway profiles near the crest. Three steps, each having a vertical rise of 0.75 ft (0.23 m) and one step with a rise of 0.5 ft (0.15 m) were added. The worst jet from the new first step only deflected to the new third step, a vertical distance of 1.5 ft (0.46 m) in the prototype.

With extremely low flows in the model of the order of 0.03 cfs/ft (0.0028 $m^3/s/m$), the flow cascades over the steps, falling from step to step in a thin choppy layer that clings to the face of each step.

Thus, with the modified profile, for flow rates from zero up to the probable maximum flood, there is effectively a smooth flow transition from the crest to the spillway steps.

Energy Dissipation.—Table 1 also lists the flow velocities at the spillway toe for models B and C. Fig. 7 is a plot of toe velocity versus discharge for both models. Data scatter is somewhat greater for model B, owing to the smaller depths being measured at the toe. However, the expected trend of increasing toe velocity with increasing discharge is clear and the general amount of velocity reduction caused by the addition of steps is well defined.

For the model probable maximum flood discharge of 0.80 cfs/ft (0.074 $m^3/s/m$), the toe velocity was reduced from approximately 18 fps (5.5 m/s) for the standard spillway to 6.5 fps (2.0 m/s) for the stepped spillway. At the model energy dissipation design discharge of 0.52 cfs/ft (0.048 $m^3/s/m$), the toe velocity was reduced from approximately 16.5 fps (5.0 m/s) to 5.5 fps (1.7 m/s). The model B to model C toe velocity ratios thus were 2.8 and 3.0, respectively, and increased to just over 4 for a low model discharge of 0.1 cfs/ft (0.009 $m^3/s/m$). Consequently, the kinetic energy in the flow at the stepped spillway toe varies from about 12 to 6% of the energy at the standard spillway toe for this range of model discharges.

Training Wall Heights.—Vertical water depth measurements made at the crest and at the tips of selected steps along the spillway for model

TABLE 2.—Depth Measurements and Step at which Air Entrainment Commences—Model C

Run (1)	Crest (2)	Step 6 (3)	Step 15 (4)	Step 24 (5)	Step 33 (6)	Step 42 (7)	Step 51 (8)	Step 59 (9)	Air entrainment step (10)
C-1	0.318	0.308	0.230	0.213	0.230	0.279	0.279	0.295	36
C-2	0.289	0.256	0.187	0.197	0.230	0.262	0.279	0.295	31
C-3	0.249	0.213	0.148	0.171	0.197	0.213	0.213	0.230	25–26
C-4	0.213	0.164	0.131	0.164	0.164	0.180	0.180	0.196	22–23
C-5	0.190	0.144	0.131	0.164	0.180	0.197	0.197	0.213	19
C-6	0.157	0.112	0.131	0.164	0.164	0.180	0.197	0.197	15
C-7	0.119	0.079	0.131	0.148	0.148	0.164	0.164	0.180	11
C-8	0.097	0.052	0.082	0.082	0.082	0.092	0.098	0.115	8–9
C-9	0.056	0.046	0.059	0.059	0.059	0.066	0.066	0.069	6
C-10	0.038	0.046	0.049	0.049	0.049	0.049	0.049	0.049	4

Note: Depths measured vertically, in feet, from tip of step to water surface. Step at which air entrainment starts is listed in last column.

C are listed in Table 2. Also listed for each run is the step at which air entrainment commenced. Typically, the depth decreased as flow descends from the crest to the point at which air entrainment commences. Beyond this point, owing to bulking of the flow by the air entrainment, the depth continually increased toward the spillway toe.

Discussion

The toe velocity and discharge data for the stepped and unstepped spillway models (C and B, respectively) was scaled up to prototypical conditions using the scale ratios given in a previous section. These data are plotted in Fig. 8.

Bradley and Peterka (1) present a chart based on "experience, computation, and a limited amount of experimental information obtained from prototype tests on Shasta and Grand Coulee Dams," that yields a preliminary design estimate of toe velocities for ogee spillways with slopes between $0.6H:1V$ and $0.8H:1V$. Toe velocities calculated using this chart are also plotted in Fig. 8. Velocities scaled from model B test results for the unstepped spillway typically exceed velocities calculated from Ref. 1 by 15–20%. Even at discharges exceeding the probable maximum flood in model B, there was no air entrainment on the spillway face. In the prototype, air entrainment would be expected. This scale effect is likely to be the primary cause for the unstepped spillway velocity differences in Fig. 8. In other words, the model surface roughness and resulting turbulent boundary layer growth and air entrainment were not sufficient to exactly simulate the prototypical conditions.

For the stepped spillway, the scaled steps form the dominant "surface roughness." So with typical model spillway face depth Reynolds numbers in the order of 10^5, scale effects should be less and predicted prototypical toe velocities should be closer (but still probably a bit higher) to true prototypical values.

Thus (see Fig. 7), the prototype stepped spillway designed for Monksville Dam will typically have toe velocities of 30 fps (9.2 m/s) at the probable maximum flood discharge, compared to toe velocities of 75 fps

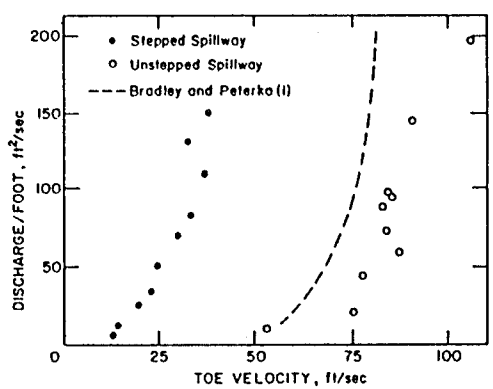


FIG. 8.—Prototypical Toe Velocities for Stepped and Unstepped Spillways

(22.9 m/s) for a prototypical unstepped spillway. This represents a kinetic energy dissipation of 84%, which is as good as or superior to the energy dissipation achieved in a typical well-developed hydraulic jump (see Ref. 2, p. 396).

Morris (5,6) investigated flow past surfaces with large uniformly spaced surface roughness elements and defined three regimes: (1) Isolated roughness flow in which each element generates a wake that diffuses into the main flow before reaching the next element; (2) wake interference flow where the elements are sufficiently close together that the wake extends to the following element; and (3) skimming or quasi-smooth flow in which surface elements are spaced so close as to form a "pseudo-wall' that flow skims over and between which stable depression vortices form. In skimming flow, the vortices are maintained through transmission of shear stress from the fluid flowing past the tips of the elements. In addition, small-scale vorticity will be generated continuously at the tips of the elements. Energy is expended to generate the tip vorticities and to maintain the stable depression vortices. Quasi-smooth flow well defines the conditions observed on the stepped spillway.

Knight and MacDonald (4) investigated subcritical open channel flow resistance caused by roughness elements with square cross sections at 10 uniform spacings including one that caused quasi-smooth flow. Their results, and their evaluation of data from other authors, shed further insight into the rectangular cross section depression dimensions for which quasi-smooth flow will develop, and they recommend an improved form of bed resistance equation for quasi-smooth flow. They also demonstrate that the maximum bed resistance for square resistance elements will occur for the wake interference flow regime.

Much fruitful research can still be done to develop useful information to guide the design of steps for spillway faces, designs that will optimize response to construction and energy dissipation requirements. Specifically, the optimum depression shapes, sizes and spacings for supercritical flow need to be determined.

Conclusion

An ogee spillway with a stepped face can be designed to perform satisfactorily for the full range of discharges encountered by the spillway. A stepped spillway's energy dissipation characteristics are comparable to those of an unstepped spillway with a hydraulic jump stilling basin at the toe.

Research should be conducted to provide additional information necessary to optimize the step geometry for a given spillway discharge, face slope, and crest elevation.

Acknowledgment

This study was funded through a contract with O'Brien and Gere Engineers, Inc., Blue Bell, PA.

G. P. Lennon and R. N. Weisman from the Hydraulics Division assisted with planning of the test setup and procedure and conduct of the tests. Cathy Miller, Nate Beil, Gary Wisniewksi, Tim Kyper, Pete Sit-

kowski, Joe Clifford, Jeff Bell and Jon Sorensen, all from the Hydraulics Division, provided support in numerous ways.

Appendix.—References

1. Bradley, J. N., and Peterka, A. J., "The Hydraulic Design of Stilling Basins: Short Stilling Basin for Canal Structures, Small Outlet Works and Small Spillways (Basin III)," *Journal of the Hydraulics Division*, Vol. 83, No. HY5, Paper 1403, Oct., 1957, 22 pp.
2. Chow, V. T., *Open-Channel Hydraulics*, McGraw-Hill Book Co., Inc., New York, NY, 1959, 680 pp.
3. *Hydraulic Laboratory Techniques*, Water and Power Resource Service, Department of the Interior, Denver, CO, 1980, 208 pp.
4. Knight, D. W., and MacDonald, J. A., "Hydraulic Resistance of Artificial Strip Roughness," *Journal of the Hydraulics Division*, ASCE, Vol. 105, No. HY6, June, 1979, pp. 675–690.
5. Morris, H. M., "A New Concept of Flow in Rough Conduits," *Transactions*, ASCE, Vol. 120, 1955, pp. 373–410.
6. Morris, H. M., "Design Methods for Flow in Rough Conduits," *Transactions*, ASCE, Vol. 126, Part 1, 1961, pp. 454–490.
7. Sharp, J. J., *Hydraulic Modeling*, Butterworth and Co., London, U.K., 1981, 242 pp.
8. Wegman, E., "The Design of the New Croton Dam," *Transactions*, ASCE, Paper 1047, June, 1907, pp. 398–457.
9. Young, M. F., "Feasibility Study of a Stepped Spillway," Hydraulics Division Specialty Conference, Jackson, MS, Aug., 1982, pp. 96–105.

Turbidity Current with Erosion and Deposition

By J. Akiyama[1] and H. Stefan[2]

Abstract: The equations which govern the movement of two-dimensional gradually varied turbidity currents in reservoirs and over beaches are derived and solved numerically. Turbidity currents are sediment-laden gravity currents that exchange sediment with the bed by erosion or deposition as the flow travels over the downslope. Turbidity currents derive this driving force from the sediment in suspension. They experience a resisting shear force on the bed and entrain water from above. Turbidity currents can be eroding or depositive, accelerating or decelerating, dependent on the combination of initial conditions, bed slope, and size of sediment particles. They can be controlled from upstream (supercritical) or downstream (subcritical). Gravity currents with and without erosion and deposition are examined in order to understand the effects of sediment exchange on the flow.

Introduction

Turbidity currents are gravity currents consisting of a sediment-water mixture flowing over a sloping bottom (Fig. 1). Similar gravity currents can be produced by salinity or temperature differences and have then been referred to as inclined plumes or underflows (8,17,18,19,26). In turbidity currents, suspended sediment makes the density of the mixture greater than the density of the ambient water and provides the driving force; the sediment laden flow must generate enough turbulence to hold the sediment in suspension. Uniform or gradually varied turbidity currents with very fine sediment and therefore without erosion or deposition of sediment have been studied by Ashida and Egashira (3), Bonnefille and Goddet (5), and Stefan (25) among others.

Turbidity currents have been observed where inflows carrying a relatively high concentration of suspended material enter lakes (20), reservoirs (11), or the ocean (16). Two types of turbidity currents can be distinguished: (1) Low velocity, low density (5); and (2) high velocity, high density (16). High velocity, high density turbidity currents often carry suspended materials introduced near the shore to the deep sea, and even have enough erosive power to produce submarine canyons (13).

Turbidity currents can be originated by various processes. Discharges of large amounts of sediments, e.g., mine tailings (Silver Bay in Lake Superior), underwater landslides caused by earthquakes (the Grand Banks), and resuspension of suspended materials by waves during storms are three possibilities. Turbidity currents can be erosive or depositional.

[1]Grad. Student, St. Anthony Falls Hydraulic Lab., Dept. of Civ. & Mineral Engrg., Univ. of Minnesota, Minneapolis, MN 55414.

[2]Prof. and Assoc. Dir., St. Anthony Falls Hydraulic Lab., Dept. of Civ. & Mineral Engrg., Univ. of Minnesota, Minneapolis, MN 55414.

Note.—Discussion open until May 1, 1986. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on August 20, 1984. This paper is part of the *Journal of Hydraulic Engineering*, Vol. 111, No. 12, December, 1985. ©ASCE, ISSN 0733-9429/85/0012-1473/$01.00. Paper No. 20216.

NBS LOWRY ENGINEERS & PLANNERS

DATE 9-7-91

JOB NO PT9-086-016

Basin #3A - Pepperidge Channel Inlet Stability Analysis

BY BJF CHK'D

SHEET OF

R.C.C.

Lanes Weighted Creep:

$$C = \frac{\Sigma V + \frac{1}{3}\Sigma H}{H}$$

$\Sigma V = 6 + 7.5 = 13.5'$

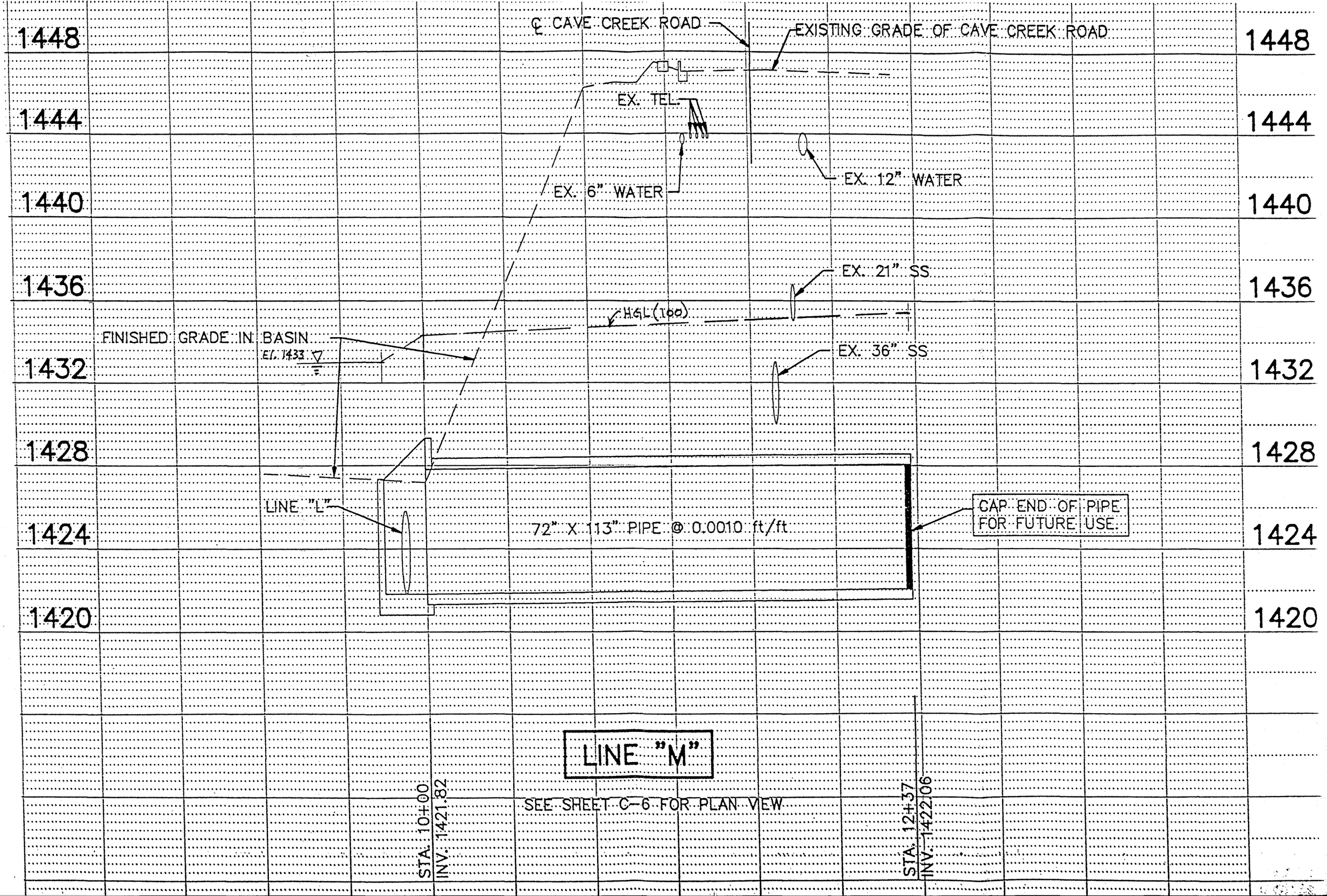
$\Sigma H = 81 + 208 + 48 + 16 = 353$ ft

$H = 1447.4 - 1427.5 = 19.9'$

$$C = \frac{13.5 + \left(\frac{353}{3}\right)}{19.9} = 6.6 : 1 \quad \underline{ok}$$

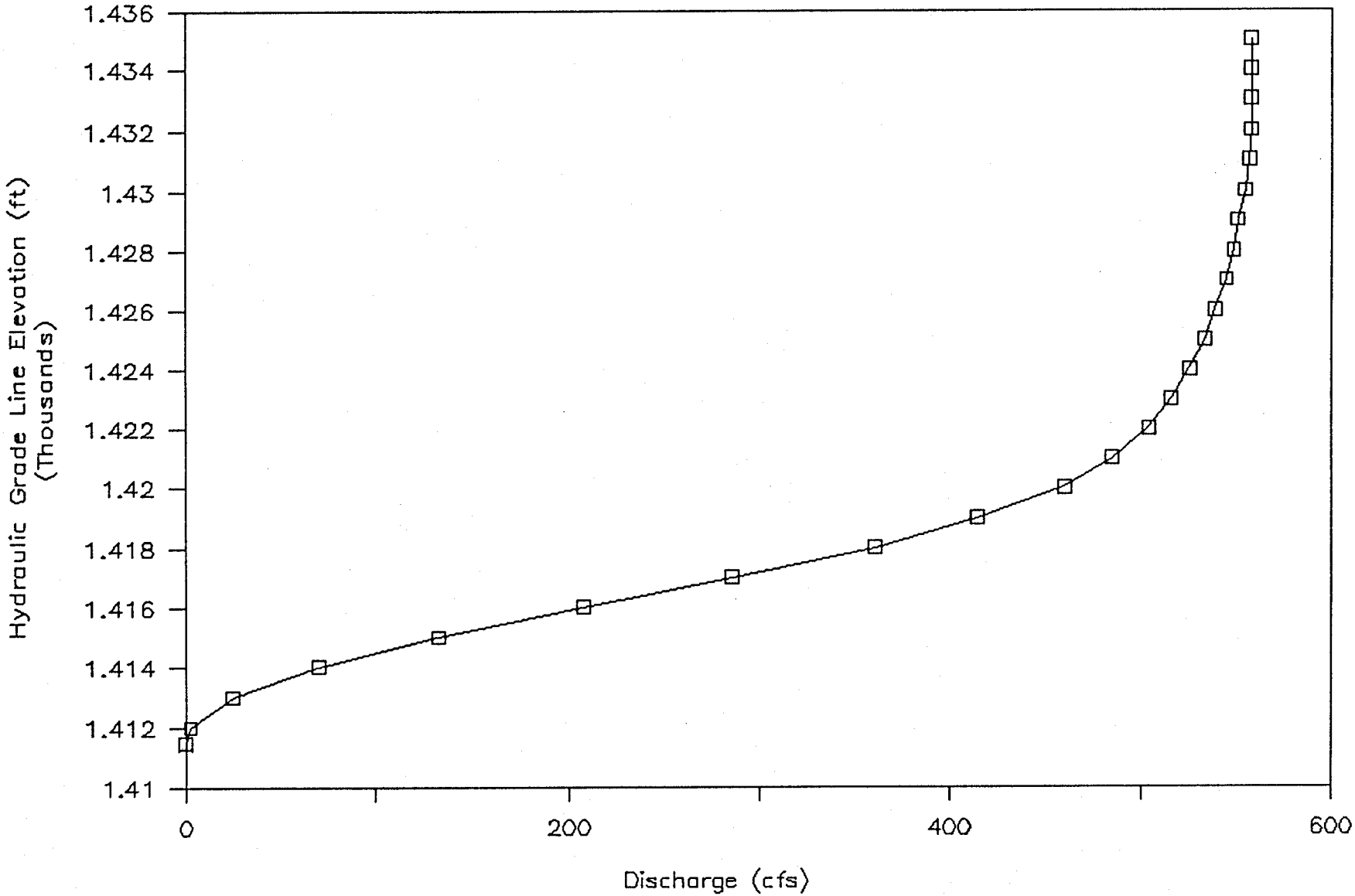
Lane's recommended weighted-creep ratios are:

Material	Ratio
Very fine sand or silt	8.5:1
Fine sand	7.0:1
Medium sand	6.0:1
Coarse sand	5.0:1
Fine gravel	4.0:1
Medium gravel	3.5:1
Coarse gravel including cobbles	3.0:1
Boulders with some cobbles and gravel	2.5:1
Soft clay	3.0:1
Medium clay	2.0:1
Hard clay	1.8:1
Very hard clay or hardpan	1.6:1


1448
1444
1440
1436
1432
1428
1424
1420
℄ CAVE CREEK ROAD
EXISTING GRADE OF CAVE CREEK ROAD
EX. TEL.
EX. 6" WATER
EX. 12" WATER
EX. 21" SS
HGL (100)
FINISHED GRADE IN BASIN
EL. 1433
EX. 36" SS
LINE "L"
72" X 113" PIPE @ 0.0010 ft/ft
CAP END OF PIPE FOR FUTURE USE.
LINE "M"
SEE SHEET C-6 FOR PLAN VIEW
STA. 10+00
INV. 1421.82
STA. 12+37
INV. 1422.06

DETENTION BASIN NO. 3 DESIGN

Basin 3B – TW Rating Curve (108" pipe)



CIRCULAR PIPE RATING TABLE

MANNINGS n = .013
SLOPE = .0011 FT./FT.
DIAMETER = 108 IN.

EL.[FT.]	DISCH [CFS]	AREA [FT^2]	Rh [FT.]	VEL [FPS]
.1	7.843528E-02		.1261907	
.638476E-02	.6215616			
.2	.3494734	.3555498	.1320117	.9829097
.3	.8352627	.6508675	.1969418	1.283307
.4	1.546808	.9985724	.2611726	1.54902
.5	2.490708	1.39068	.3247023	1.791
.6	3.670993	1.8217	.387527	2.015147
.7000001	5.09001	2.287532	.4496445	2.22511
.8000001	6.748866	2.784939	.5110516	2.423344
.9000001	8.647728	3.311272	.5717457	2.611603
1	10.786	3.864295	.6317238	2.791195
1.1	13.16245	4.442084	.6909823	2.963124
1.2	15.77537	5.042961	.7495194	3.128195
1.3	18.62251	5.665418	.8073304	3.287049
1.4	21.70128	6.308112	.8644124	3.440218
1.5	25.00877	6.969824	.9207629	3.58815
1.6	28.5417	7.649433	.9763779	3.731218
1.7	32.29649	8.345898	1.031253	3.869744
1.8	36.2694	9.058269	1.085387	4.004009
1.9	40.45632	9.785642	1.138774	4.134253
2	44.85302	10.52718	1.191411	4.260688
2.1	49.45503	11.28208	1.243295	4.383503
2.2	54.25767	12.04959	1.29442	4.502864
2.3	59.25609	12.829	1.344783	4.618918
2.4	64.4453	13.61962	1.394381	4.731798
2.5	69.8201	14.4208	1.443208	4.841626
2.6	75.37513	15.23189	1.49126	4.948507
2.7	81.10492	16.05231	1.538533	5.052541
2.8	87.00389	16.88146	1.585022	5.153815
2.9	93.06618	17.71876	1.630722	5.25241
2.999999	99.28598	18.56368	1.675629	5.3484
3.099999	105.6571	19.41567	1.719736	5.441849
3.199999	112.1736	20.2742	1.76304	5.532822
3.299999	118.829	21.13878	1.805534	5.621373
3.399999	125.6169	22.00888	1.847213	5.707552
3.499999	132.5308	22.88403	1.888071	5.791408
3.599999	139.564	23.76374	1.928101	5.872981
3.699999	146.7097	24.64753	1.967299	5.95231
3.799999	153.9612	25.53494	2.005657	6.029434
3.899999	161.3112	26.42549	2.043169	6.104379
3.999998	168.7528	27.31874	2.079828	6.17718
4.099999	176.2785	28.21422	2.115627	6.247859
4.199999	183.881	29.11149	2.150557	6.316444
4.299999	191.5529	30.01009	2.184612	6.382951
4.399998	199.2866	30.90959	2.217785	6.447403
4.499998	207.074	31.80952	2.250064	6.509814
4.599998	214.8954	32.70758	2.281445	6.570201
4.699998	222.7669	33.60708	2.311916	6.628573
4.799998	230.6683	34.50568	2.341468	6.684938
4.899998	238.5913	35.40295	2.37009	6.739306
4.999998	246.5273	36.29843	2.397772	6.791681

5.099998	254.4679	37.19167	2.424504	6.842066
5.199998	262.4041	38.08223	2.450273	6.89046
5.299998	270.3271	38.96963	2.475068	6.936866
5.399997	278.2279	39.85343	2.498875	6.981279
5.499997	286.097	40.73314	2.521682	7.023692
5.599997	293.925	41.60829	2.543473	7.064098
5.699997	301.7022	42.47839	2.564234	7.102486
5.799997	309.4187	43.34297	2.58395	7.138846
5.899997	317.0644	44.2015	2.602602	7.173159
5.999997	324.6287	45.05349	2.620172	7.205406
6.099997	332.1011	45.89841	2.636642	7.235569
6.199997	339.4707	46.73572	2.651991	7.263624
6.299997	346.7258	47.56486	2.666195	7.289536
6.399997	353.8551	48.38528	2.679232	7.31328
6.499996	360.8463	49.19638	2.691075	7.334814
6.599996	367.687	49.99755	2.701696	7.354101
6.699996	374.3642	50.78817	2.711064	7.371092
6.799996	380.8645	51.56758	2.719147	7.385736
6.899996	387.1735	52.33509	2.725907	7.397971
6.999996	393.2764	53.08999	2.731304	7.407733
7.099996	399.1578	53.83153	2.735293	7.414944
7.199996	404.8007	54.5589	2.737824	7.419518
7.299996	410.1878	55.27127	2.738842	7.421357
7.399996	415.3	55.96774	2.738283	7.420347
7.499995	420.117	56.64735	2.736076	7.416358
7.599995	424.6164	57.30906	2.732136	7.409238
7.699995	428.7739	57.95175	2.72637	7.398808
7.799995	432.5623	58.57421	2.718663	7.384859
7.899995	435.951	59.17509	2.708882	7.367137
7.999995	438.9047	59.75288	2.696865	7.345332
8.099995	441.3828	60.3059	2.682412	7.319064
8.199995	443.3361	60.83224	2.665269	7.287848
8.299996	444.7045	61.32965	2.645109	7.251052
8.399996	445.4109	61.79548	2.621491	7.207824
8.499996	445.3528	62.2265	2.593793	7.156964
8.599997	444.3834	62.61862	2.561082	7.096665
8.699997	442.2721	62.96632	2.521819	7.023947
8.799998	438.602	63.26164	2.473075	6.933142
8.899998	432.3717	63.49101	2.407463	6.809968
8.999998	414.2171	63.6172	2.250724	6.511088

PIPE FLOWING FULL Q = 414.1293 CFS

NBS LOWRY

ENGINEERS & PLANNERS

2600 North 44th Street
Phoenix, Arizona
85008-1599

602 468-1688 Tel
602 956-4538 Fax

April 24, 1991

Mr. John Fincel, P.E.
City of Phoenix
Engineering Department
125 E. Washington St.
Phoenix, AZ 85004

Re: ST-896829 W.O.#69209, East Fork Cave Creek Detention Basin #3
Preliminary Sediment Analysis Results.

Dear Mr. Fincel:

We have completed the preliminary sediment analysis as described in our Scope of Work. It is our opinion that no significant sediment deposition problems exist for Detention Basin 3 and recommend that a detailed analysis not be conducted.

On February 15, 1991 Dr. Gary Guymon, a special consultant to NBS/Lowry, and myself conducted a field investigation of the area that would potentially deliver sediment to detention basin 3. We considered the primary source of sediment to be the mountain northwest of the intersection of Beardsley and Cave Creek Roads and the easterly slopes of Buffalo Ridge. We took one grab sample of sediment at approximately Rose Garden Lane at the base of the mountain to get a feel for grain size distribution in the area. The other areas tributary to detention basin 3 are mostly developed and shouldn't produce much sediment. Attached is a memo from Dr. Guymon describing his findings and recommendations relative to this investigation.

If you have any questions about our conclusions please give me a call at 468-1688.

Very Truly Yours,

NBS/LOWRY

Brian Fry

Brian J. Fry, P.E.
Project Manager

NBS LOWRY ENGINEERS & PLANNERS

MEMORANDUM

To: Brian Fry
NBS/Lowry - Phoenix

Date: March 4, 1991

Job No.: P79-086-016

From: Gary L. Guymon

Page 1 of 1

Subject: EAST FORK CAVE CREEK
RETENTION BASIN PROJECT

The purpose of the memorandum is to record my impressions of the East Fork Cave Creek watershed area during our 15 February 1991 reconnaissance and provide some follow-up estimates of sediment production.

From my observations of the watershed above the proposed retention basins there probably will be only a slight tendency to erode the top soil. In upland areas in the northern portion of the watershed, soil profiles are shallow with considerable outcrops of highly weathered plutonic rock. This rock provides the courser angular rock fragments seen on moderate slopes closer to the East Fork Cave Creek. Soil profiles are deeper on moderate slopes and are very deep close to the creek system. Soils appear to be colluvial in nature; i.e. formed in place, and do not appear to have been washed from upland areas to subsequently be deposited. There is very little evidence of significant erosion on moderate slopes where drainage pathways are poorly defined. Soils on moderate slopes appear to be resistent to extensive erosion and where drainage rills or other similar features are seen, medium to course sands are in the bottoms indicating the fines have been washed out. On the basis of what I saw I would conjecture that the bed load during intense rainfall events is very small and that the main form of sediment transport is suspended load.

To get some idea of what kind of soils we are dealing with, one sample was obtained on a moderate sloping area near the upland and northern portion of the watershed (about a mile from the East Fork Cave Creek watercourse). Analysis of the grain size of the material is attached. Almost thirty percent of the material is angular rock fragments greater than 2mm in diameter. Another thirty percent is in the medium sand range. There is very little fine sand and about twenty five percent of the sample consists of fines. Although we did not do a hydrometer test on the fines they appear to be predominantly in the silt range with very little clay. Although not tested, the soil appears

IRVINE\P1355016.MEM

to have a very low plasticity. The soil appears to be a SW soil based upon the unified soil classification system. The soil is strong and hard with a very low compressibility. Such a soil is not easily eroded by rain drop impact or flowing water particularly with moderate to low velocities which would be the case in much of the watershed.

To make some rough estimates of long term sediment transport, the Leopold, Wolman, and Miller book entitled Fluvial Processes in Geomorphology (Freeman Co., 1964) has some information that applies to the area. From Figure 3-11 in the book, average annual effective precipitation for the area may be approximately 1.2 inches. From Figure 3-7 and using the 1.2 inches of effective rainfall, the annual sediment yield is approximately 100 tons per square mile per year. This value is based upon desert shrub lands. Assuming the suspended load soils unit weight is 120 lb. per cubic foot, the average annual suspended load is 62 cubic yards per square mile per year. I am assuming that this load will be predominately suspended and would consist of silt range sizes. Depending on the retention time in the purposed retention basin a fraction of this suspended sediment may drop out. Using stokes equation for a particles fall velocity;

$$V = \frac{2\,a^2(W_s - W_f)}{9u}$$

(a = particle radius, u = absolute fluid velocity, W_s = soil unit weight, and W_f = fluid unit weight), the fall velocity, V, is estimated to be about 22 feet per hour for the largest particle at the top of the silt syes (diameter equal 0.075 mm). As you can see some of the largest particle in the silt size might tend to drop out in the retention basin, but most finer sizes would pass through the retention basin system.

In summary, it is my view the sedimentation is not a problem in the area of the proposed project.

GLG/paj

Attachment

IRVINE\P1355016.MEM

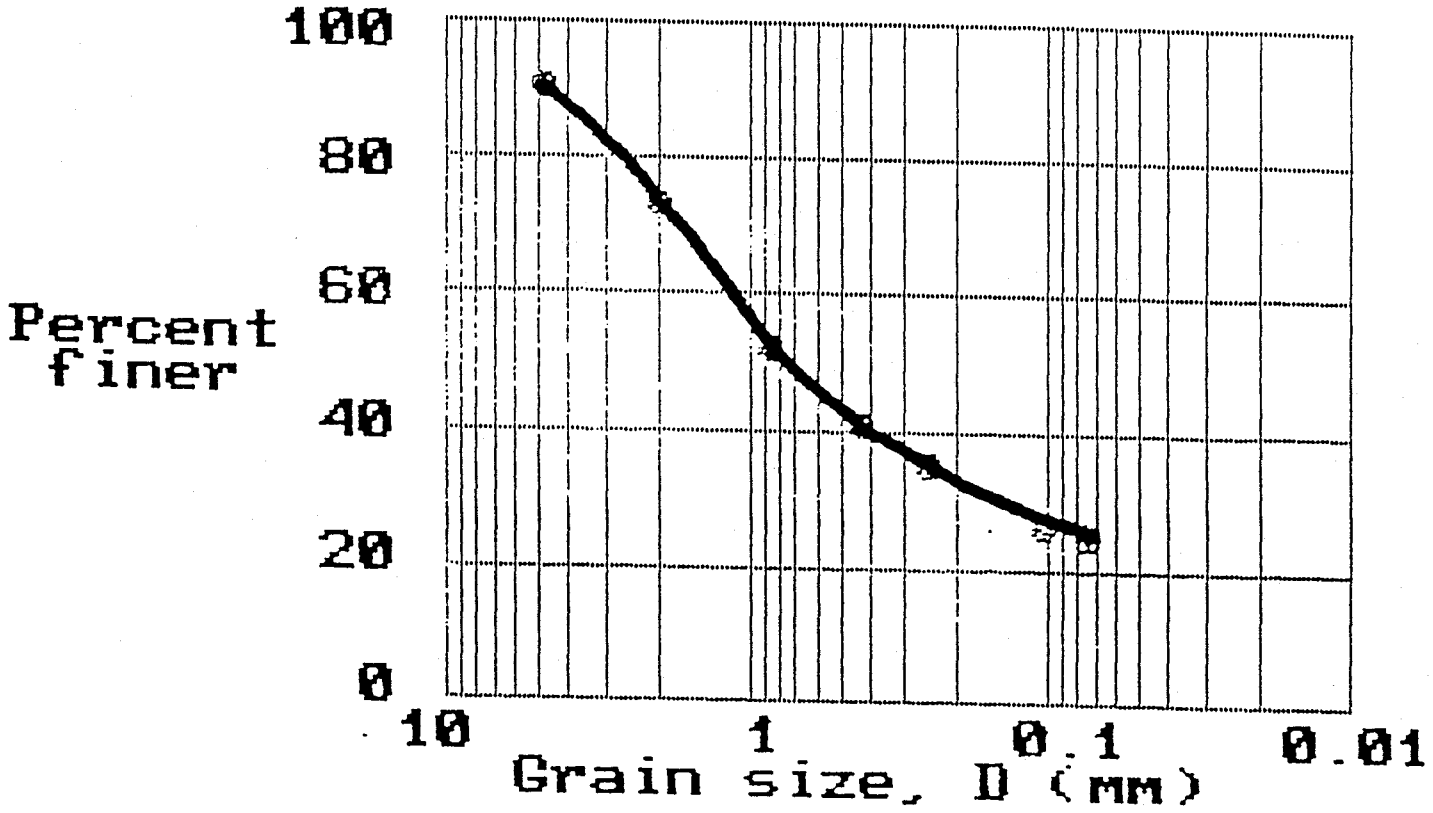
S I E V E - A N A L Y S I S

Description of soil:
Location:
Sample No.: 0 Weight of oven dry sample, W (g): 499.5

Sieve No.	Sieve opening (mm)	Weight retained on each sieve (g)	Percent of weight retained on each sieve	Cumulative percent retained	Percent finer
4	4.750	49.50	9.91	9.91	90.09
10	2.000	85.90	17.20	27.11	72.89
20	0.850	106.20	21.26	48.37	51.63
40	0.425	55.90	11.19	59.56	40.44
60	0.250	28.20	5.65	65.21	34.79
140	0.106	42.80	8.57	73.77	26.23
200	0.075	9.30	1.86	75.64	24.36
Pan	---	120.90			

Σ 498.70 = W1

Loss during sieve analysis=[(W-W1)/W]x100= 0.16% (OK if less than 2%)



Plot of percent finer vs. grain size

REINFORCED CONCRETE PIPE: Job No: P79-086-016
D-load Calculations Filename:C:\WORKSHT\RCPPIPE.WK1
Line: All 18" D(0.01")= 1423
From Sta: N/A To Sta: N/A

1. DETERMINATION OF EARTH LOAD:

A. Trench type installation:

Wd = 429 lbs/ft of trench.

Input Data:

Km' = 0.13
Ht. of fill, H = 1 ft.
fill unit wt, w = 120 lb/ft^3
Trench width, Bd = 3.7 ft. In Diam.= 18 in
Outside Diam, Dt = 25.75 in Wall Th.= 3.875 in

Calculated Data:

Cd = 0.2610 0

2. DETERMINATION OF LIVE LOAD:

A. No Pavement (Boussinesq Eqns.):

Wl = 2,774 lbs/ft of trench.

Calculated Data:

Avg Intens., Wl = 2357.31 psf
Wheel Loads, P = 16,000 lb
Impact Fact. If = 30%
Dist LL Area, ALL= 8.8 sf

Tot Live load, Wt= 13,051 lb Wt = 17,300 lb
La = 2.58 ft Lb = 3.42 ft
Sla =2.145833 ft Slb =2.145833 ft

Eff Supp len. Le = 5.4 ft Le = 6.2 ft

3. SELECTION OF BEDDING:

A. Trench Bedding:

Bedding Class C (A,B,C, or D)

REINFORCED CONCRETE PIPE: Job No: P79-086-016
D-load Calculations Filename:C:\WORKSHT\RCPPIPE.WK1
Line: All 18" D(0.01")= 1423
From Sta: N/A To Sta: N/A

---

4. DETERMINATION OF BEDDING FACTOR:

A. Trench Bedding:

Bfv = 1.50 (Trench variable bedding factor)

			See	Table	Page
Fixed bed fact.=	1.50				
Reinforced As =	0.00%	(Class A only)			
Trans. width, Bdt=	3.50	CPDM		18	112
Project. Ratio, P=	0.8	CPDM			
Settle Ratio, rsd=	0.5	CPDM		43	162
H/Bc =	0.47				
rsd*P =	0.4				
Bfe =	1.50			62	180

* CPDM=Concrete Pipe Design Manual

---

5. APPLICATION OF FACTOR OF SAFETY:

F.S. = 1.5

---

6. SELECTION OF PIPE STRENGTH:

D(0.01) = 1423
Dult = 2135 lb

Class IV (ASTM C 76 Strength Classification)

1429 lb/ft
2774
3203

$$B_{fv} = \left(D_{0.01} = \frac{3203}{1.5 \times 4.0} \times 1.5 \right.$$

```
REINFORCED CONCRETE PIPE:            Job No:  P79-086-016
D-load Calculations                  Filename:C:\WORKSHT\RCPPIPE.WK1
   Line:   "B"                       D(0.01")=    1231
From Sta:  25+00    To Sta:  38+20
1.  DETERMINATION OF EARTH LOAD:
=========================================================================
A.  Trench type installation:
-------------------------------------------------------------------------
     Wd =  13,237 lbs/ft of trench.
     ------------------------------
         Input Data:
                     Km' =    0.13
          Ht. of fill, H =      13 ft.
         fill unit wt, w =     120 lb/ft^3
         Trench width, Bd =     10 ft.      In Diam.=      72 in
         Outside Diam, Dt =     84 in       Wall Th.=       6 in
         Calculated Data:
                      Cd =  1.1031                 0

-------------------------------------------------------------------------
2.  DETERMINATION OF LIVE LOAD:
=========================================================================
A.  No Pavement (Boussinesq Eqns.):
-------------------------------------------------------------------------
     Wl =       0 lbs/ft of trench.
     ------------------------------
         Calculated Data:
          Avg Intens., Wl =   61.24 psf
           Wheel Loads, P =  48,000 lb
          Impact Fact. If =       0%
         Dist LL Area, ALL=   783.8 sf

         Tot Live load, Wt=  11,823 lb            Wt =  12,183 lb
                      La =   27.58 ft             Lb =   28.42 ft
                     Sla =      7 ft            Slb =      7 ft

         Eff Supp len. Le =    36.8 ft            Le =    37.6 ft

-------------------------------------------------------------------------
3.  SELECTION OF BEDDING:
=========================================================================
A.  Trench Bedding:
-------------------------------------------------------------------------
    Bedding Class C        (A,B,C, or D)
     ------------------------------
```

REINFORCED CONCRETE PIPE: Job No: P79-086-016
D-load Calculations Filename:C:\WORKSHT\RCPPIPE.WK1
Line: "B" D(0.01")= 1231
From Sta: 25+00 To Sta: 38+20

---

4. DETERMINATION OF BEDDING FACTOR:

A. Trench Bedding:

Bfv = 1.79 (Trench variable bedding factor)

Fixed bed fact.= 1.50
Reinforced As = 0.00% (Class A only)

			See Table	Page
Trans. width, Bdt=	11.83	CPDM	30	136
Project. Ratio, P=	0.8	CPDM		
Settle Ratio, rsd=	0.5	CPDM	43	162
H/Bc =	1.86			
rsd*P =	0.4			
Bfe =	2.06		62	180

* CPDM=Concrete Pipe Design Manual

5. APPLICATION OF FACTOR OF SAFETY:

F.S. = 1.5

6. SELECTION OF PIPE STRENGTH:

D(0.01) = 1231
Dult = 1846

Class III (ASTM C 76 Strength Classification)

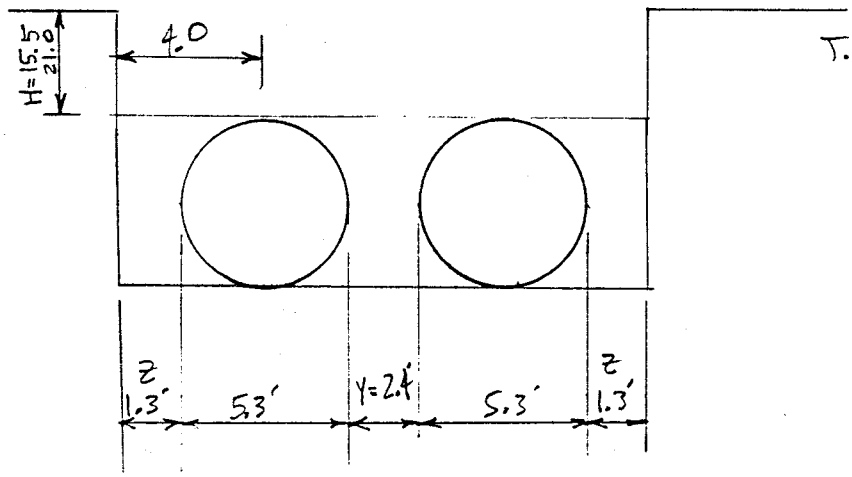
```
REINFORCED CONCRETE PIPE:            Job No:  P79-086-016
D-load Calculations                  Filename:C:\WORKSHT\RCPPIPE.WK1
   Line:   "F"                       D(0.01")=    1069
From Sta:  10+00    To Sta:  10+85
1.  DETERMINATION OF EARTH LOAD:
=========================================================================
A.  Trench type installation:
-------------------------------------------------------------------------
     Wd =   7,585 lbs/ft of trench.
     -------------------------------
         Input Data:
                     Km' =    0.13
          Ht. of fill, H =      11 ft.
         fill unit wt, w =     120 lb/ft^3
         Trench width, Bd =      7 ft.      In Diam.=      48 in
         Outside Diam, Dt =   56.25 in      Wall Th.=   4.125 in
         Calculated Data:
                      Cd =  1.2900                 0

-------------------------------------------------------------------------
2.  DETERMINATION OF LIVE LOAD:
=========================================================================
A.  No Pavement (Boussinesq Eqns.):
-------------------------------------------------------------------------
     Wl =       0 lbs/ft of trench.
     -------------------------------
         Calculated Data:
          Avg Intens., Wl =   79.99 psf
           Wheel Loads, P =  48,000 lb
          Impact Fact. If =       0%
         Dist LL Area, ALL=   600.1 sf

         Tot Live load, Wt=   9,029 lb            Wt =   9,344 lb
                      La =   24.08 ft             Lb =   24.92 ft
                     Sla =  4.6875 ft            Slb =  4.6875 ft

         Eff Supp len. Le =    30.2 ft            Le =    31.1 ft

-------------------------------------------------------------------------
3.  SELECTION OF BEDDING:
=========================================================================
A.  Trench Bedding:
-------------------------------------------------------------------------
    Bedding Class C        (A,B,C, or D)
     -------------------------------
```

```
REINFORCED CONCRETE PIPE:            Job No:  P79-086-016
D-load Calculations                  Filename:C:\WORKSHT\RCPPIPE.WK1
   Line:   "F"                       D(0.01")=    1069
From Sta:  10+00    To Sta:  10+85
-----------------------------------------------------------------------
4.  DETERMINATION OF BEDDING FACTOR:
=======================================================================
A.  Trench Bedding:
-----------------------------------------------------------------------
    Bfv =    1.77   (Trench variable bedding factor)
     -------------------------------
          Fixed bed fact.=    1.50
           Reinforced As =    0.00% (Class A only)
                                         See   Table    Page
                                             ------------------
        Trans. width, Bdt=    8.33 CPDM          26      128
        Project. Ratio, P=     0.8 CPDM
        Settle Ratio, rsd=     0.5 CPDM          43      162
                   H/Bc =    2.35
                 rsd*P =     0.4
                   Bfe =    2.05                 62      180
                                       * CPDM=Concrete Pipe Design Manual
-----------------------------------------------------------------------
5.  APPLICATION OF FACTOR OF SAFETY:
=======================================================================
            F.S. =     1.5


-----------------------------------------------------------------------
6.  SELECTION OF PIPE STRENGTH:
=======================================================================
         D(0.01) =    1069
            Dult =    1604
     -------------------------------
            Class III      (ASTM C 76 Strength Classification)
```

```
REINFORCED CONCRETE PIPE:            Job No:  P79-086-016
D-load Calculations                  Filename:C:\WORKSHT\RCPPIPE.WK1
   Line:   "L"                       D(0.01")=     690
From Sta:  10+00    To Sta:  13+34
1.  DETERMINATION OF EARTH LOAD:
=========================================================================
A.  Trench type installation:
-------------------------------------------------------------------------
     Wd =   3,122 lbs/ft of trench.
     ------------------------------
         Input Data:
                     Km' =    0.13
           Ht. of fill, H =       4 ft.
          fill unit wt, w =     120 lb/ft^3
         Trench width, Bd =       7 ft.      In Diam.=      48 in
         Outside Diam, Dt =   56.25 in       Wall Th.=   4.125 in
         Calculated Data:
                      Cd =  0.5310                 0

-------------------------------------------------------------------------
2.  DETERMINATION OF LIVE LOAD:
=========================================================================
A.  No Pavement (Boussinesq Eqns.):
-------------------------------------------------------------------------
     Wl =   1,018 lbs/ft of trench.
     ------------------------------
         Calculated Data:
          Avg Intens., Wl =  322.56 psf
           Wheel Loads, P =  32,000 lb
          Impact Fact. If =       0%
         Dist LL Area, ALL=    99.2 sf

         Tot Live load, Wt=  11,839 lb            Wt =  19,157 lb
                      La =    7.83 ft             Lb =   12.67 ft
                     Sla =  4.6875 ft            Slb =  4.6875 ft

         Eff Supp len. Le =    14.0 ft            Le =    18.8 ft

-------------------------------------------------------------------------
3.  SELECTION OF BEDDING:
=========================================================================
A.  Trench Bedding:
-------------------------------------------------------------------------
    Bedding Class C        (A,B,C, or D)
     ------------------------------
```

```
REINFORCED CONCRETE PIPE:            Job No:  P79-086-016
D-load Calculations                  Filename:C:\WORKSHT\RCPPIPE.WK1
   Line:   "L"                       D(0.01")=     690
From Sta:  10+00    To Sta:  13+34
-----------------------------------------------------------------------
4.  DETERMINATION OF BEDDING FACTOR:
=======================================================================
A.  Trench Bedding:
-----------------------------------------------------------------------
    Bfv =    1.50   (Trench variable bedding factor)
     -------------------------------
          Fixed bed fact.=    1.50
           Reinforced As =    0.00% (Class A only)
                                          See   Table    Page
                                          ------------------
        Trans. width, Bdt=    6.50 CPDM          26      128
        Project. Ratio, P=     0.8 CPDM
        Settle Ratio, rsd=     0.5 CPDM          43      162
                    H/Bc =    0.85
                  rsd*P =     0.4
                     Bfe =    1.50               62      180
                                       * CPDM=Concrete Pipe Design Manual
-----------------------------------------------------------------------
5.  APPLICATION OF FACTOR OF SAFETY:
=======================================================================
            F.S. =     1.5


-----------------------------------------------------------------------
6.  SELECTION OF PIPE STRENGTH:
=======================================================================
        D(0.01) =     690
           Dult =    1035
     -------------------------------
            Class I        (ASTM C 76 Strength Classification)
```

2-54"



T.W. = 9'-8"
10'-4"

$W_c = w B_c^2 C_c$ 19,800 lb/ft

$120 (5.3)^2$ $C_c = 5.87$

$\frac{H}{B_c}$	B_f
.19	2.55
3.96	2.01
2.92	2.02

(H=21') pos. proj. cond. D = 2200

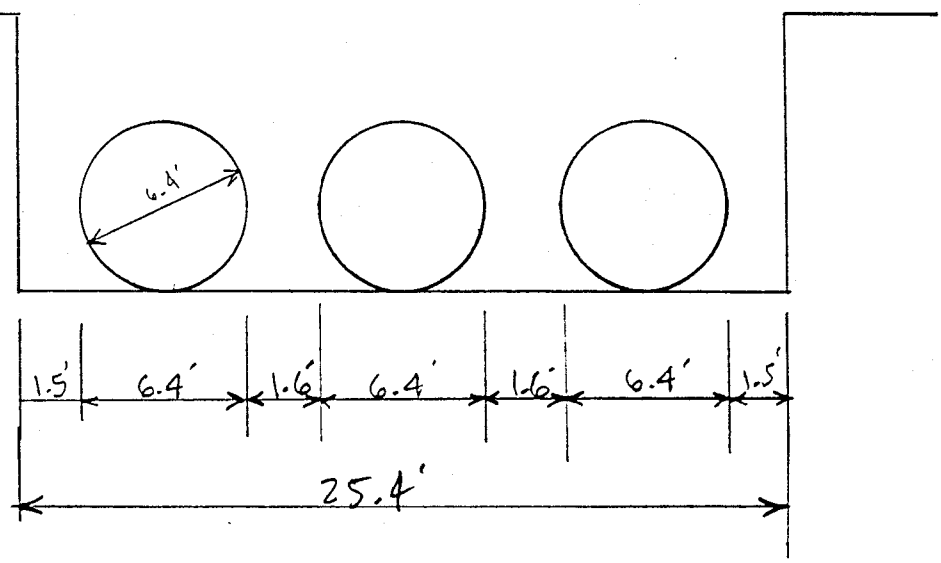
Trench Cond D = 2150

(H=15.5') 14700 lb/ft D = 1650

NBS LOWRY ENGINEERS & PLANNERS

DATE
JOB NO
BY CHK'D
SHEET OF
R.C.C.

3 - 66"



14.5' W_c = 16,200

H/B_c = 2.6 B_f = 2.03

D-load = 1450

3 - 48"

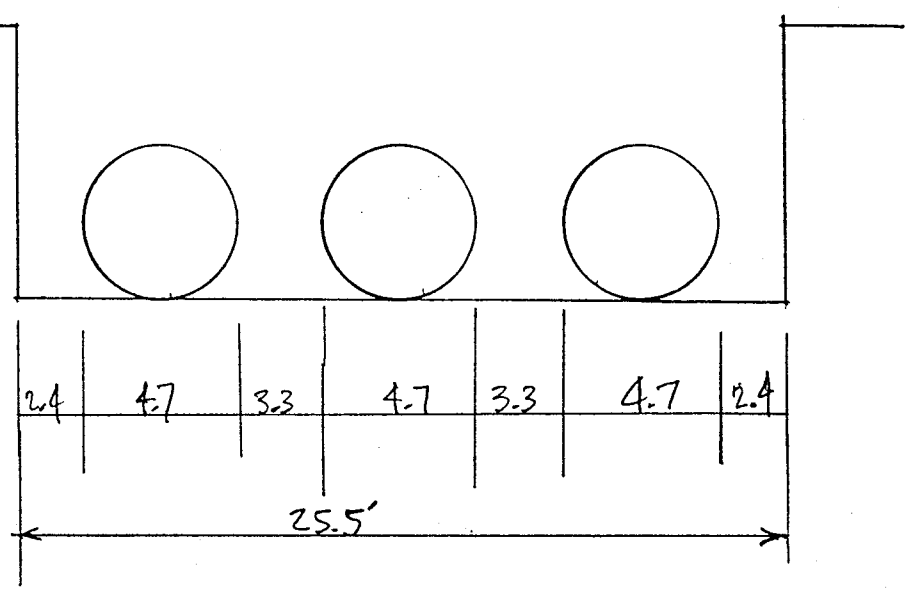
W_c = 12,600

15'

H/B_c = 3.2

B_f = 2.02

D-load = 1600



NBS LOWRY ENGINEERS & PLANNERS

DATE

JOB NO

D-Load for 72" x 113" Ellipt. Pipe

BY BJF CHK'D

SHEET OF

R.C.C.

$K\mu' = 0.13$
$H = 19'$
$w = 120\ lb/ft^3$

72" x 113" Horizontal Elliptical Pipe

1. $W_d = 27{,}500\ lb/ft$ (Fig 157.2, pg 365)

2. $W_\ell = 0$

3. Bedding class "C"

4. Bedding Factor = 1.5

5. D(0.01")

$$= \frac{27{,}500 + 0}{1.5 \times \left(\frac{113''}{12}\right)} = 1947$$

CORRUGATED METAL PIPE:
Min. Gage Calculations
Line: All 18"
From Sta: N/A To Sta: N/A

Job No: P79-086-016
Filename:C:\WORKSHT\CMPPIPE.WK1

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction (value for routine use)

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,920 psf $P_V = DL + LL$ / or $P_v = DL + 0.75LL$

$= 1800 + 120$

$= 1920$ psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	1	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	18	in
Load Factor =	1	
LL =	1800	psf
DL =	120	psf

---

3. RING COMPRESSION:

C = 1,440 lbs/ft of trench.

C = Ring Compression
P_v = Design Pressure
S = Span, in ft.

$$C = P_v \times \frac{S}{2}$$

$$= 1920 \times \frac{1.5}{2}$$

$$= 1440 \text{ lbs/ft of trench}$$

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: All 18"
From Sta: N/A To Sta: N/A

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

For $\frac{D}{r} < 294$, $f_a = \frac{33,000}{2K}$

$D/r < 294 \Rightarrow f_a = \frac{33,000}{2*(.86)} = 19,186$ psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 106

0

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.075 in^2/ft required

$A = \frac{C}{f_c} = \frac{1440 \text{ lbs/ft}}{19,186 \text{ lbs/in}^2} = .075 \frac{in^2}{ft}$ req'd.

Required thick. = 0.052 in
= 18 ga

A =(0.619)

---

6. HANDLING STIFFNESS:

FF = 0.0072 OK

$FF = \frac{D^2}{EI} = \frac{18^2}{30\times10^6 * .018} = \frac{D}{r_m} = 166$

Max FF = 0.043

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation) ? How

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

$Y = 2.94 R^{0.41}$
$= 2.94(1000)^{0.41}$
$= 49.9$ yrs. $* 3.4 = 170$ yrs.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 25+00 To Sta: 38+20

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction (Value for routine use)

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,560 psf

$DL = 120 \frac{lb}{ft^3} * (13\ ft) = 1560\ lb/ft^2$

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	13	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	72	in
Load Factor =	1	
LL =	0	psf
DL =	1560	psf

---

3. RING COMPRESSION:

C = 4,680 lbs/ft of trench.

$C = P_v * \frac{S}{2} = 1560 * \frac{6}{2} = 4{,}680\ lbs/ft$

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 25+00 To Sta: 38+20

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. = 3 x 1"
D/rmin = 211

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

$f_b = \frac{33,000}{2K} = \frac{33,000}{2(0.86)} = 19,186$ psi

$f_c = 19,186$ psi

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.244 in^2/ft required

$A = \frac{C}{f_c} = \frac{4,680 \text{ lb/ft}}{19,186 \text{ lb/in}^2} = 0.244 \text{ in}^2/\text{ft}$

Required thick. = 0.052 in
= 18 ga

A =(0.711)

---

6. HANDLING STIFFNESS:

FF = 0.0251 OK

Max FF = 0.06

$FF = \frac{D^2}{EI} = \frac{(72)^2}{30\times10^6 \times .0827} = .0021$

Flexibility Factor

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

$Y = 2.94R^{0.41}$
$= 2.94(1000)^{0.41}$
$= 49.9$ years.

using multiple factor $Y = 3.4 \times 49.9$
$= 170$ years.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 25+00 To Sta: 38+20

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,560 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	13	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	72	in
Load Factor =	1	
LL =	0	psf
DL =	1560	psf

---

3. RING COMPRESSION:

C = 4,680 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 25+00 To Sta: 38+20

---

4. ALLOWABLE WALL STRESS:

fc = 14,860 psi

fb = 25,560 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 422

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

$\frac{D}{r} = \frac{72}{.1707} = 421 > 294$ (500)

$f_a = \left[\frac{40,000 - .091(422)^2}{2(.86)}\right] = 14,860$ psi

5. WALL CROSS-SECTIONAL AREA:

A = 0.315 in^2/ft required

$A = C/f_c = \frac{4680}{14,860} = 0.315 \text{ in}^2/\text{ft}$

Required thick. = 0.052 in
= 18 ga

A =(0.619) Use 18 ga. thickness = .052 in

6. HANDLING STIFFNESS:

FF = ~~0.1152~~ .0096 TOO HIGH

Max FF = 0.06

New Th = 12 ga

New Th = 0.109 in
New FF = 0.0505

$FF = \frac{D^2}{EI} = \frac{(72)^2}{30 \times 10^6 \times .0180} = .0096$

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

$.0505 = \frac{D^2}{30 \times 10^6 \times .0401}$

$D^2 = \frac{(72)^2}{30 \times 10^6 \times .0505}$

Year $= 2.94 R^{0.41} = 2.94 \times (1000)^{0.41}$
$= 49.93$ yrs

w/ SF = 3.4 years = $3.4 \times 49.93 = 170$ yrs.

CORRUGATED METAL PIPE:
Min. Gage Calculations
Line: "F"
From Sta: 10+00 To Sta: 10+85

Job No: P79-086-016
Filename:C:\WORKSHT\CMPPIPE.WK1

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction (used very frequently)

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,320 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	11	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	48	in
Load Factor =	1	
LL =	0	psf
DL =	1320	psf

$DL = 120 \frac{lb}{ft^3} \times (11\ ft) = 1320\ psf$

$LL = 0$

$P_v = 1320 + 0 = 1320$

---

3. RING COMPRESSION:

C = 2,640 lbs/ft of trench.

$C = P_v \times \frac{S}{2} = 1320 \times \frac{4'}{2} = 2640\ lbs/ft$

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "F"
From Sta: 10+00 To Sta: 10+85

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

$D/r = 48/.1707 = 281$

Corrugation Profiles

$f_b = \frac{33,000}{2K} = \frac{33,000}{2 \times .86}$

$= 19,186$ psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 281

1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.138 in^2/ft required

$A = C/f_c = \frac{2640}{19,186} = 0.138 in^2/ft$

Required thick. = 0.052 in ✓
= 18 ga ✓

A =(0.619)

---

6. HANDLING STIFFNESS:

FF = 0.0512 OK

$FF = \frac{D^2}{EI} = \frac{(48)^2}{30 \times 10^6 \times .0150} = .0512$

Max FF = 0.06

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

$Life = 2.94 * R^{.41}$

$= 2.94 * (1000)^{.41}$

$= 49.9$

W/ USING ga factor = 3.4 Life = 3.4 * 49.9 = 170 years

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "L"
From Sta: 10+00 To Sta: 13+34

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 880 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	4	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	48	in
Load Factor =	1	
LL =	400	psf
DL =	480	psf

---

3. RING COMPRESSION:

C = 1,760 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "L"
From Sta: 10+00 To Sta: 13+34

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 281

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.092 in^2/ft required

Required thick. = 0.052 in A =(0.619)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0512 OK

Max FF = 0.06

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 10+00 To Sta: 22+00

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,860 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	15.5	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	54	in
Load Factor =	1	
LL =	0	psf
DL =	1860	psf

---

3. RING COMPRESSION:

C = 4,185 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 10+00 To Sta: 22+00

---

4. ALLOWABLE WALL STRESS:

fc = 18,533 psi

fb = 31,877 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 317

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.226 in^2/ft required

Required thick. = 0.052 in
= 18 ga

A =(0.619)

---

6. HANDLING STIFFNESS:

FF = 0.0648 TOO HIGH

Max FF = 0.06

New Th = 16 ga

New Th = 0.064 in
New FF = 0.0514

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 10+00 To Sta: 22+00

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,860 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	15.5	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	54	in
Load Factor =	1	
LL =	0	psf
DL =	1860	psf

---

3. RING COMPRESSION:

C = 4,185 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 10+00 To Sta: 22+00

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. = 3 x 1"
D/rmin = 158

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.218 in^2/ft required

Required thick. = 0.052 in A =(0.711)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0141 OK

Max FF = 0.06

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 22+00 To Sta: 25+00

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 2,520 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	21	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	54	in
Load Factor =	1	
LL =	0	psf
DL =	2520	psf

---

3. RING COMPRESSION:

C = 5,670 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 22+00 To Sta: 25+00

---

4. ALLOWABLE WALL STRESS:

fc = 18,533 psi

fb = 31,877 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 317

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.306 in^2/ft required

Required thick. = 0.052 in A =(0.619)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0648 TOO HIGH New Th = 16 ga

Max FF = 0.06 New Th = 0.064 in
New FF = 0.0514

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 22+00 To Sta: 25+00

REFERENCES:

==========

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

==========

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

==========

Pv = 2,520 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	21	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	54	in
Load Factor =	1	
LL =	0	psf
DL =	2520	psf

---

3. RING COMPRESSION:

==========

C = 5,670 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "B"
From Sta: 22+00 To Sta: 25+00

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. = 3 x 1"
D/rmin = 158

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.296 in^2/ft required

Required thick. = 0.052 in A =(0.711)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0141 OK

Max FF = 0.06

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "H"
From Sta: 22+38 To Sta: 23+08

REFERENCES:

==========

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

==========

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

==========

Pv = 1,740 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	14.5	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	66	in
Load Factor =	1	
LL =	0	psf
DL =	1740	psf

---

3. RING COMPRESSION:

==========

C = 4,785 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "H"
From Sta: 22+38 To Sta: 23+08

---

4. ALLOWABLE WALL STRESS:

fc = 16,201 psi

fb = 27,866 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 387

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.295 in^2/ft required

Required thick. = 0.052 in A =(0.619)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0968 TOO HIGH New Th = 12 ga

Max FF = 0.06 New Th = 0.109 in
New FF = 0.0424

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "H"
From Sta: 22+38 To Sta: 23+08

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,740 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	14.5	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	66	in
Load Factor =	1	
LL =	0	psf
DL =	1740	psf

---

3. RING COMPRESSION:

C = 4,785 lbs/ft of trench.

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "H"
From Sta: 22+38 To Sta: 23+08

---

4. ALLOWABLE WALL STRESS:

fc = 19,186 psi

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. = 3 x 1"
D/rmin = 194

Corrugation Profiles
1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

A = 0.249 in^2/ft required

Required thick. = 0.052 in A =(0.711)
= 18 ga

---

6. HANDLING STIFFNESS:

FF = 0.0211 OK

Max FF = 0.06

---

7. SERVICE LIFE:

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

CORRUGATED METAL PIPE: Job No: P79-086-016
Min. Gage Calculations Filename:C:\WORKSHT\CMPPIPE.WK1
Line: "H"
From Sta: 23+13 To Sta:24+85.34

REFERENCES:

1. "Handbook of Steel Drainage & Highway Construction Products," American Iron & Steel Institute, Third Edition, 1983.

2. "Modern Sewer Design," America Iron & Steel Institute, 1990.

---

1. BACKFILL DENSITY:

Dens = 85% Compaction

---

2. DESIGN PRESSURE (DEAD & LIVE LOAD):

Pv = 1,800 psf

Live Loading =	H-20	(H-20 or H-25)
Ht. of fill, H =	15	ft.
fill unit wt, w =	120	lb/ft^3
Diameter =	48	in
Load Factor =	1	
LL =	0	psf
DL =	1800	psf

---

3. RING COMPRESSION:

C = 3,600 lbs/ft of trench.

CORRUGATED METAL PIPE:
Min. Gage Calculations
Line: "H"
From Sta: 23+13 To Sta:24+85.34

Job No: P79-086-016
Filename:C:\WORKSHT\CMPPIPE.WK1

---

4. ALLOWABLE WALL STRESS:

==========

fc = 19,186 psi

---

fb = 33,000 psi
Load Fact, K = 0.86
Corrug. Prof. =2-2/3 x 1/2"
D/rmin = 281

Corrugation Profiles

1-1/2 x 1/4"
2 x 1/2"
2-2/3 x 1/2"
3 x 1"
5 x 1"
6 x 2"

---

5. WALL CROSS-SECTIONAL AREA:

==========

A = 0.188 in^2/ft required

Required thick. = 0.052 in A =(0.619)
= 18 ga

---

6. HANDLING STIFFNESS:

==========

FF = 0.0512 OK

Max FF = 0.06

---

7. SERVICE LIFE:

==========

Life = 85 yrs (to first perforation)

Soil pH = 7.9
Min Res, R = 1000 ohm cm
Selected th. = 8 gage
ga Fact = 3.4
Avg life= 170 yrs (25% avg thickness reduction)

Item No.	Item Description	Unit	Qty.	Unit Price	Amount
CIVIL					
	Mobilization & Demobilization	L.S.	1	$100,000	$100,000.00
201	Clearing & Grubbing	L.S.	1	$30,000.00	$30,000.00
350	Removal of Exist. Improvements	L.S.	1	$3,000.00	$3,000.00
618	18" Storm Drain Pipe	L.F.	1978	$22.00	$43,516.00
618	48" Storm Drain Pipe	L.F.	923	$48.00	$44,304.00
618	54" Storm Drain Pipe	L.F.	3000	$55.00	$165,000.00
618	66" Storm Drain Pipe	L.F.	210	$70.00	$14,700.00
618	72" Storm Drain Pipe	L.F.	1320	$80.00	$105,600.00
618	72" x 113" Storm Drain Pipe	L.F.	237	$175.00	$41,475.00
618	66" x 48" Storm Drain Reducer	L.F.	15	$170.00	$2,550.00
505	Catch Basin (Type N)	Ea.	8	$1,400.00	$11,200.00
625	48" M.H. (MAG 520 & 522)	Ea.	1	$2,200.00	$2,200.00
625	Manhole	Ea.	2	$1,900.00	$3,800.00
505	Concrete (Class A)	C.Y.	492	$200.00	$98,400.00
505	Reinforcing Steel	Lbs.	24648	$0.50	$12,324.00
340	Sidewalk	S.F.	20112	$1.25	$25,140.00
310	Untreated Aggregate Base	C.Y.	48	$35.00	$1,680.00
206	Excavation	C.Y.	666021	$1.20	$799,225.20
206	Surplus Material	C.Y.	665788	$0.75	$499,341.00
206	Backfill	C.Y.	233	$4.00	$932.00
	Grasscrete	S.Y.	210	$45.00	$9,450.00
618	Trash Rack	Ea.	1	$4,000.00	$4,000.00
	Pipe Support	Ea.	3	$100.00	$300.00
630	6" Tapping Sleeve & Valve	Ea.	1	$1,300.00	$1,300.00
610	6" ACP & Fittings	L.F.	691	$18.00	$12,438.00
610	6" DIP & Fittings	L.F.	84	$22.00	$1,848.00
610	12" DIP & Fittings	L.F.	104	$42.00	$4,368.00
630	6" Valve, Box & Cover	Ea.	2	$1,100.00	$2,200.00
630	12" Valve, Box & Cover	Ea.	2	$2,300.00	$4,600.00
	Pavement Replacement	S.F.	75	$2.50	$187.50
625	48" Manhole (P-1430)	Ea.	3	$2,200.00	$6,600.00
615	8" Sewer Pipe	L.F.	56.5	$43.00	$2,429.50
615	Drop Sewer Connection	Ea.	1	$510.00	$510.00
615	Manhole Plug	Ea.	1	$100.00	$100.00
401	Detours & Traffic Control	L.S. Lump Sum	1	$10,000.00	$10,000.00
401	Traff. Cont. - Unif. Officer	Hrs	120	$40.00	$4,800.00
	Rip-Rap	C.Y.	15	$45.00	$675.00
	Pre-fab Grating	S.F.	1702	$5.50	$9,361.00
	Structural Steel	Lbs	5798	$1.00	$5,798.00
				Sub-total	$2,085,352
LANDSCAPE					
	Salvaged Tree - 36" Box	Ea.	38	$335.00	$12,730.00
	Salvaged Tree - 48" Box	Ea.	30	$669.00	$20,070.00

Need paving plans.

Item No.	Item Description	Unit	Qty.	Unit Price	Amount
	Salvaged Tree - 54" Box	Ea.	25	$950.00	$23,750.00
	Salvaged Tree - 60" Box	Ea.	9	$1,140.00	$10,260.00
	Salvaged Tree - 66" Box	Ea.	2	$1,330.00	$2,660.00
	Salvaged Tree - 72" Box	Ea.	2	$1,520.00	$3,040.00
	Salvaged Tree - 96" Box	Ea.	2	$2,229.00	$4,458.00
	24" Box Trees (New)	Ea.	301	$225.00	$67,725.00
	5 Gal. Shrubs	Ea.	1260	$17.50	$22,050.00
	1 Gal. Shrubs	Ea.	848	$7.50	$6,360.00
	Turf	S.F.	881571	$0.10	$88,157.10
	Concrete Header (6" x 8")	L.F.	6108	$5.50	$33,594.00
	Concrete Curb (8" x 24")	L.F.	468	$9.00	$4,212.00
	Decomposed Granite Type "A"	S.F.	486283	$0.22	$106,982.26
	Decomposed Granite Type "B"	S.F.	58208	$0.22	$12,805.76
	Stabilized Granite	S.F.	36189	$0.70	$25,332.30
	Sand (12" depth)	Ton	443	$10.00	$4,430.00
	River Rock	S.F.	2775	$3.00	$8,325.00
	Concrete Tire Stops	Ea.	98	$20.00	$1,960.00
				Sub-total	$458,901
IRRIGATION					
	1401 Bubbler (0.25 gpm)	Ea.	2094	$12.00	$25,128.00
	1404 Bubbler (1.00 gpm)	Ea.	499	$15.00	$7,485.00
	1408 Bubbler (2.00 gpm)	Ea.	116	$15.00	$1,740.00
	640 Series Rotary Sprinkler	Ea.	429	$80.00	$34,320.00
	S700C Series Rotary Sprinkler	Ea.	36	$70.00	$2,520.00
	570C Series Pop-up Sprinkler	Ea.	25	$18.00	$450.00
	EFB-CP-1" (Turf) Valve	Ea.	1	$160.00	$160.00
	EFB-CP-1-1/2" (Turf) Valve	Ea.	19	$180.00	$3,420.00
	EFB-CP-2" (Turf) Valve	Ea.	86	$200.00	$17,200.00
	EFB-CP-PRS 1" Valve Bubbler	Ea.	44	$170.00	$7,480.00
	EFB-CP-PRS 1-1/2" Valve Bubble	Ea.	16	$190.00	$3,040.00
	EFB-CP-PRS 2" Valve Bubbler	Ea.	2	$210.00	$420.00
	MIR-5000F Controller	Ea.	4	$5,000.00	$20,000.00
	3/4" Quick Coupler Valve	Ea.	33	$70.00	$2,310.00
	3" Cast Iron Gate Valve	Ea.	22	$300.00	$6,600.00
	825Y-Series 2" Backflow Preven	Ea.	6	$700.00	$4,200.00
	HYDROVISOR Moisture Sensor	Ea.	4	$200.00	$800.00
	3" Class 200 PVC Pipe	L.F.	8660	$3.00	$25,980.00
	2" Class 200 PVC Pipe	L.F.	4600	$1.75	$8,050.00
	1-1/2" Class 200 PVC Pipe	L.F.	4080	$1.50	$6,120.00
	1-1/4" Class 200 PVC Pipe	L.F.	9400	$1.30	$12,220.00
	1" Class 200 PVC Pipe	L.F.	15640	$1.10	$17,204.00
	3/4" Class 200 PVC Pipe	L.F.	8620	$1.00	$8,620.00
	1/2" Class 315 PVC Pipe	L.F.	21060	$0.90	$18,954.00
	8" Sch 40 PVC Pipe - Sleeve	L.F.	80	$10.00	$800.00
	6" Sch 40 PVC Pipe - Sleeve	L.F.	100	$8.00	$800.00

Item No.	Item Description	Unit	Qty.	Unit Price	Amount
	#14 Common Wire (White)	L.F.	13400	$0.18	$2,412.00
	#14 Common Wire (Red)	L.F.	166100	$0.18	$29,898.00
	2" Turbine Water Meter	Ea.	6	$2,000.00	$12,000.00
				Sub-total	$280,331
ELECTRICAL/LIGHTING					
	20' Cemtec Pole	Ea.	35	$600.00	$21,000.00
	100 Watt Sodium Fixture	Ea.	35	$200.00	$7,000.00
	225A - 120/240V Control Cabine	Ea.	1	$2,750.00	$2,750.00
	400A - 277/480V Control Cabine	Ea.	1	$3,500.00	$3,500.00
	#2 Type XHHW Wire	L.F.	5520	$1.13	$6,237.60
	#4 Type XHHW Wire	L.F.	1440	$0.85	$1,224.00
	#6 Type XHHW Wire	L.F.	4060	$0.64	$2,598.40
	#8 Type XHHW Wire	L.F.	4260	$0.56	$2,385.60
	#12 Type XHHW Wire	L.F.	940	$0.37	$347.80
	#2 Bare Copper Bond	L.F.	2760	$0.66	$1,821.60
	#4 Bare Copper Bond	L.F.	720	0.47	$338.40
	#6 Bare Copper Bond	L.F.	2030	0.32	$649.60
	#8 Bare Copper Bond	L.F.	2130	0.26	$553.80
	#12 Bare Copper Bond	L.F.	470	0.19	$89.30
	1-1/4" PVC Conduit	L.F.	3480	2.73	$9,500.40
	1" PVC Conduit	L.F.	4160	2.62	$10,899.20
	3/4" PVC Conduit	L.F.	470	2.47	$1,160.90
				Sub-total	$72,057
PAVING					
	Catch Basin - Type M	Ea.	3	$2,000.00	$6,000.00
	Catch Basin - Type M-1	Ea.	2	$2,500.00	$5,000.00
	Catch Basin - Type N	Ea.	1	$2,000.00	$2,000.00
	Drainage Scupper	Ea.	1	$400.00	$400.00
	18" RCP Connector Pipe	L.F.	396	$25.00	$9,900.00
	Remove & Salvage Exist. Trees	Ea.	18	$500.00	$9,000.00
	Construct Drainage Swale	L.F.	255	$5.00	$1,275.00
	2" water service	Ea.	5	$750.00	$3,750.00
	Asphalt Conc. Pvmt D 1/2 Mix	Tons	1340	$25.00	$33,500.00
	Asphalt Conc. Pvmt A 1-1/2 Mix	Tons	2919	$25.00	$72,975.00
	Concrete Sidewalk	S.F.	41913	$1.25	$52,391.25
	Relocate Exist. Block Wall	L.S.	1	$3,000.00	$3,000.00
	Remove Exist. Signs	Ea.	6	$15.00	$90.00
	Remove Exist. Wire Fence	L.F.	1314	$2.00	$2,628.00
	Remove Exist. Curb & Gutter	L.F.	904	$1.00	$904.00
	Remove Exist. Pavement	S.Y.	165	$1.00	$165.00
	Remove Exist. Sidewalk	S.Y.	411	$1.00	$411.00
	Roll Curb, MAG 220 Type "C"	L.F.	49	$8.00	$392.00
	Survey Monument	Ea.	15	$250.00	$3,750.00
	Relocate Traffic Sign	Ea.	3	$75.00	$225.00

Item No.	Item Description	Unit	Qty.	Unit Price	Amount
	Vert. Curb & Gutter	L.F.	6173	$8.00	$49,384.00
	Adjust M.H. Frame & Cover	Ea.	4	$500.00	$2,000.00
	Adjust Water Valve Box	Ea.	4	$75.00	$300.00
	Driveway Entrance	S.F.	1842	$2.60	$4,789.20
				Sub-total	$264,229
				Total	$3,160,871
			15%	Contingency	$474,131
				GRAND TOTAL	$3,635,001

Registered Professional Engineer (Civil)
CERTIFICATE NO. 21033
KENNETH D. WALSH
Date Signed 4-11-91
ARIZONA, U.S.A.

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

REPORT FOR
GEOTECHNICAL ENGINEERING SERVICES
DETENTION BASINS 3A & 3B
CITY OF PHOENIX NO. ST-896829
24TH STREET AND GROVERS AVENUE
PHOENIX, ARIZONA

Submitted To:

NBS/Lowry Engineers & Planners
2600 North 44th Street
Phoenix, Arizona

Project No. 91-0524

11 April 1991

THOMAS-HARTIG & ASSOCIATES, INC.

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

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

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

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

NBS/Lowry Engineers & Planners
2600 North 44th Street
Phoenix, Arizona 85008-1599

11 April 1991

Attention: Brian J. Fry

Project: Detention Basins 3A & 3B
City of Phoenix No. ST-896829
24th Street and Grovers Avenue
Phoenix, Arizona

Project No. 91-0524

In accordance with your authorization, geotechnical engineering services were performed for the proposed storm water detention basin facilities. The detention basins will include approximately 37 acres divided into two areas and will range from about 15 to 20 feet below existing grade with a low point at Elevation 1417 feet. Inlet and outlet structures, a low flow outlet, and a storm drain will be included. The basin will be landscaped for recreational use when empty, and paved parking lots may be added in the future.

low flow
Ele - 1417'

Site Description: The site is vacant, undeveloped desert terrain divided into two interconnected areas, designed to allow stormwater flow from northeast to southwest. The upper or northeastern basin is bounded by Grovers Avenue on the south, Cave Creek Road on the east, and by an irregular northern boundary of John Cabot, 22nd Place, and Libby Street. A small panhandle extends along the Libby Street cul-de-sac toward 21st Street. The lower or southwestern basin is a roughly rectangular area bounded on the north by Grovers Avenue, on the east by 22nd Street, on the west by 20th Street, and by a line approximately 600 feet south of Grovers Avenue on the south.

The surface topography is slightly irregular and slopes down to the southeast with an overall elevation differential of about 20 feet across the site. At the time of test drilling, surface vegetation consisted of sparse to moderate desert flora including

low grass and weeds, bushes, and small trees. Construction debris and fill dirt had been dumped in some areas, particularly in the lower basin area. The site is surrounded by residential developments including a mobile home park, apartments, and horse properties.

Investigation: Subsurface conditions at the basin were evaluated by drilling six test borings advanced with a CME-55 drill rig using 7-inch diameter, hollow-stem augers. During the field exploration, soils encountered were visually classified, and representative soil samples were obtained at selected depths. The boring logs are attached and test locations are shown on the site plan, also attached.

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

Tests: • DIRECT SHEAR
• pH VALUE
• SOLUABLE SALTS
• SULFATES
• RESISTIVITY

Test	Sample(s)	Purpose
Compression	Undisturbed (3)	Foundation settlement analyses
Expansion	Compacted sub-surface soil (2)	Expansion potential of excavated soils
Sieve Analysis and Plasticity Index	Representative surface soil (2)	Pavement design criteria
Dry Density and Moisture Content	Undisturbed (8)	In situ density and moisture determination to correlate engineering properties
Agronomy	Representative subsurface soil (4)	Horticultural evaluation

what about Resistivity. Direct-shear, pH value, soluable salts, sulfates

The results of the dry density and moisture content tests are presented on the graphical boring logs, and other test results are tabulated on the attached data sheets. The agronomy test results and recommendations are also appended.

Soil Conditions: As shown on the attached graphical boring logs, the soil profiles at test boring locations vary somewhat. At all test borings but Test Boring 2, surface soils consisted of sandy silty clay with low plasticity and stiff consistency. These soils were underlain at depths of 4 to 8 feet at all locations except Test Boring 3 by a clayey sand deposit of dense to very dense consistency with granitic sand particles. This materials was encountered at the surface at Test Boring 2. Below

the surface clay at Test Boring 3, the underlying soils below about 8 feet deep consisted of a silty sand of dense to very dense consistency. This material was coarser grained, with fine to medium gravel particles. Many of the coarse particles were vesicular basalt in various stages of weathering. The silty sand was also encountered below the clayey sand at a depth of about 22 feet at Test Boring 4. Soils were described as slightly damp to damp, and no groundwater was encountered in the test borings during drilling.

Foundations: Spread or mat foundations based at or below the bottom elevation of the detention basins appear suitable for supporting the various outlet/inlet and culvert structures. However, temporary inundation is likely and could induce some post-construction differential settlements as well as temporarily reduce the bearing capacity of supporting soils. Therefore, structures should be designed to accommodate some differential foundation movements.

The following tabulation presents foundation bearing design recommendations for footings and/or structure base slabs at selected depths. These values have been developed for buoyant conditions. The bearing materials should be either natural undisturbed soils or fill materials compacted as recommended in Parts II and III of this report. However, support of shallow footings on backfills of an adjoining buried structure is not recommended. Recommendations for other foundation conditions are possible and will be considered upon request.

Footing Type	Footing Depth	Allowable Foundation Bearing Pressure	Maximum Foundation Load Walls	Maximum Foundation Load Columns
Mat	0.5'	2000 psf	-	-
Mat	1.5'	2500 psf	-	-
Wall or Col.	1.5'	1500 psf	3 klf	20 kips
Wall or Col.	2.5'	2000 psf	5 klf	40 kips

Footing depth refers to the depth of the base of the footing below finish grade which is defined as structure floor level or basin bottom for interior footings or mats, and the lowest adjacent grade (either floor level or outside grade) within 5 feet for perimeter or exterior footings. All footing excavations should be observed by a representative of the geotechnical engineer to evaluate bearing conditions. If disturbed soils or other unsuitable bearing conditions are observed, the bearing level should be either stepped down to penetrate these undesirable materials or

the undesirable materials should be removed and be replaced with lean concrete or other materials as directed.

The recommended bearing pressures should be considered allowable maximums for dead plus design live loads, and may be increased by one-third when considering total loads including wind or seismic forces. The weight of the foundation concrete below subgrade may be neglected in dead load computations. Two (2.0) feet and 1.33 feet are recommended as the minimum width of isolated column and continuous footings, respectively, and mats should have a minimum dimension of 8 feet. At locations of grade change between adjoining structures, footings in the higher area should be positioned so that a surface projected downward at 45 degrees from the lower edge of the footing passes below the adjoining walls, foundations, backfills, etc., at the lower level.

Estimated foundation settlements for estimated structural loading conditions are on the order of 1/4 to 3/8 inch provided foundation bearing soils remain at normal moisture conditions. Additional post-construction differential foundation movements of comparable or slightly greater magnitude could be experienced if the natural bearing soils become wet after construction.

Lateral Design Parameters: The following tabulation presents recommendations for lateral stability analyses assuming compacted granular backfill. The values do not include compaction forces.

[1]Foundation Toe Pressures ..... 1.33 X allowable
[2]Lateral Backfill Pressures:
- Above Water Level:
  - Unrestrained walls ..... 35 psf/ft.
  - Rigid, permanently braced walls ..... 50 psf/ft.
- Below Water Level:
  - Unrestrained walls ..... 80 psf/ft.
  - Rigid, permanently braced walls ..... 92 psf/ft.

[3]Lateral Passive Pressures:
- Above Water Level:
  - Continuous walls/footings ..... 250 psf/ft.
  - Isolated columns/footings ..... 350 psf/ft.
- Below Water Level:
  - Continuous walls/footings ..... 125 psf/ft.
  - Isolated column/footings ..... 180 psf/ft.

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 pressures due to eccentric or lateral loading. The entire footing bearing surface should remain in compression.

[2] Equivalent fluid pressures for vertical walls and horizontal backfill surfaces (maximum 12-foot height). Pressures do not include temporary forces imposed during compaction of the backfill, swelling pressures developed by over-compacted clayey backfill, hydrostatic pressures from inundation of backfill, or surcharge loads. Walls should be suitably braced during backfilling to prevent damage and excessive deflection.

[3] Allowable values for confining soils below the base level of the detention basin, or on slopes for forces perpendicularly away from the basin.

Structural Backfills: Backfill behind structure walls should be compacted to density criteria presented later in this report. If backfills are not compacted as recommended, subsidence may result in areas adjoining backfilled subsurface walls or over utilities. Even properly compacted deep backfills may tend to settle differentially relative to subsurface walls and should not be used for support of adjoining facilities or utilities prone to damage from differential settlements.

Saturation of backfill and development of hydrostatic pressures is possible in below-grade areas due to infiltration of retained water through backfills. Backfills should consist of granular soils which exhibit low expansive potentials, although we recommend a clayey soil blanket at exterior, exposed backfill surfaces to impede water infiltration. Backfill compaction should be accomplished by mechanical methods. Water jetting or flooding of loose, dumped backfills must be prohibited in all structure backfills and in utility trench backfills within 10 feet of the structures.

On-Site Pavements: Parking areas to service public recreation facilities in the proposed basins may be added in the future. The following recommendations are provided for potential use in these parking areas. Site grading should be accomplished within pavement areas as recommended later in this report to

provide subgrade support for flexible pavements. A compacted subgrade of on-site soils or soils with comparable supporting properties is assumed.

Traffic Classification	Asphalt Concrete Surfacing	Granular Base Course
Automobile Parking	2.0"	7.0"

These pavement sections are considered minimal sections, but they are expected to function with some periodic maintenance or overlays where subgrades are compacted and drainage is provided and maintained. If the subgrade soils experience a significant increase in moisture content, accelerated pavement deterioration and increased maintenance costs should be anticipated. Base course or pavement materials should not be placed when the subgrade surface is wet. Good surface drainage should be provided away from the edges of paved areas to minimize lateral moisture transmission into the subgrade. The bituminous surfacing should be sealed after an initial summer of use of after weathering is apparent to minimize water infiltration directly through the pavement section.

Concrete Slab/Mat Foundation Support: Site grading should be accomplished within facility areas as recommended later in this report to provide subgrade support for concrete slabs-on-grade. A minimum 4 inch thickness of well graded sand and gravel (ABC) is recommended beneath all unreinforced, interior slabs-at-grade. In our opinion, base course is not required beneath reinforced concrete mat foundations on natural subgrades at in situ moisture content. Disturbed soils should be removed and either compacted to specified densities or be replaced prior to placement of base course.

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

Soils encountered in our test borings were non-cemented to lightly cemented, and can probably be removed with conventional excavating equipment. The soils primarily contained cohesive fines, although silty fines were encountered at Test Borings 3 and 4. Stability problems ranging from slope raveling to caving may occur in excavations which encounter random non-cohesive sand and gravel lenses or layers. All excavations should be braced or sloped as required to provide personnel safety and satisfy local safety code regulations.

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

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

Permanent Slopes: Low cut or fill slopes in site soils (height less than about 20 feet) which do not support or adjoin structures, roads, or other facilities should be no steeper than 2:1 (H:V.). The stability of slopes with greater height or which are used for structural support must be analyzed on an individual basis. Subgrade preparation and fill compaction for fill slopes should be performed as recommended in "Fill Materials" and "Site Grading". Fills should be constructed beyond the design slope surface and trimmed to final configuration. Erosion protection will be required for both cut and fill slopes.

Percolation Rates: Based on our experience in this area and the soil types, percolation rates are likely to be moderate to slow. Predicted percolation rates are on the order of 25 to 35 minutes per inch for bottom area seepage. Compaction or silting of basin surfaces from introduction of turbid water could significantly reduce seepage rates.

Fill Materials: All fill materials should be inorganic soils free of vegetation, debris, organic contaminants and fragments larger than +6 inches in size. Clayey site soils exhibit low to moderate expansion potentials when compacted. These soils may be used in embankment fills, and in fills below structures and concrete slabs; however, in areas below structures and concrete slabs these soils should be compacted at or above optimum moisture content as recommended in "Site Grading". These soils are not recommended for use in retaining wall backfill. Rather, granular soils meeting the requirements tabulated below are recommended.

Any imported fill or backfill materials for use within structure, concrete slab areas, and as retaining wall backfill should conform with the following specification requirements:

Maximum particle size	6 inches*
Maximum percent expansion	1.5**
Maximum percent passing 200 sieve	25***
Maximum plasticity index	5***

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

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

***Materials for structural wall backfill.

Site Grading: The following recommendations are presented for site grading within structure, concrete slab, and pavement areas. These recommended site grading procedures are intended to provide support for structural elements and pavement sections constructed on-grade. Therefore, all phases of earthwork should be performed under observation and testing directed by the geotechnical engineer.

1. Remove vegetation and organic contaminants, subsurface remnants of any former facilities, all surface fills, any backfills, and any unstable soils (loose, disturbed, etc.) from structure and pavement areas. Observe the cleared surface before and during subsequent scarification for evidences of debris-laden soils, disturbance, or loose zones requiring additional removal.

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

3. Scarify, moisture condition and compact exposed surface soils to a minimum 8-inch depth in areas beneath structures, concrete slabs, and pavements.

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

5. Compaction of cleaned exposed soil and each lift of backfill, subbase fill, and base course materials should be accomplished to the following density criteria:

Material	Percent Compaction (ASTM D698)
Cleaned Exposed Soil, Backfill, and Subbase Fill:	
Below foundation level:	
Less than 5 feet deep	95% min.
More than 5 feet deep	100% min.
Below concrete slabs above foundation level:	
On-site soils	90% min.
Imported soils	95% min.
Below asphalt paving	95% min.
*Miscellaneous Backfill	90% min.
Base Course:	
Below concrete slabs	95% min.
Below asphalt paving	100% min.

*Utility trench and exterior fill or backfill not intended for utility line, floor slab, foundation or pavement support.

Compaction of exposed site soils or fills of site soils within structure and concrete slab areas should be performed with soils uniformly mixed at a moisture content between optimum and optimum plus 3 percent. Compaction of imported fill soils with low expansive potentials should be accomplished at optimum content ±3 percent in areas beneath structures and exterior concrete slabs. Compaction of subgrade soil and fill material below asphaltic pavement should be accomplished at a moisture content 2 percent below optimum, or lower.

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

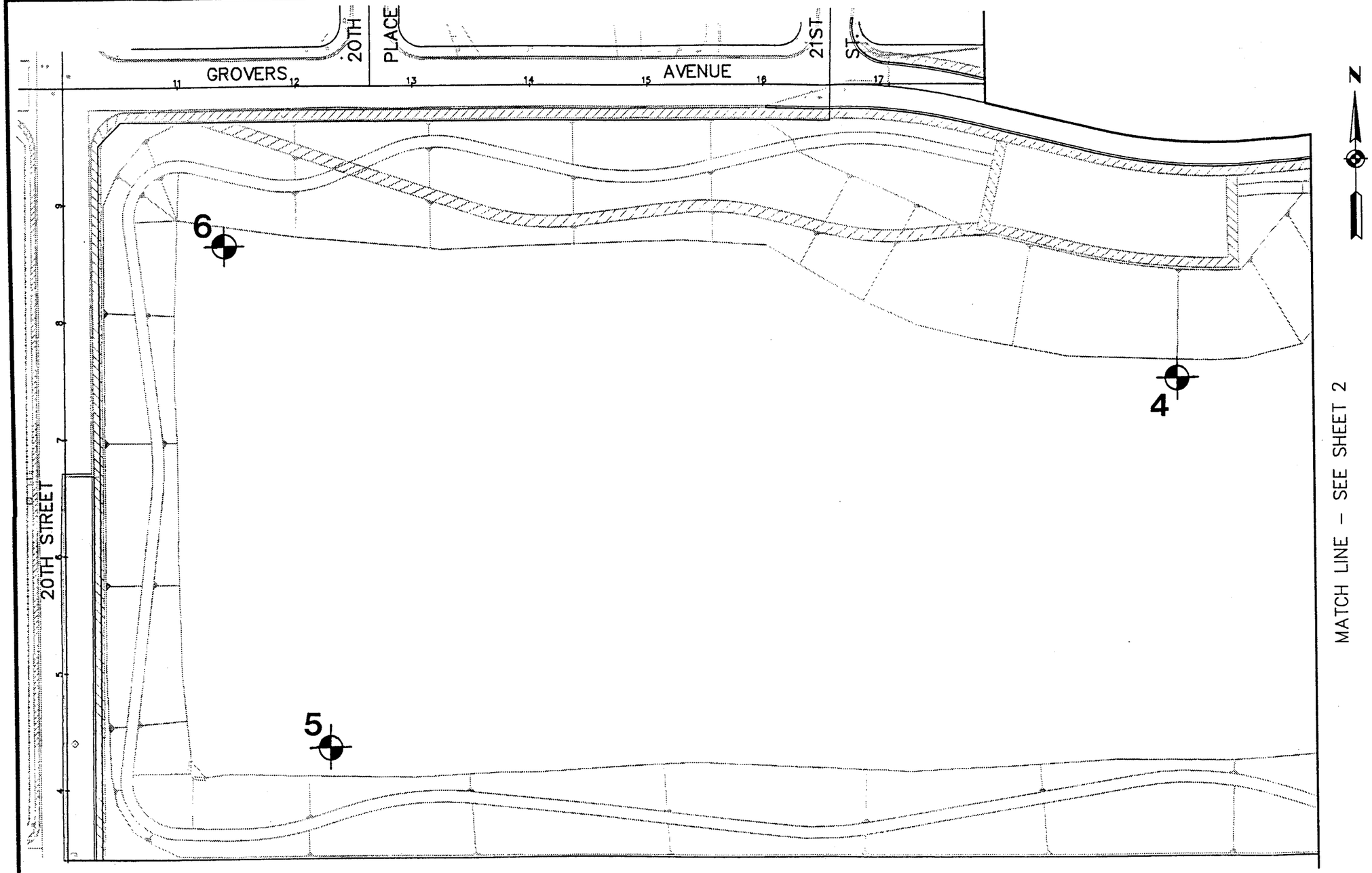
Please call if you have any questions or if we may be of further service.

Respectfully submitted,
THOMAS-HARTIG & ASSOCIATES, INC.

By: Kenneth D. Walsh — Reviewed by: Kenneth L. Ricker, P.E.

/dkl-s
Copies to: Addressee (5)

FIELD AND LABORATORY TEST RESULTS

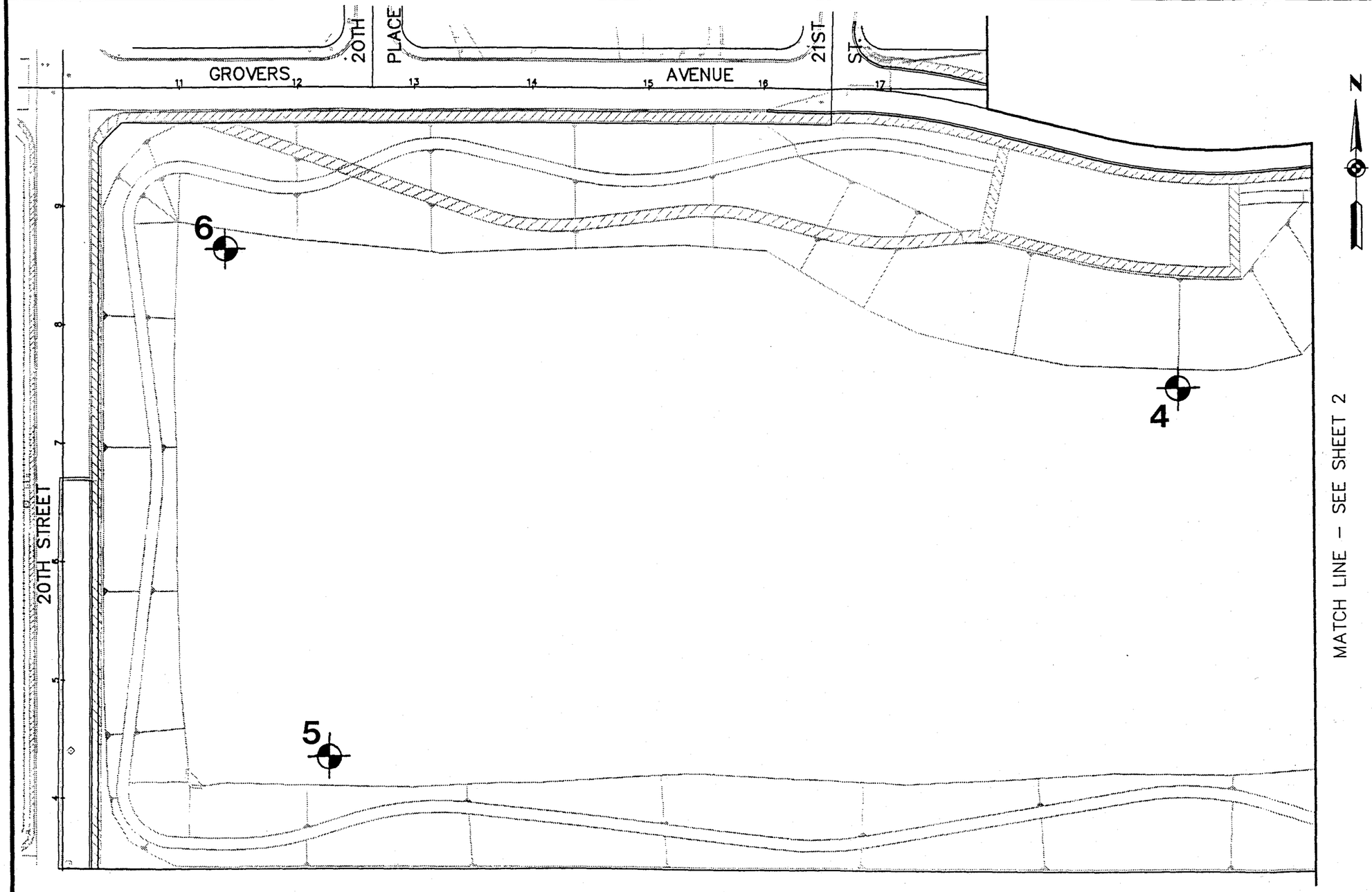


LEGEND

Test Boring Location
Elevation from topographic maps provided by NBS/Lowry.

Site Plan
Sheet 1 of 3
Project No. 91-0524

THOMAS-HARTIG & ASSOCIATES, INC.

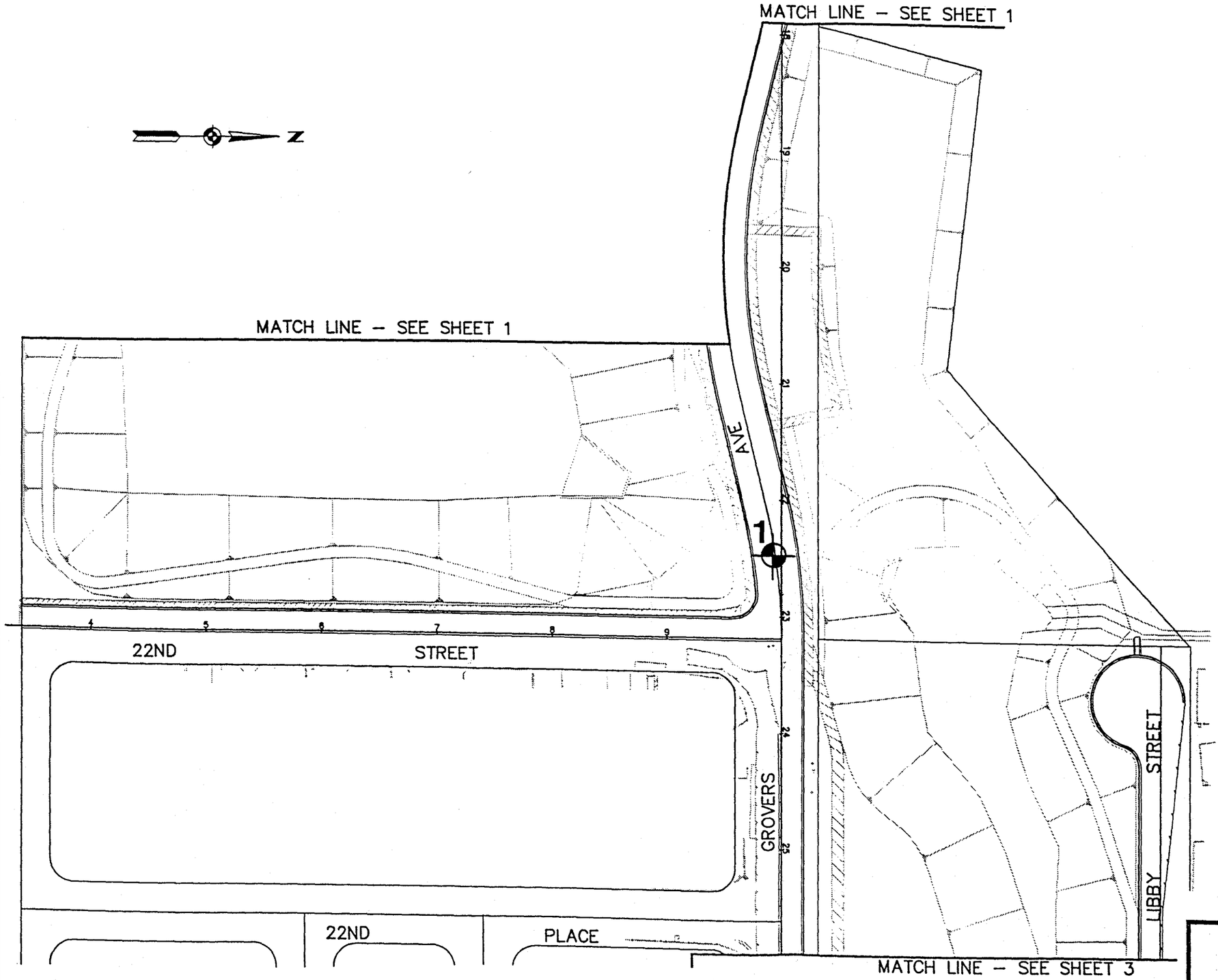


LEGEND

Test Boring Location
Elevation from topographic
maps provided by NBS/Lowry.

Site Plan
Sheet 1 of 3
Project No. 91-0524

THOMAS-HARTIG & ASSOCIATES, INC.

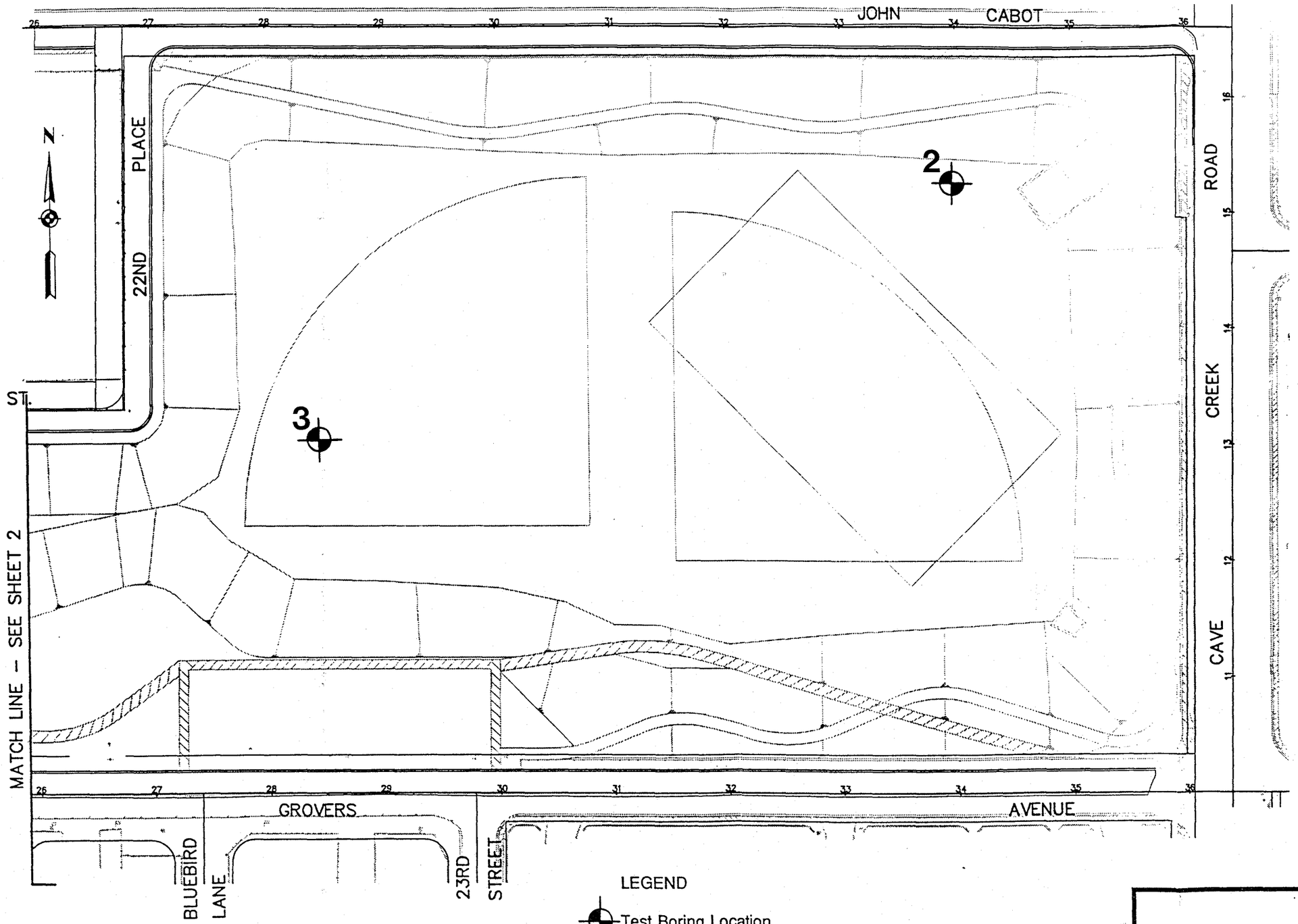


LEGEND

Test Boring Location
Elevation from topographic maps provided by NBS/Lowry.

Site Plan
Sheet 2 of 3
Project No. 91-0524

THOMAS-HARTIG & ASSOCIATES, INC.



LEGEND

Test Boring Location
Elevation from topographic maps provided by NBS/Lowry.

Site Plan
Sheet 3 of 3
Project No. 91-0524

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	

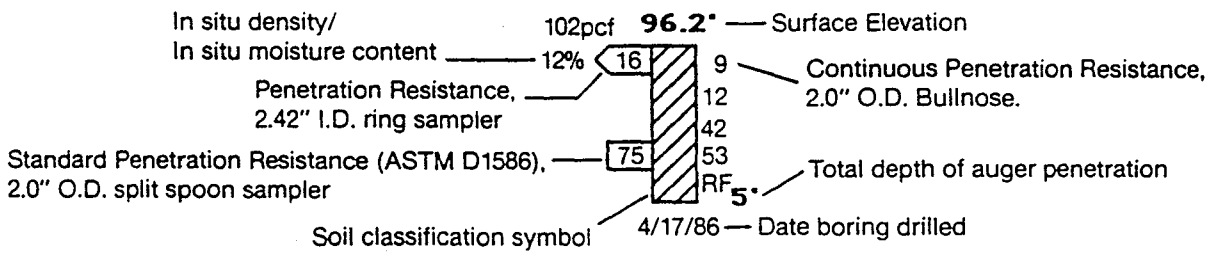
FINE-GRAINED SOIL

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				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).

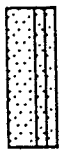
Project No. 91-0524

THOMAS-HARTIG & ASSOCIATES, INC.

LEGEND OF SOIL TYPES

SANDY SILTY CLAY (CL-ML); light brown to brown; stiff; with fine to medium subrounded to subangular sand; low plasticity; damp.

CLAYEY SAND (SC); light brown to reddish brown; dense to very dense; fine to coarse subrounded to subangular granitic sands; medium plasticity fines; stratified with traces to some fine to medium gravel; damp.



SILTY SAND (SM; SP; SP-SM); light brown to reddish brown; stiff; with fine to medium subrounded to subangular sand; low plasticity; damp.

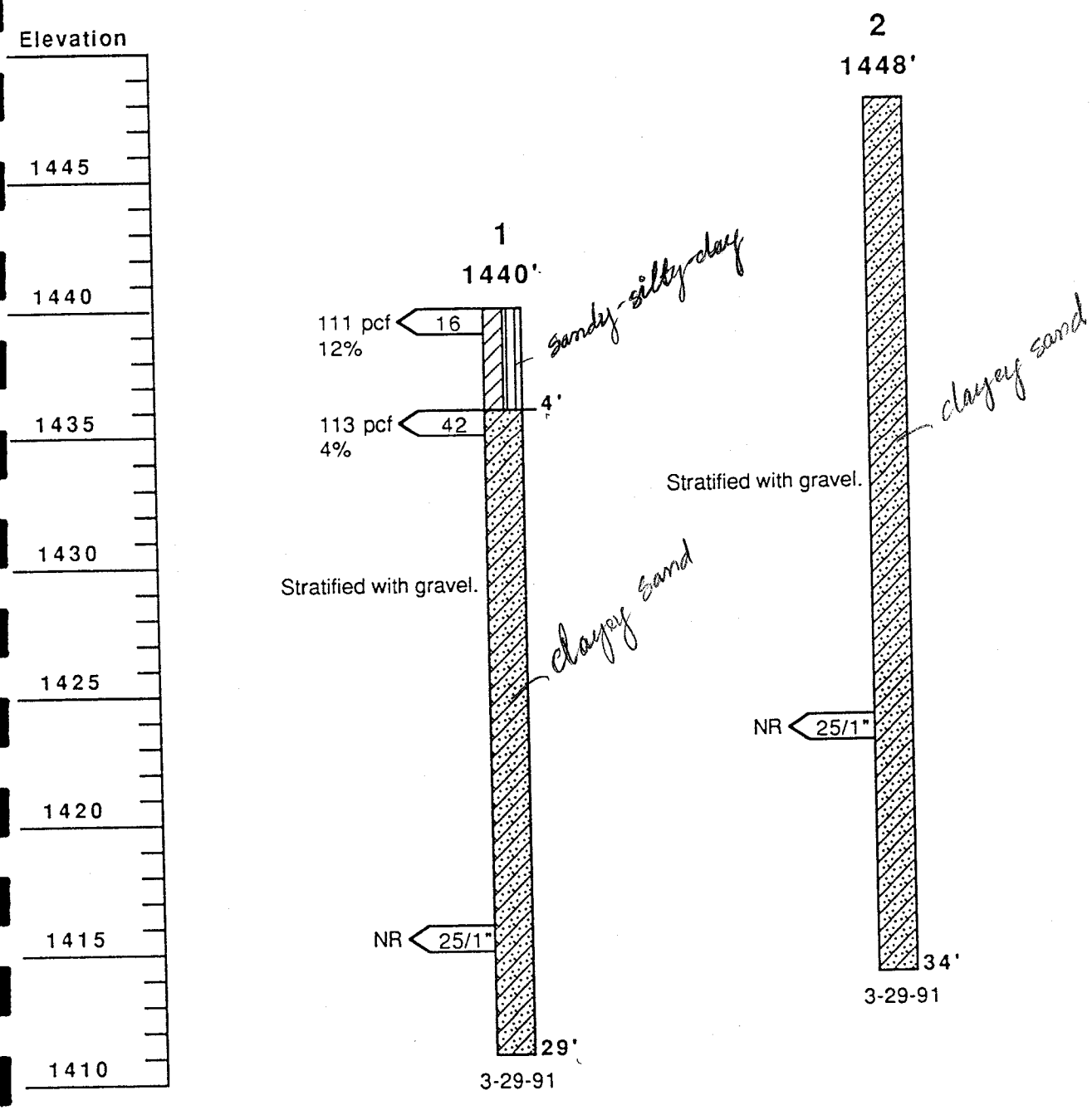
No free groundwater was encountered in any of the borings during drilling.

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

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

Project No. 91-0524
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



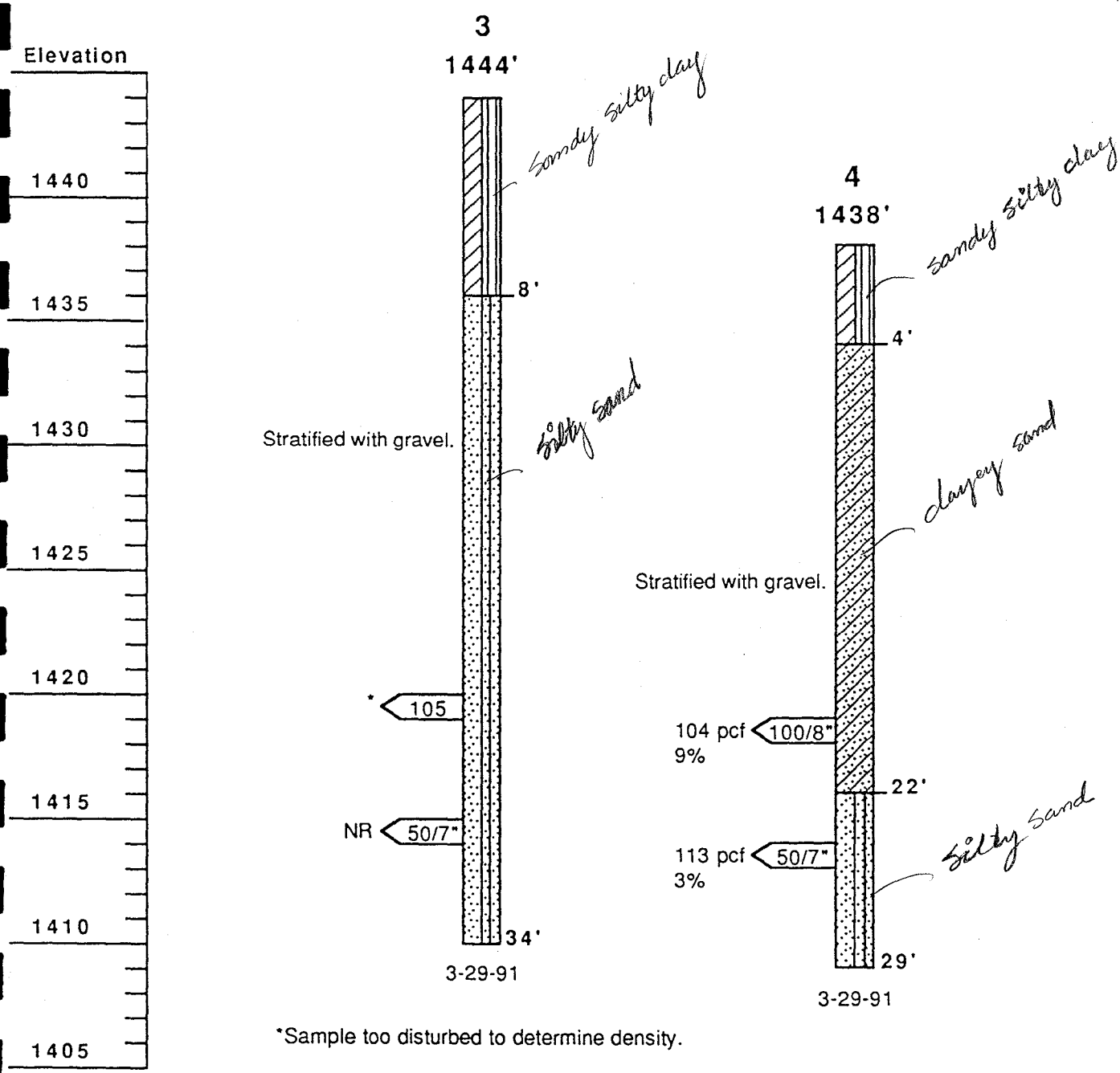
No free groundwater was encountered in any of the borings during drilling.

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

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

Project No. 91-0524
Thomas-Hartig & Associates

GRAPHICAL BORING LOGS



*Sample too disturbed to determine density.

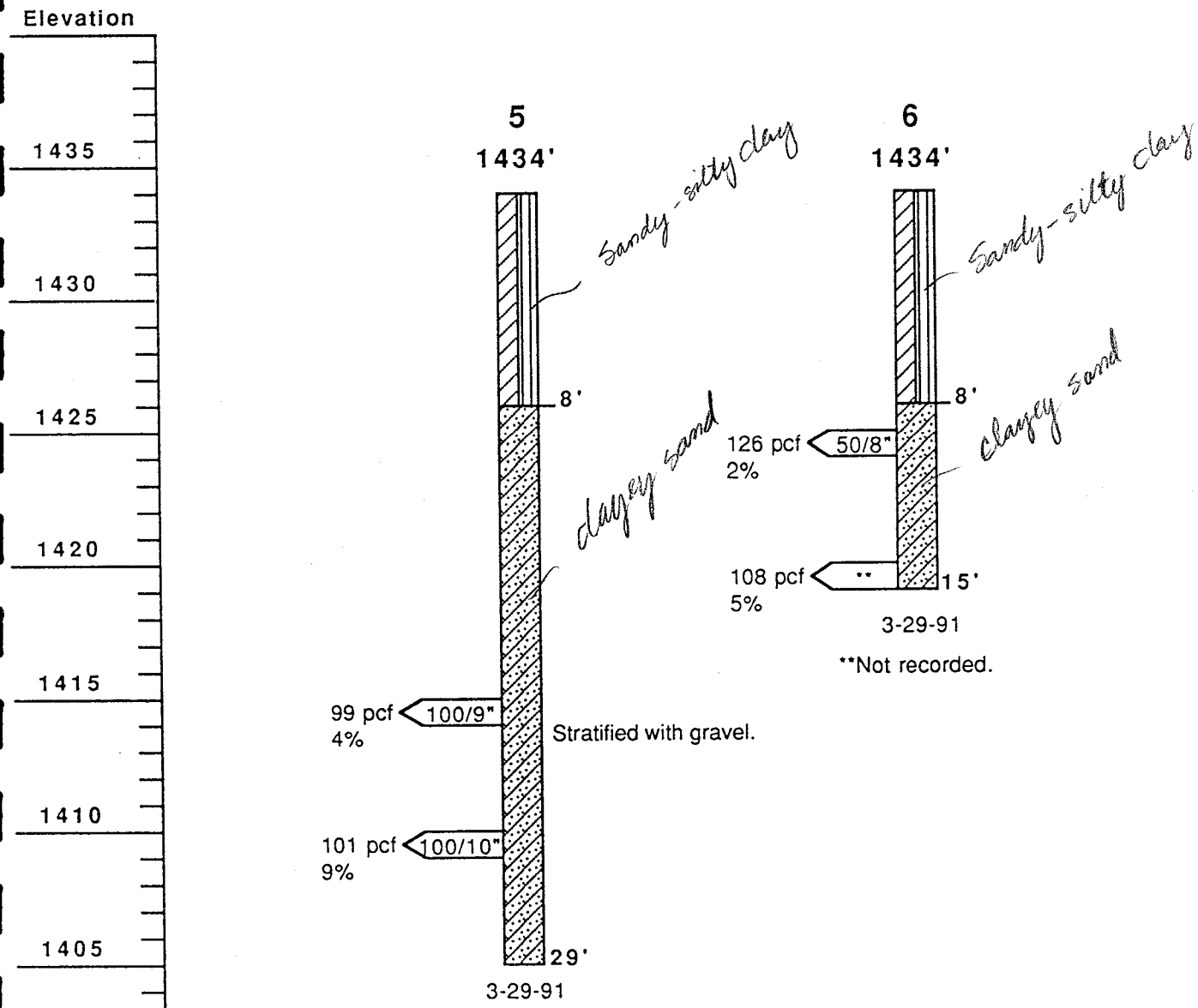
No free groundwater was encountered in any of the borings during drilling.

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

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

Project No. 91-0524
Thomas-Hartig & Associates

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.

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

Project No. 91-0524
Thomas-Hartig & Associates

REPORT ON GRADATION AND PLASTICITY INDEX

SAMPLE: Date: 4-4-91

Source: Noted Below

Type: Driven Ring Samples

Material: Subsurface Soil

Sampled By: TH/Thompson

TESTED: Sieve Analysis and Plasticity Index

50% passes #200

50% passes #8

RESULTS

			Sieve Size - Accumulative % Passing											*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
1; 0 - 1'	25	7	55	63	70	76	84	94	99	100				CL-ML
1; 4 - 5'	53	29	16	17	19	25	38	56	83	100				SC

*Unified Soil Classification

Project No. 91-0524

Thomas-Hartig & Associates, Inc.

REPORT ON REMOLDED EXPANSION TEST

SAMPLE: Date: 4-5-91

Source: Noted Below

Type: Bulk Sample

Material: Subsurface Soil

Sampled By: TH/Thompson

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

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
3; 0 - 8'	115	9	100	0.65
3; 18 - 26'	118	7	100	1.90

Project No. 91-0524

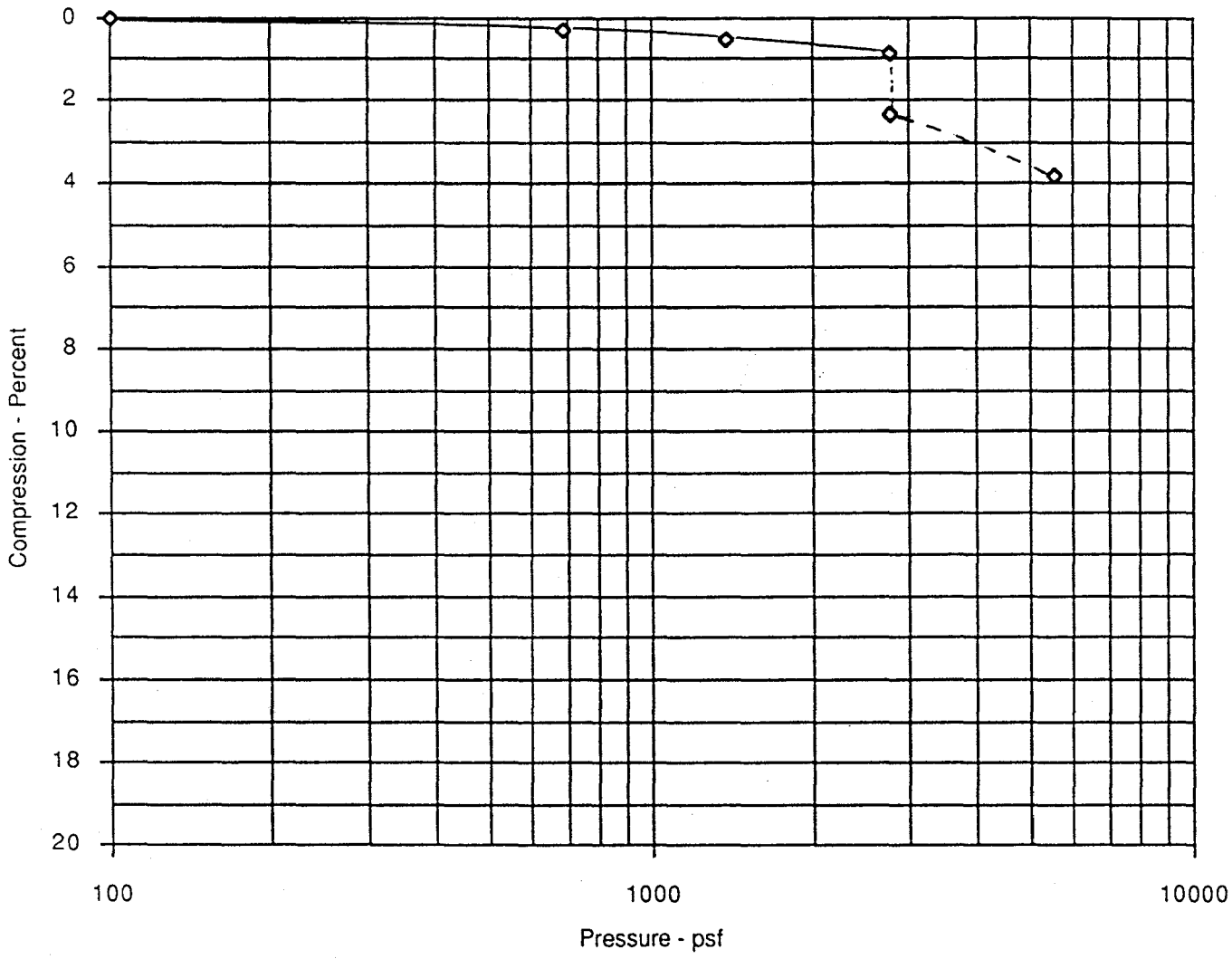
Thomas-Hartig & Associates, Inc.

REPORT ON COMPRESSION TESTS

SAMPLE: Date: 4-4-91

Source: Test Boring 4; 24' - 25'
Type: Driven Ring Sample; 113 pcf Dry Density; 3% Field Moisture
Material: Silty Sand (SM)
Sampled by: TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 91-0524

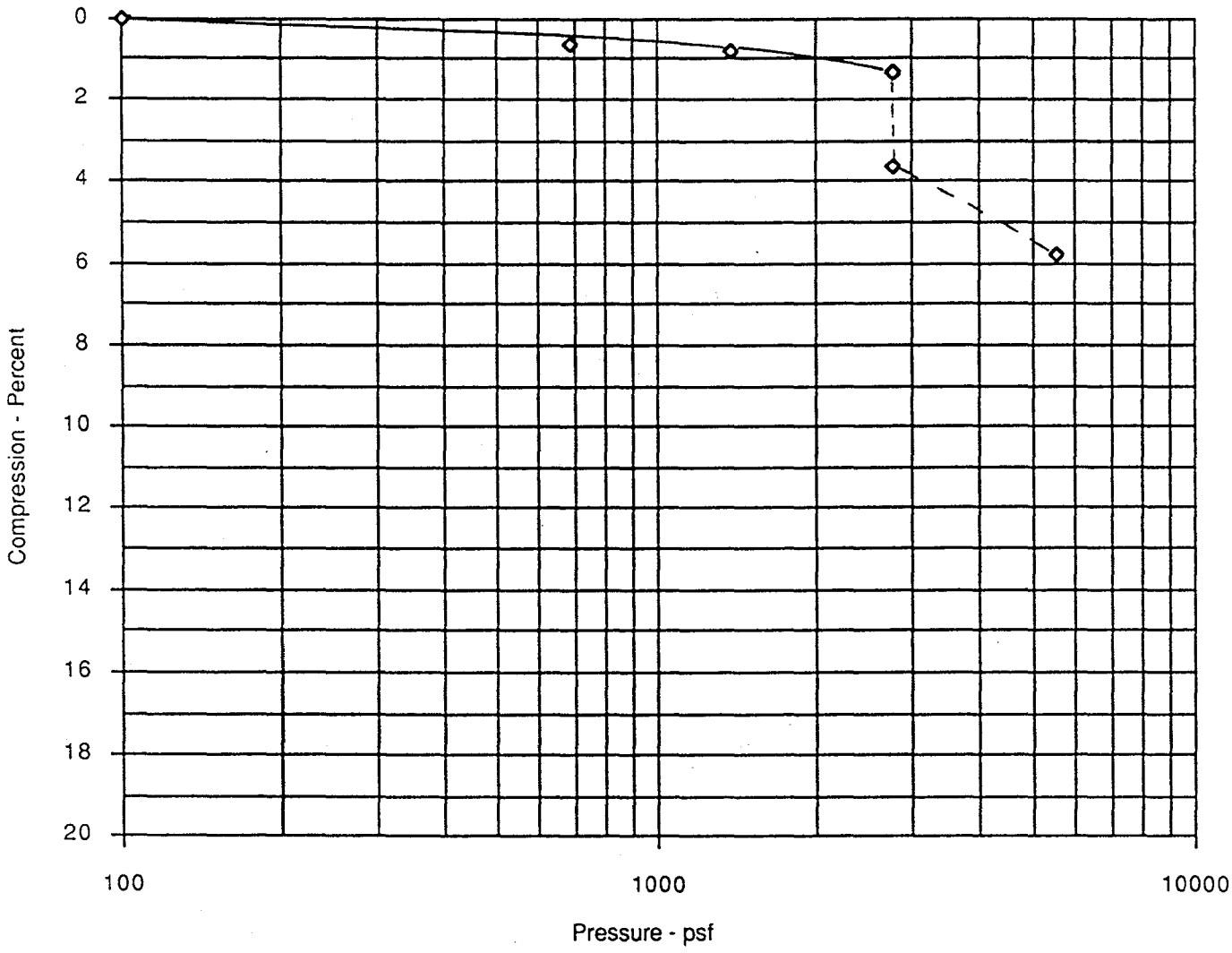
Thomas-Hartig & Associates, Inc.

REPORT ON COMPRESSION TESTS

SAMPLE: Date: 4-4-91

Source: Test Boring 5; 19' - 20'
Type: Driven Ring Sample; 99 pcf Dry Density; 4% Field Moisture
Material: Clayey Sand (SC)
Sampled by: TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 91-0524

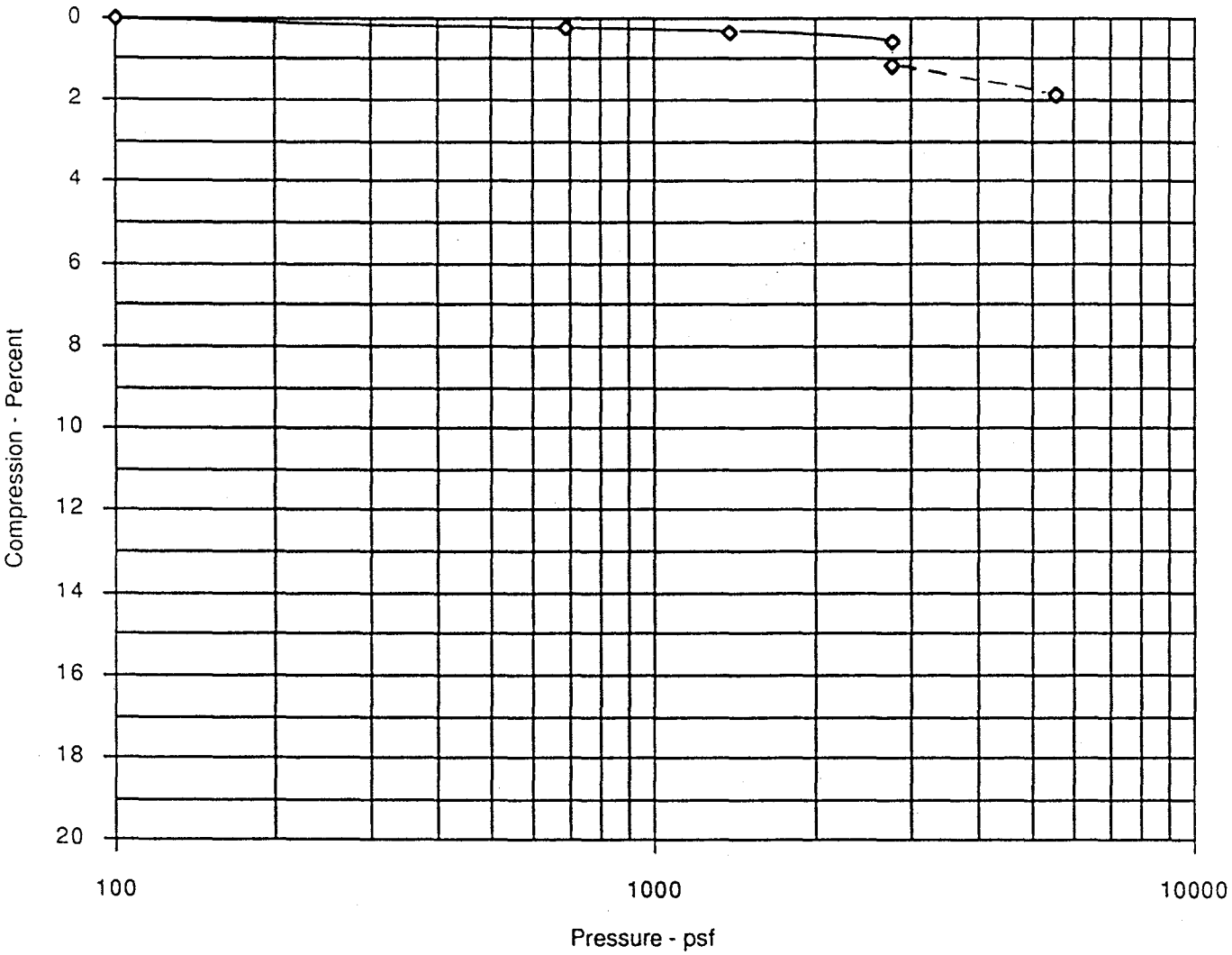
Thomas-Hartig & Associates, Inc.

REPORT ON COMPRESSION TESTS

SAMPLE: Date: 4-4-91

Source: Test Boring 6; 9' - 10'
Type: Driven Ring Sample; 126 pcf Dry Density; 2% Field Moisture
Material: Clayey Sand (SC)
Sampled by: TH/Thompson

TESTED: Compression; test sample soaked at 2770 psf



Project No. 91-0524

Thomas-Hartig & Associates, Inc.

RECEIVED APR 9 1991

Laboratory Consultants, Ltd.

April 4, 1991

Thomas-Hartig & Associates
7031 West Oakland Street
Chandler, Arizona 85226

RE: Soil analysis for 91-521; laboratory #91-HS-3707 to 91-HS-3710

Based on the enclosed reports of analysis, the pH levels are normal for this area, but high enough to affect nutrient availability to plants. To lower to a more desirable range, apply 20 pounds sulfur per 1000 square feet preplant. Incorporate and water in thoroughly.

The salinity levels are low to medium and not damaging to turf or ornamentals.

The exchangeable sodium percentages are within an acceptable range indicating that water permeability problems and/or foliar burn due to excess sodium will not occur.

The organic matter content of these soils is low. To annual beds/planters, apply 20 cubic yards per 1000 square feet nitrogen stabilized organic material to the surface and incorporate into the top 12 inches of soil. A light surface mulch (1/4-1/2 inch deep) will suffice for lawn areas. These applications are not mandatory, but will increase water and nutrient holding capacity and improve soil structure.

Of the nutrients tested, nitrogen, phosphorus, and zinc were found to be deficient. To correct, apply 2 pounds nitrogen, 2 to 5 pounds phosphorus and 1/2 pound zinc (as sulfate) per 1000 square feet preplant. Incorporate and water in thoroughly. To maintain nitrogen levels, apply 2 pounds per 1000 square feet in June to trees and shrubs. To annuals and ground covers (other than turf), apply 1 to 1 1/2 pounds per 1000 square feet every 4 to 6 weeks during the active growing season. To lawn areas, apply 1 pound per 1000 square feet once per month during the active growing season. Water in applications to incorporate. Phosphorus can be split with nitrogen applications.

5861 South Kyrene, Suite 15 Tempe, Arizona 85283 Telephone (602) 491-9655

If you have any questions or I can be of further assistance, please call at your convenience.

Respectfully submitted,

Sharon Murray

Sharon Murray
Horticulturist

Laboratory Consultants, Ltd.
5861 South Kyrene, Suite 15 Tempe, Arizona 85283
Telephone (602) 491-9655

SUBMITTED BY: THOMAS-HARTIG
ADDRESS:

GROWER:
LAB NO.: 91-524
91HS3707

SOIL FERTILITY REPORT

SAMPLE MARKING	2 @ 20-23'	CROP	ORNAMENTAL	DATE RECEIVED	4/1/91
		YIELD GOAL	0 N/A	DATE REPORTED	4/4/91

SATURATION PERCENTAGE		SOIL pH	8.2	FREE LIME	YES	ELECTRICAL CONDUCTIVITY mmhos/cm	1.0 VL	EXCHANGEABLE SODIUM PERCENTAGE	8.0

LAB TESTS	RESULT	SUGGESTED RECOMMENDATIONS		FERTILIZATION PLAN
ORGANIC MATTER, %	0.2			lbs/A
ORGANIC NITROGEN	ppm / lbs/A: 36		SIDEDRESS	lbs/A
			TOPDRESS	lbs/A
NITRATE - NITROGEN Depth, in. 0 - 36	ppm 7 / lbs/A 76		STARTER	lbs/A
			FERTIGATION	lbs/A
AVAILABLE NITROGEN	112	N	3 * lbs/A	TOTAL lbs/A
AVAILABLE PHOSPHORUS — BRAY P1, ppm		P_2O_5	lbs/A	**BROADCAST**
AVAILABLE PHOSPHORUS — BICARBONATE P, ppm	18 M		2 * lbs/A	
EXCHANGEABLE POTASSIUM, ppm	271 VH	K_2O	0 * lbs/A	
SULFATE - SULFUR, ppm Depth, in. 0 - 36	10	S	0 * lbs/A	
EXCHANGEABLE MAGNESIUM, ppm	937 VH	MgO	0 * lbs/A	
EXCHANGEABLE CALCIUM, ppm	2264 M			
EXCHANGEABLE SODIUM, ppm	392 H			
AVAILABLE ZINC, ppm	0.5 L	Zn	4 ** lbs/A	**STARTER**
AVAILABLE COPPER, ppm	0.7 H	Cu	0 ** lbs/A	
AVAILABLE IRON, ppm	19.2 H	Fe	0 ** lbs/A	
AVAILABLE MANGANESE, ppm	11.0 H	Mn	0 ** lbs/A	
AVAILABLE BORON, ppm	0.5 L	B	0 ** lbs/A	
LIME REQUIREMENT		Ag - Lime (60% effectiveness)	Tons/A	
GYPSUM REQUIREMENT		Gypsum (100% basis)	Tons/A	

SOIL TEXTURE		% SAND		% SILT		% CLAY	

* = lbs. per 1000 sq. ft. ** = ozs. per 1000 sq. ft

LOW MEDIUM HIGH VERY HIGH

BY: [illegible signature]

PLEASE NOTE SPECIAL COMMENTS ON BACK

Laboratory Consultants, Ltd.
5861 South Kyrene, Suite 15 Tempe, Arizona 85283
Telephone (602) 491-9655

SUBMITTED BY: THOMAS-HARTIG
ADDRESS:

GROWER: 91-524
LAB NO.: 91AS3708

SOIL FERTILITY REPORT

SAMPLE MARKING	3 @ 16-18'	CROP	ORNAMENTAL	DATE RECEIVED	4/1/91
		YIELD GOAL	0 N/A	DATE REPORTED	4/4/91

SATURATION PERCENTAGE		SOIL pH	7.9	FREE LIME	YES	ELECTRICAL CONDUCTIVITY mmhos/cm	0.65 VL	EXCHANGEABLE SODIUM PERCENTAGE	1.7

LAB TESTS		RESULT		SUGGESTED RECOMMENDATIONS		FERTILIZATION PLAN	
ORGANIC MATTER, %		0.3					lbs/A
ORGANIC NITROGEN			31	SIDEDRESS			lbs/A
NITRATE - NITROGEN	Depth, in. 0 - 24	4	29	TOPDRESS			lbs/A
				STARTER			lbs/A
				FERTIGATION			lbs/A
AVAILABLE NITROGEN		60	N	4 *	lbs/A	TOTAL	lbs/A
AVAILABLE PHOSPHORUS	BRAY P1, ppm		P_2O_5		lbs/A	BROADCAST	
	BICARBONATE P, ppm	2 VL		5 *	lbs/A		
EXCHANGEABLE POTASSIUM, ppm		319 VH	K_2O	0 *	lbs/A		
SULFATE - SULFUR, ppm	Depth, in. 0 - 24	7	S	1 *	lbs/A		
EXCHANGEABLE MAGNESIUM, ppm		513 VH	MgO	0 *	lbs/A		
EXCHANGEABLE CALCIUM, ppm		3044 H					
EXCHANGEABLE SODIUM, ppm		79 L					
AVAILABLE ZINC, ppm		0.5 L	Zn	4 **	lbs/A	STARTER	
AVAILABLE COPPER, ppm		0.8 H	Cu	0 **	lbs/A		
AVAILABLE IRON, ppm		18.2 H	Fe	0 **	lbs/A		
AVAILABLE MANGANESE, ppm		10.9 H	Mn	0 **	lbs/A		
AVAILABLE BORON, ppm		1.4 H	B	0 **	lbs/A		
LIME REQUIREMENT			Ag - Lime (60% effectiveness)		Tons/A		
GYPSUM REQUIREMENT			Gypsum (100% basis)		Tons/A		

SOIL TEXTURE	% SAND	% SILT	% CLAY

* = lbs. per 1000 sq. ft. ** = ozs. per 1000 sq. ft

PLEASE NOTE SPECIAL COMMENTS ON BACK

Laboratory Consultants, Ltd.
5861 South Kyrene, Suite 15 Tempe, Arizona 85283
Telephone (602) 491-9655

SUBMITTED BY: THOMAS-HARTIG
ADDRESS:

GROWER:
LAB NO.: 91-524
91AS3709

SOIL FERTILITY REPORT

SAMPLE MARKING	5	CROP	ORNAMENTAL	DATE RECEIVED	4/1/91
	@ 15-18'	YIELD GOAL	0 N/A	DATE REPORTED	4/4/91

SATURATION PERCENTAGE		SOIL pH	8.3	FREE LIME	YES	ELECTRICAL CONDUCTIVITY mmhos/cm	1.6 M	EXCHANGEABLE SODIUM PERCENTAGE	10.0

LAB TESTS		RESULT		SUGGESTED RECOMMENDATIONS		FERTILIZATION PLAN	
ORGANIC MATTER, %		0.4					lbs/A
ORGANIC NITROGEN		ppm	lbs/A 63		SIDEDRESS		lbs/A
NITRATE - NITROGEN	Depth, in. 0 - 36	ppm 4	lbs/A 43		TOPDRESS		lbs/A
					STARTER		lbs/A
					FERTIGATION		lbs/A
AVAILABLE NITROGEN		106		N	3 * lbs/A	TOTAL	lbs/A
AVAILABLE PHOSPHORUS	BRAY P1, ppm			P_2O_5	lbs/A	BROADCAST	
	BICARBONATE P, ppm	2 VL			5 * lbs/A		
EXCHANGEABLE POTASSIUM, ppm		195 H		K_2O	0 * lbs/A		
SULFATE - SULFUR, ppm	Depth, in. 0 - 36	43		S	0 * lbs/A		
EXCHANGEABLE MAGNESIUM, ppm		983 VH		MgO	0 * lbs/A		
EXCHANGEABLE CALCIUM, ppm		1979 M					
EXCHANGEABLE SODIUM, ppm		474 H					
AVAILABLE ZINC, ppm		0.2 L		Zn	4 ** lbs/A	STARTER	
AVAILABLE COPPER, ppm		0.4 H		Cu	0 ** lbs/A		
AVAILABLE IRON, ppm		9.1 H		Fe	0 ** lbs/A		
AVAILABLE MANGANESE, ppm		5.0 H		Mn	0 ** lbs/A		
AVAILABLE BORON, ppm		1.2 H		B	0 ** lbs/A		
LIME REQUIREMENT				Ag - Lime (60% effectiveness)	Tons/A		
GYPSUM REQUIREMENT				Gypsum (100% basis)	Tons/A		

SOIL TEXTURE	% SAND	% SILT	% CLAY

* = lbs. per 1000 sq. ft. ** = ozs. per 1000 sq. ft

BY: [illegible]

PLEASE NOTE SPECIAL COMMENTS ON BACK

Laboratory Consultants, Ltd.
5861 South Kyrene, Suite 15 Tempe, Arizona 85283
Telephone (602) 491-9655

SUBMITTED BY: THOMAS-HARTIG
ADDRESS:

GROWER: 91-524
LAB NO.: 91AS3710

SOIL FERTILITY REPORT

SAMPLE MARKING	6 @ 11-14'	CROP	ORNAMENTAL	DATE RECEIVED	4/1/91
		YIELD GOAL	0 N/A	DATE REPORTED	4/4/91

SATURATION PERCENTAGE		SOIL pH	8.2	FREE LIME	YES	ELECTRICAL CONDUCTIVITY mmhos/cm	0.70 VL	EXCHANGEABLE SODIUM PERCENTAGE	2.9

LAB TESTS		RESULT	SUGGESTED RECOMMENDATIONS		FERTILIZATION PLAN
ORGANIC MATTER, %		0.8			lbs/A
ORGANIC NITROGEN		ppm / lbs/A 131		SIDEDRESS	lbs/A
				TOPDRESS	lbs/A
NITRATE - NITROGEN	Depth, in. 0 - 36	ppm / lbs/A 8 86		STARTER	lbs/A
				FERTIGATION	lbs/A
AVAILABLE NITROGEN		217	N	0.5 * lbs/A	TOTAL lbs/A
AVAILABLE PHOSPHORUS	BRAY P1, ppm		P_2O_5	lbs/A	**BROADCAST**
	BICARBONATE P, ppm	1 VL		5 * lbs/A	
EXCHANGEABLE POTASSIUM, ppm		243 VH	K_2O	0 * lbs/A	
SULFATE - SULFUR, ppm	Depth, in. 0 - 36	14	S	0 * lbs/A	
EXCHANGEABLE MAGNESIUM, ppm		957 VH	MgO	0 * lbs/A	
EXCHANGEABLE CALCIUM, ppm		2476 M			
EXCHANGEABLE SODIUM, ppm		144 M			
AVAILABLE ZINC, ppm		0.3 L	Zn	4 ** lbs/A	**STARTER**
AVAILABLE COPPER, ppm		0.3 H	Cu	0 ** lbs/A	
AVAILABLE IRON, ppm		10.1 H	Fe	0 ** lbs/A	
AVAILABLE MANGANESE, ppm		3.9 H	Mn	0 ** lbs/A	
AVAILABLE BORON, ppm		0.8 M	B	0 ** lbs/A	
LIME REQUIREMENT			Ag - Lime (60% effectiveness)	Tons/A	
GYPSUM REQUIREMENT			Gypsum (100% basis)	Tons/A	

SOIL TEXTURE		% SAND		% SILT		% CLAY	

* = lbs. per 1000 sq. ft. ** = ozs. per 1000 sq. ft

VL = VERY LOW L = LOW M = MEDIUM H = HIGH VH = VERY HIGH

BY: [illegible]

PLEASE NOTE SPECIAL COMMENTS ON BACK