Mine Permit Renewal Application

APPENDICES
SECTIONS R614-301-742.221a thru 760

VALLEY CAMP OF UTAH, INC.

SCOFIELD ROUTE * HELPER, UTAH 84526 * PHONE [801] 448-9454

A3

APPENDIX R614-301-742.221a

Sediment Pond Calculations and Details

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I - Curve Number Selection -

A - Hydrologic Soil Group

Ref: "Soils of Utah" - Agriculture Experiment Station - Bulletin 492, Utah State University, Logan, Utah. March 1975

Soils at the Utah No 2 Mine are defined as:

Soil	Hydrologic Soil Group	
Typic Argiborolls	C	Generally B & C for deeper soils and B for shallow soils.
Lithic Argiborolls		
Typic Haploborolls	B	

Soils at the Belina Mine are defined as:

Soil	Hydrologic Soil Group	
Argic Cryoborolls	B & C	Mainly B & C
Pachic Cryoborolls	A, B & C	
Cryic Paleborolls	B & C	

Assume hydrologic soil group C for the whole area which should be conservative.

B - Vegetative cover and CN for Sage Grass Areas & Forest Aspen Areas.

In undisturbed areas the vegetative cover consists of Sage/Grass cover complex and a Forest/Aspen cover complex. To be conservative assume the ground cover density to be only 25%.

From the attached figures taken from "Design of Small Dams" USBR (1977)

CN ≈ 75 for Sage/Grass, Soil Group C, Density 25%

CN ≈ 70 for Oak-Aspen, Soil Group C, Density 25%

C - Disturbed areas were assumed to be equivalent roads (dirt) (hard surface) in Table A-2 attached, taken from "Design of Small Dams," again assuming Hydrologic Soil Group C.

CN = 90 for Disturbed Areas.

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CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Curve Number Selection
PROJECT NO.

SHEET 2 OF 4
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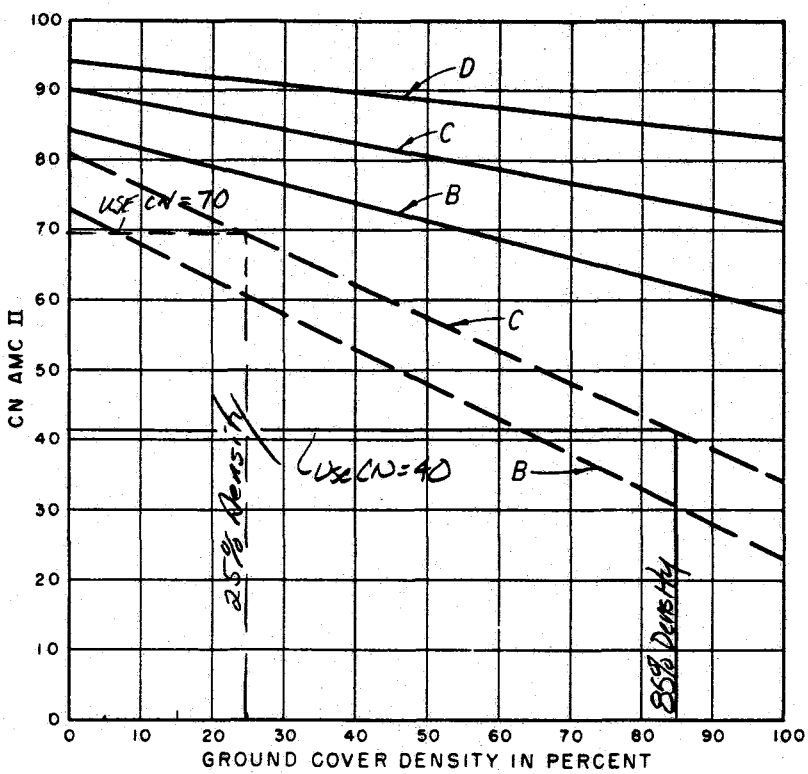

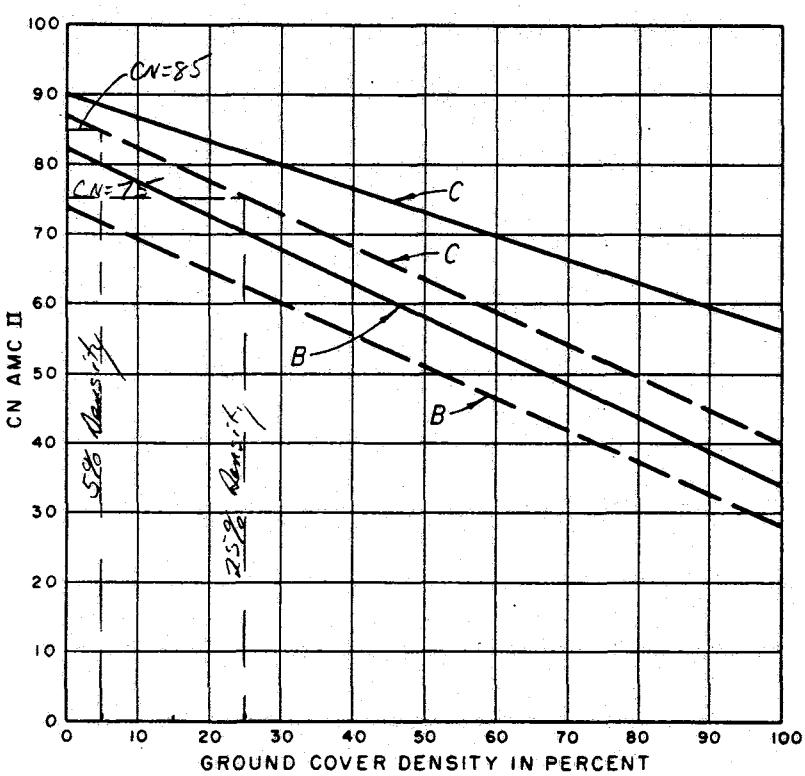

Figure A–3. Determining CN for forest-range in the western United States. 288–D–2824.

Taken from "Design of Small Dams" USBR (1977)

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TABLE A-2.—*Runoff curve numbers (CN) for hydrologic soil-cover complexes*
(FOR WATERSHED CONDITION AMC-II AND $I_a=0.2S$)[3]

Taken from "Design of Small Dams" USBR (1977)

Land use or cover	Treatment or practice	Hydrologic condition for infiltrating	Hydrologic soil group			
			A	B	C	D
Fallow	SR		77	86	91	94
Row crops	SR	Poor	72	81	88	91
	SR	Good	67	78	85	89
	C	Poor	70	79	84	88
	C	Good	65	75	82	86
	C&T	Poor	66	74	80	82
	C&T	Good	62	71	78	81
Small grain	SR	Poor	65	76	84	[illegible]
	SR	Good	63	75	83	[illegible]
	C	Poor	63	74	82	[illegible]
	C	Good	61	73	81	84
	C&T	Poor	61	72	79	82
	C&T	Good	59	70	78	81
Close-seeded legumes[1] or rotation meadow.	SR	Poor	66	77	85	89
	SR	Good	58	72	81	85
	C	Poor	64	75	83	85
	C	Good	55	69	78	83
	C&T	Poor	63	73	80	83
	C&T	Good	51	67	76	[illegible]
Pasture or range		Poor	68	79	86	[illegible]
		Fair	49	69	79	84
		Good	39	61	74	80
	C	Poor	47	67	81	88
	C	Fair	25	59	75	83
	C	Good	6	35	70	79
Meadow (permanent).		do	30	58	71	78
Woods (farm woodlots).		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	[illegible]
Farmsteads			59	74	82	[illegible]
Roads (dirt)[2] (hard surface).[2]			72	82	87	89
			74	84	90	92

[1] Close-drilled or broadcast.
[2] Including right-of-way.
[3] See sec. A-5.
SR = Straight row.
C = Contoured.
T = Terraced.
C&T = Contoured and terraced.

(U.S. Soil Conservation Service.)

D - Reclaimed Areas were assumed to have a Sage-Grass cover with only 5% ground cover density.

CN = 85 for sage/Grass, 5% cover density, and hydr. by soil group C.

E - Paved Areas were assumed to be nearly impervious

CN = 98 was assumed.

II - Summary

Area Description	CN
Forest/Aspen - Low density	70
Sage/Grass	75
Disturbed Areas	90
Reclaimed	85
Paved Areas	98
Forest/Aspen - High Density	40 * Value used in Runoff Calc's for selected Ditches & Culverts

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Runoff & Peak Flow Projections
PROJECT NO.

SHEET 1 OF 8
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I- Drainage Basin Characteristics

A- Basin Areas

Sed Pond	Disturbed Area	Paved Area	Forested; Aspen Area	Sage/Grass Area	Reclaimed Area	Total
001A	8.13	—	—	3.4	—	11.53
002A	6.3	0.82	—	—	—	7.12
003A	4.0	0.18	1.65	1.0	0.25	7.08
004A	17.3	—	15.1	—	4.7	37.1

B- Curve Numbers

Use:
CN = 90 - Disturbed Areas
CN = 98 - Paved Areas
CN = 70 - Forest & Aspen
CN = 75 - Sage - Grass
CN = 85 - Reclaimed Areas.

} See Calculations 9/18/88 for Justification.

Volume Wt. Curve #5 -
10yr-24hr Precip = 2.45 inches
25yr-24hr Precip = 2.92 inches

Pond	Area ac	CN	Runoff Depth Q inches 10yr 24hr.	Runoff Depth Q inches 25yr 24hr	Wt. CN Based on Volume
001A	8.13	90	1.486	1.911	86.4 ← 10yr ← Use
	3.4	75	0.622	0.909	86.3 ← 25yr
		WtQ-	1.23	1.616	Use 86.4
002A	6.3	90	1.486	1.911	91.2 ← 10yr.
	0.82	98	2.221	2.689	91.0 ← 25yr.
		WtQ	1.571	2.001	Use 91.2
003A	4.0	90	1.486	1.911	84.8 ← 10yr.
	0.18	98	2.221	2.689	84.5 ← 25yr
	1.65	70	0.432	0.670	Use 85.0
	1.0	75	0.622	0.909	
	0.25	85	1.139	1.521	
		WtQ	1.125	1.486	
004A	17.3	90	1.486	1.911	82.3 ← 10yr.
	15.1	70	0.432	0.670	82.6 ← 25yr
	4.7	85	0.854	1.521	Use 83.0
		WtQ	0.977	1.356	

C - Hydraulic Length ; Watershed Slope

Sed Pond	Hydraulic Length ft	Watershed Slope %
001A	2760	38.0
002A	860	17.8
003A	1100	32.
004A	2040	40.

II - Runoff Volume - 10yr-24hr event

P = 2.45 inches.

Sediment Pond	Area acres	Wt. CN	Runoff Depth in	Runoff Volume ac-ft
001A	11.53	86.4	1.229	1.18
002A	7.12	91.2	1.581	0.94
003A	7.08	85.0	1.139	0.67
004A	37.1	83.0	1.018	3.15

III - Peak Flow - 25-year 24-Hour Event

Sed Pond	Peak cfs See attached Printouts
001A	18.3
002A	13.7
003A	11.0
004A	52.0

PROJECT : VALLEY CAMP SEDIMENT POND 001A: 25-YR, 24-HR PEAK INFLOW

AREA= 11.5 ACRES
AVERAGE BASIN SLOPE= 38.0 PERCENT
CURVE NUMBER= 86.4
DESIGN STORM= 2.92 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 2760. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1041 HOURS QPCFS= 83.80 CFS QPIN= 7.2073 INCHES
C3= 35.5247 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
7.35	.3143	.0000	.0000	.0	.00
7.37	.3156	.0000	.0000	4.2	.00
7.39	.3169	.0000	.0000	26.0	.00
7.41	.3183	.0000	.0000	55.6	.00
7.43	.3196	.0000	.0000	76.9	.00
7.45	.3209	.0000	.0000	83.8	.00
7.47	.3223	.0000	.0000	78.5	.00
7.49	.3236	.0000	.0000	66.3	.00
7.51	.3248	.0000	.0000	51.8	.00
7.53	.3259	.0000	.0000	38.2	.00
7.55	.3270	.0000	.0000	27.0	.00
7.58	.3281	.0001	.0000	18.3	.00
7.60	.3292	.0001	.0000	12.1	.00
7.62	.3303	.0002	.0000	7.7	.00
7.64	.3314	.0002	.0000	4.9	.00
7.66	.3325	.0002	.0000	3.0	.00
7.68	.3336	.0002	.0000	1.8	.00
7.70	.3346	.0002	.0000	1.1	.01
7.72	.3357	.0003	.0000	.6	.01
7.74	.3368	.0003	.0000	.4	.01
7.76	.3379	.0003	.0000	.2	.01
7.78	.3390	.0004	.0000	.1	.01
7.80	.3401	.0004	.0000	.0	.01
11.88	1.6770	.6319	.0327	.0	16.72
11.90	1.7232	.6651	.0331	.0	17.03
11.93	1.7694	.6986	.0335	.0	17.33
11.95	1.8156	.7325	.0339	.0	17.61
11.97	1.8617	.7667	.0343	.0	17.88
11.99	1.9079	.8013	.0346	.0	18.13
12.01	1.9394	.8251	.0238	.0	18.32
12.03	1.9481	.8318	.0066	.0	18.19
12.05	1.9569	.8384	.0066	.0	17.33
12.07	1.9656	.8451	.0067	.0	15.70
12.09	1.9744	.8517	.0067	.0	13.60

PROJECT : VALLEY CAMP SEDIMENT POND 001A: 25-YR, 24-HR PEAK INFLOW
(Continued)

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
12.11	1.9832	.8584	.0067	.0	11.40
12.13	1.9919	.8651	.0067	.0	9.41

HYDROGRAPH PEAK= 18.32 cfs
TIME TO PEAK= 12.01 Hours
RUNOFF VOLUME= 1.56 Acre-Feet

PROJECT : VALLEY CAMP SEDIMENT POND 002A: 25-YR, 24-HR PEAK INFLOW

AREA= 7.1 ACRES
AVERAGE BASIN SLOPE= 17.8 PERCENT
CURVE NUMBER= 91.2
DESIGN STORM= 2.92 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 860. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0495 HOURS QPCFS= 108.75 CFS QPIN=15.1465 INCHES
C3= 74.6572 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
5.19	.1920	.0000	.0000	.0	.00
5.21	.1932	.0000	.0000	33.8	.00
5.23	.1943	.0000	.0000	99.8	.00
5.25	.1955	.0000	.0000	101.9	.00
5.27	.1966	.0000	.0000	67.3	.00
5.29	.1978	.0000	.0000	35.0	.00
5.31	.1989	.0000	.0000	15.7	.00
5.33	.2001	.0000	.0000	6.3	.00
5.35	.2012	.0000	.0000	2.4	.00
5.37	.2024	.0000	.0000	.8	.00
5.39	.2036	.0001	.0000	.3	.00
5.41	.2047	.0001	.0000	.0	.00
11.88	1.6698	.8932	.0370	.0	13.21
11.90	1.7137	.9304	.0372	.0	13.31
11.92	1.7577	.9679	.0374	.0	13.40
11.94	1.8016	1.0055	.0377	.0	13.49
11.96	1.8456	1.0434	.0379	.0	13.58
11.98	1.8895	1.0814	.0381	.0	13.66
12.00	1.9334	1.1197	.0383	.0	13.74
12.02	1.9438	1.1288	.0091	.0	12.82
12.04	1.9521	1.1360	.0073	.0	9.89
12.06	1.9605	1.1433	.0073	.0	6.77
12.08	1.9688	1.1506	.0073	.0	4.63
12.10	1.9771	1.1579	.0073	.0	3.50
12.12	1.9854	1.1652	.0073	.0	2.98

HYDROGRAPH PEAK= 13.74 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= 1.20 Acre-Feet

PROJECT : VALLEY CAMP SEDIMENT POND 003A: 25-YR. 24-HR PEAK INFLOW

AREA= 7.1 ACRES
AVERAGE BASIN SLOPE= 32.0 PERCENT
CURVE NUMBER= 85.0
DESIGN STORM= 2.92 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 1100. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0571 HOURS QPCFS= 93.76 CFS QPIN=13.1325 INCHES
C3= 64.7302 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
8.03	.3522	.0000	.0000	.0	.00
8.04	.3532	.0000	.0000	8.9	.00
8.05	.3542	.0000	.0000	45.9	.00
8.07	.3552	.0000	.0000	61.6	.00
8.08	.3562	.0000	.0000	93.8	.00
8.10	.3572	.0000	.0000	84.9	.00
8.11	.3582	.0000	.0000	66.1	.00
8.13	.3592	.0000	.0000	46.4	.00
8.14	.3602	.0000	.0000	30.1	.00
8.15	.3612	.0000	.0000	18.5	.00
8.17	.3622	.0000	.0000	10.8	.00
8.18	.3632	.0000	.0000	6.1	.00
8.20	.3642	.0000	.0000	3.3	.00
8.21	.3652	.0000	.0000	1.8	.00
8.23	.3662	.0000	.0000	.9	.00
8.24	.3672	.0001	.0000	.5	.00
8.25	.3682	.0001	.0000	.2	.00
8.27	.3692	.0001	.0000	.1	.00
8.28	.3702	.0002	.0000	.0	.00
11.91	1.7353	.6072	.0216	.0	10.36
11.92	1.7670	.6290	.0218	.0	10.47
11.94	1.7987	.6510	.0220	.0	10.58
11.95	1.8304	.6732	.0222	.0	10.69
11.97	1.8620	.6956	.0224	.0	10.80
11.98	1.8937	.7182	.0226	.0	10.90
12.00	1.9254	.7410	.0227	.0	10.99
12.01	1.9400	.7514	.0105	.0	10.98
12.02	1.9460	.7558	.0043	.0	10.44
12.04	1.9520	.7601	.0043	.0	9.23
12.05	1.9580	.7645	.0044	.0	7.63
12.07	1.9640	.7689	.0044	.0	6.05
12.08	1.9700	.7732	.0044	.0	4.74

HYDROGRAPH PEAK= 10.99 cfs
TIME TO PEAK= 12.00 Hours
RUNOFF VOLUME= .90 Acre-Feet

PROJECT : VALLEY CAMP SEDIMENT POND 004A: 25-YR, 24-HR PEAK INFLOW

AREA= 37.1 ACRES
AVERAGE BASIN SLOPE= 40.0 PERCENT
CURVE NUMBER= 83.0
DESIGN STORM= 2.92 INCHES
STORM DURATION= 24.0 HOURS
HYDRAULIC LENGTH= 2040. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0896 HOURS QPCFS= 312.99 CFS QPIN= 8.3664 INCHES
C3= 41.2379 ITERATIONS= 8 SCS 24-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
8.77	.4088	.0000	.0000	.0	.00
8.78	.4104	.0000	.0000	15.7	.00
8.80	.4120	.0000	.0000	97.2	.00
8.82	.4135	.0000	.0000	207.8	.00
8.84	.4151	.0000	.0000	287.3	.00
8.86	.4167	.0000	.0000	313.0	.00
8.87	.4182	.0000	.0000	293.2	.00
8.89	.4198	.0000	.0000	247.5	.00
8.91	.4214	.0000	.0000	193.6	.01
8.93	.4230	.0000	.0000	142.8	.02
8.95	.4245	.0001	.0000	100.7	.02
8.96	.4261	.0001	.0000	68.4	.02
8.98	.4277	.0002	.0000	45.0	.03
9.00	.4292	.0002	.0000	28.9	.03
9.02	.4309	.0002	.0000	18.1	.04
9.04	.4326	.0003	.0000	11.2	.04
9.05	.4343	.0003	.0000	6.8	.05
9.07	.4359	.0003	.0000	4.0	.05
9.09	.4376	.0004	.0000	2.4	.06
9.11	.4393	.0004	.0000	1.4	.07
9.13	.4410	.0005	.0000	.8	.07
9.14	.4426	.0005	.0000	.5	.08
9.16	.4443	.0006	.0000	.3	.08
9.18	.4460	.0006	.0000	.1	.09
9.20	.4477	.0007	.0000	.0	.09
11.90	1.7236	.5135	.0248	.0	47.30
11.92	1.7634	.5387	.0252	.0	48.20
11.94	1.8032	.5642	.0255	.0	49.06
11.96	1.8430	.5901	.0259	.0	49.89
11.98	1.8828	.6163	.0262	.0	50.69
11.99	1.9225	.6427	.0265	.0	51.46
12.01	1.9410	.6551	.0124	.0	51.98
12.03	1.9485	.6602	.0051	.0	51.17
12.05	1.9560	.6653	.0051	.0	48.13

PROJECT : VALLEY CAMP SEDIMENT POND 004A: 25-YR, 24-HR PEAK INFLOW
(Continued)

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
12.07	1.9636	.6704	.0051	.0	43.06
12.08	1.9711	.6755	.0051	.0	36.94
12.10	1.9786	.6806	.0051	.0	30.81
12.12	1.9862	.6857	.0051	.0	25.40

HYDROGRAPH PEAK= 51.98 cfs
TIME TO PEAK= 12.01 Hours
RUNOFF VOLUME= 4.27 Acre-Feet

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Mean Annual Sediment Loadings
PROJECT NO. 7 10-1

SHEET 1 OF 4
COMPUTED GJP
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DATE Sep 20, 1988

10-yr 24-hr runoff volumes

POND	Tributary Area (acres)	Composite CN	10-yr 24-hr Runoff (inches)	Runoff Volume (Ac Ft)	+ 2 yr SED	+ 3 yr sed
001A	11.5	86.4	1.23	1.18	0.62	0.93
002A	7.1	91.2	1.58	0.94	.06	0.10
003A	7.1	85	1.14	0.67	0.34	0.51
004A	37.1	83	1.02	3.15	0.50	.75

revised w/ add. of 1.2 acres sage grass 1989

Sediment Volumes

Modified Universal Soil Loss Equation

$$A = R \cdot K \cdot LS \cdot VM$$

reference: "Erosion and Sedimentation in Utah: A Guide for Control" Israelsen et al 1984

where
A = soil loss per unit area (tons/acre/year)
R = rainfall factor
K = soil erodibility factor
LS = topographic factor
VM = erosion control factor

R (rainfall factor):

ref. PRICE ISO-ERODENT (R) VALUES 1983

Mean Annual R = 15 at VC load out site
= 19 at Belina Mine

K soil erodibility factor

map for state of Utah indicates that the Valley Camp Load Out facility is about on the border between soil erodibility types — choose the more conservative range:
K = 0.21 to 0.3 use 0.3

LS see Table 2
λ_n = sum of slope segment length =

VM
Undisturbed: Aspen/grass 60% groundcover 0.012
Sage/grass 60-80% 0.039
Disturbed: use VM = 1.0 ref. Red Book Appendix 5A

HANSEN ALLEN & LUCE inc

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Mean Annual Sediment Loadings
PROJECT NO. 7-10-1

SHEET 2 OF 4
COMPUTED GJP
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DATE Sep 20, 1988

POND 001 A

- ignore conveyance deposition (conservative approach)
- assume a unit weight of sediment of 75 lb/ft³
- R = 15 K = 0.3 ⇒ R·K = 4.5

Sub Area	area (acres)	l ft	S %	LS	VM	A (4.5·LS·VM) t/ac/yr	Sed tons/yr	Sed AF/yr	
Sage/Grass	3.4	120	62%	.28	.039	4.9	16.7		
Disturbed									
(1) steep slope	1.9	80	62%	[1] 37.5	1.0	168.8	320.7		
(2) old portal yd	2.9	50	6%	[2] 1.4	1.0	6.3	18.3		
(3) rest	2.9	90	40	12.0	1.0	54.0	156.6		
							512.3	0.31	ok

1) $LS = \frac{(L\lambda_n S_{s_n})\lambda_n - (L\lambda_{n-1} S_{s_n})\lambda_{n-1}}{l_n} = \frac{29(165) - 21(85)}{80}$

2) $LS = \frac{(1.0)215 - (0.87)(165)}{50} = 1.43$

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Mean Annual Sediment Loadings
PROJECT NO. 7-10-1

SHEET 3 OF 4
COMPUTED GJP
CHECKED
DATE Sep 20, 198

Pond 002A

$A = RKLSVM$

- $R = 15 \quad K = 0.3 \Rightarrow R \cdot K = 4.5$

Sub Area	area (acres)	l (ft)	S %	LS	VM	A	SED, tons/yr	Sed AF/yr
① steep	0.7	60	33%	7.30	1.0	32.8	23.0	
② channel/sides	0.6	40	38%	7.0	1.0	31.5	18.9	
③ pads etc	4.8	130	3%	0.3	1.0	1.35	6.48	
							48.4	
Pond + Pond	1.0				ϕ		ϕ	
	7.1						48.4	0.03

Pond 003A

Sub Area	area (acres)	l (ft)	S %	LS	VM	A	SED tons/yr	SED AF/yr
SAGE/Grass	1.0	400	29%	15.9	.039	2.8	2.8	
ASPEN	1.65	250	35%	16.16	.012		0.9	
Disturbed								
① Steep	0.4	27	100%	1) 102	1.0	459	184	
② steep below trail	0.7	95	50%	17.4	1.0	78.3	54.8	
③ upper pads	0.7	55'	4%	0.32	1.0	1.44	1.0	
④ to bottom ditch	0.7	85'	30%	7.3	1.0	32.9	23.0	
⑤ rest	1.5	200'	8.5%	1.53	1.0	6.89	10.3	
⑥ Far South Area	0.25	90'	20%	3.98	1.0	17.91	4.5	
							282.5	0.17 AF/yr

1) $LS = \dfrac{(L\lambda_n S_{s_n})\lambda_n - (L\lambda_{n-1} S_{s_n})\lambda_{n-1}}{l_n} = \dfrac{(70)277 - 66.8(250)}{27}$

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Mean Annual Sediment Loadings
PROJECT NO. 7-10-1

SHEET 4 OF 4
COMPUTED GJP
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DATE Nov 8, 1988

Pond 004A

$A = RKLSVM$

$R = 15 \quad K = 0.3 \Rightarrow RK = 4.5$

Sub Area	Area (acres)	l (ft)	S %	LS	VM	A	Sed ton/yr	Sed AF/yr
D4 seg 1	0.9	42	139%	27	.02	2.43	2.2	
seg 2	2.5	540	6%	1.57	1.3	9.18	22.9	
	3.4							
D5	0.8	102	54%	20.5	0.9	83.0	66.4	
D6	3.7	110	45%	15.6	0.01	0.7	2.6	
D7 seg 1	0.8	50	90%	26.9	0.9	108.9	87.1	
seg 2	2.0	440	4%	0.73	1.3	4.27	8.5	
	2.8							
D8	1.0	100	40%	12.65	0.01	0.57	0.6	
D9 seg 1	0.7	170	47%	21.6	0.9	87.48	2.9	
seg 2	0.14	55	200%	233	0.02	20.97		
seg 3	4.7	360	5%	1.0	1.3	5.85	27.5	
	5.5							
D10 seg 1	0.6	170	47%	21.5	0.9	87.0	52.2	
seg 2	2.4	430	8.1%	2.0	1.3	11.7	28.0	
	3.0							
D11	0.8	180	47%	22.0	1.3	128.7	103.0	
U4	7.2	480	39%	23.0	.009	0.93	6.7	
U5	5.3	700	22%	12.62	.009	0.51	2.7	
U6	2.6	206	49%	25.3	.01	1.14	3.0	0.25

416 ton/yr

$$\text{Sediment Volume} = 416\ \text{t/yr}\ \frac{2000\ \text{lb}}{\text{t}}\ \frac{1\ \text{ft}^3}{75\ \text{lb}} \times \frac{1}{43560}$$

$$= 0.25\ \text{AF}$$

STAGE VOLUME RELATIONSHIP
POND 001A

AFTER CLEANING (OCTOBER 1988)
POND NPDES 001A

ELEV. FT	STAGE FT	AREA FT2	DEL VOL AF	SUM VOL AF	COMMENTS
7803	0	207	0	0	
7804	1	1050	0.01	0.01	
7805	2	3050	0.05	0.06	
7806	3	4880	0.09	0.15	
7807	4	6630	0.13	0.28	
7808	5	7750	0.17	0.45	
7809	6	9360	0.20	0.65	0.81 – 60% Sed. Cleanout
7810	7	10350	0.23	0.87	
7811	8	10670	0.24	1.11	
7812	9	11000	0.25	1.36	
7813	10	13000	0.28	1.64	
7814	11	14400	0.31	1.95	
7815	12	15416	0.34	2.29	
7816	13	16300	0.36	2.66	Exist. Primary spillway (18
7817	14	17500	0.39	3.05	EL-7816.1
7818	15	18800	0.42	3.46	
7819	16	20000	0.45	3.91	Exist. Emergency spillway (
7820	17	20900	0.47	4.38	EL - 7818.8'
7821	18	22100	0.49	4.87	
7821.5	18.5	22650	0.26	5.13	

7809.73

POND 001A AFTER CLEANING

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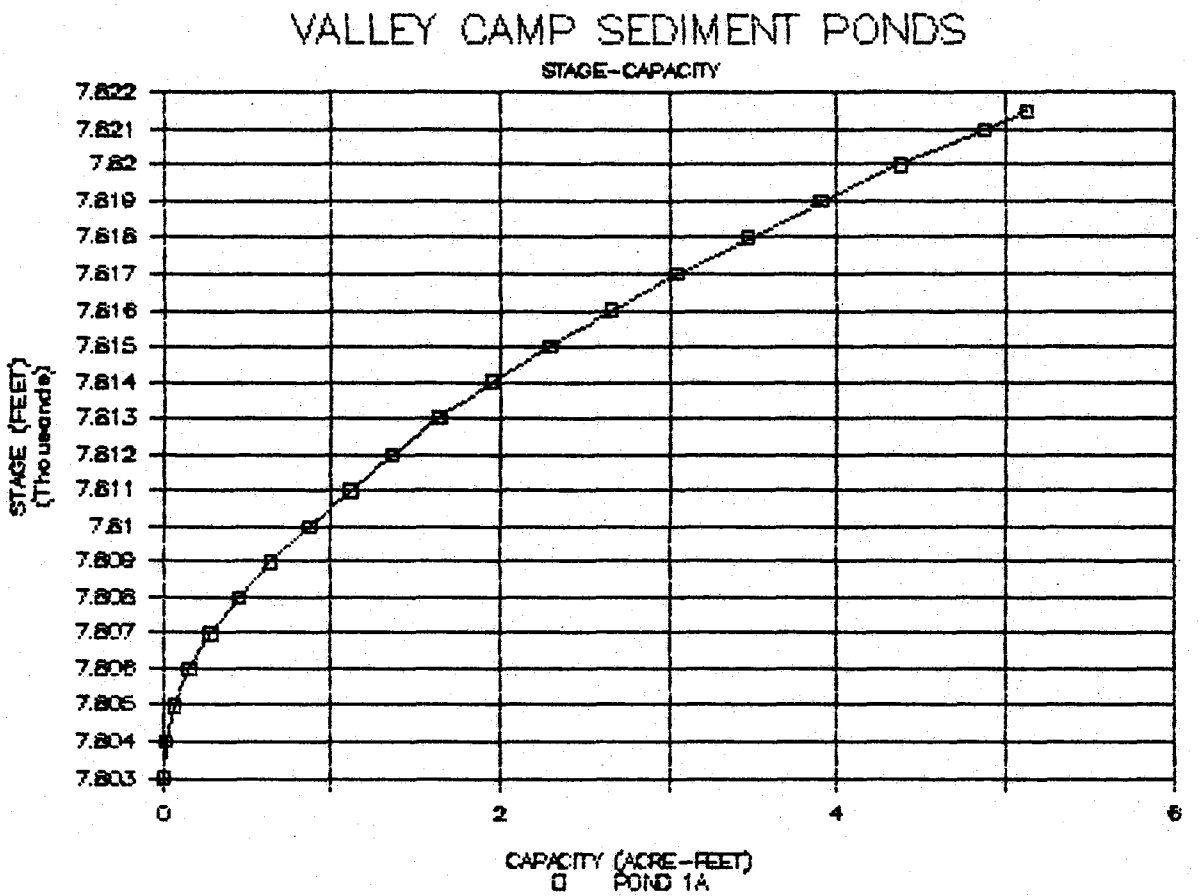
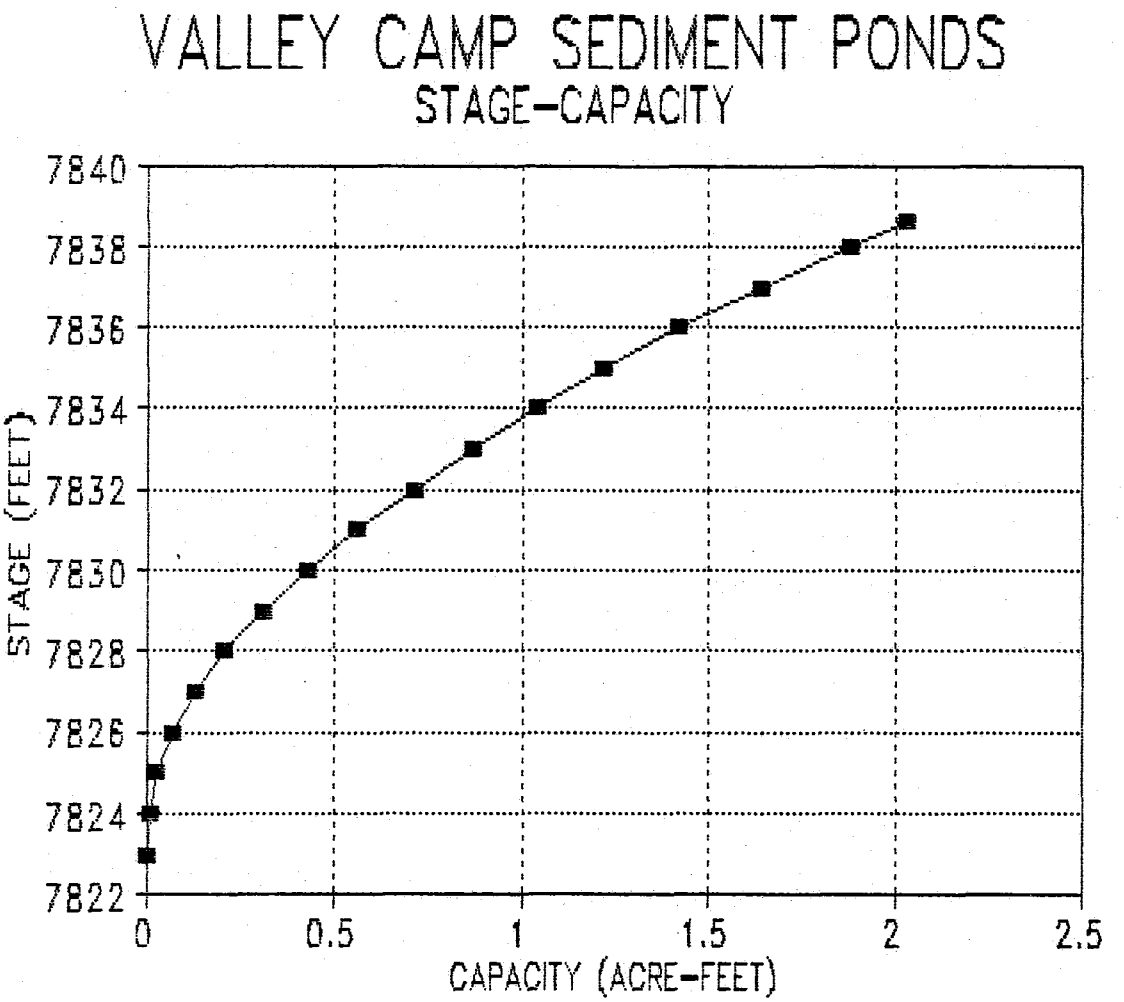


STAGE VOLUME RELATIONSHIP
POND 002A

POND NPDES 002A AFTER CLEANING, NOVEMBER 198

ELEV. FT	STAGE FT	AREA FT2	DEL VOL AF	SUM VOL AF	
7823	0	87	0	0	
7824	1	549	0.01	0.01	
7825	2	1300	0.02	0.03	
7826	3	2180	0.04	0.07	
7827	7827.63 4	3120	0.06	0.13	0.18 – 60% SEDIMENT CLEANOUT
7828	5	4000	0.08	0.21	
7829	6	4880	0.10	0.31	
7830	7	5445	0.12	0.43	Existing Decant EL - 7830.8'
7831	8	6100	0.13	0.56	
7832	9	6690	0.15	0.71	
7833	10	7020	0.16	0.87	
7834	11	7550	0.17	1.04	
7835	12	8460	0.18	1.22	Primary EL - 7835.4
7836	13	9100	0.20	1.42	Emergency EL - 7836.3'
7837	14	9920	0.22	1.64	
7838	15	10810	0.24	1.88	
7838.6	15.6	11400	0.15	2.03	

Pond 002A after Cleaning



POND 002A

STAGE VOLUME RELATIONSHIP
POND 003A

AFTER CLEANING (OCTOBER 1988)
POND NPDES 003A

ELEV. FT	STAGE FT	AREA FT~~3~~ 2	DEL VOL AF	SUM VOL AF	
7855	0	2170	0	0	
7856 7858.21	1	6430	0.10	0.10	0.47-60% SEDIMENT CLEANOUT
7859	4	8150	0.50	0.60	
7860	5	8830	0.19	0.80	
7861	6	7861	0.19	0.99	
7862	7	9310	0.20	1.18	
7863	8	10500	0.23	1.41	Primary El - 7863.2
7864	9	11650	0.25	1.67	
7865	10	12700	0.28	1.95	Emergency El - 7865.5
7866	11	14000	0.31	2.25	
7867	12	14580	0.33	2.58	
7868	13	15960	0.35	2.93	

POND NPDES 003A AFTER CLEANING

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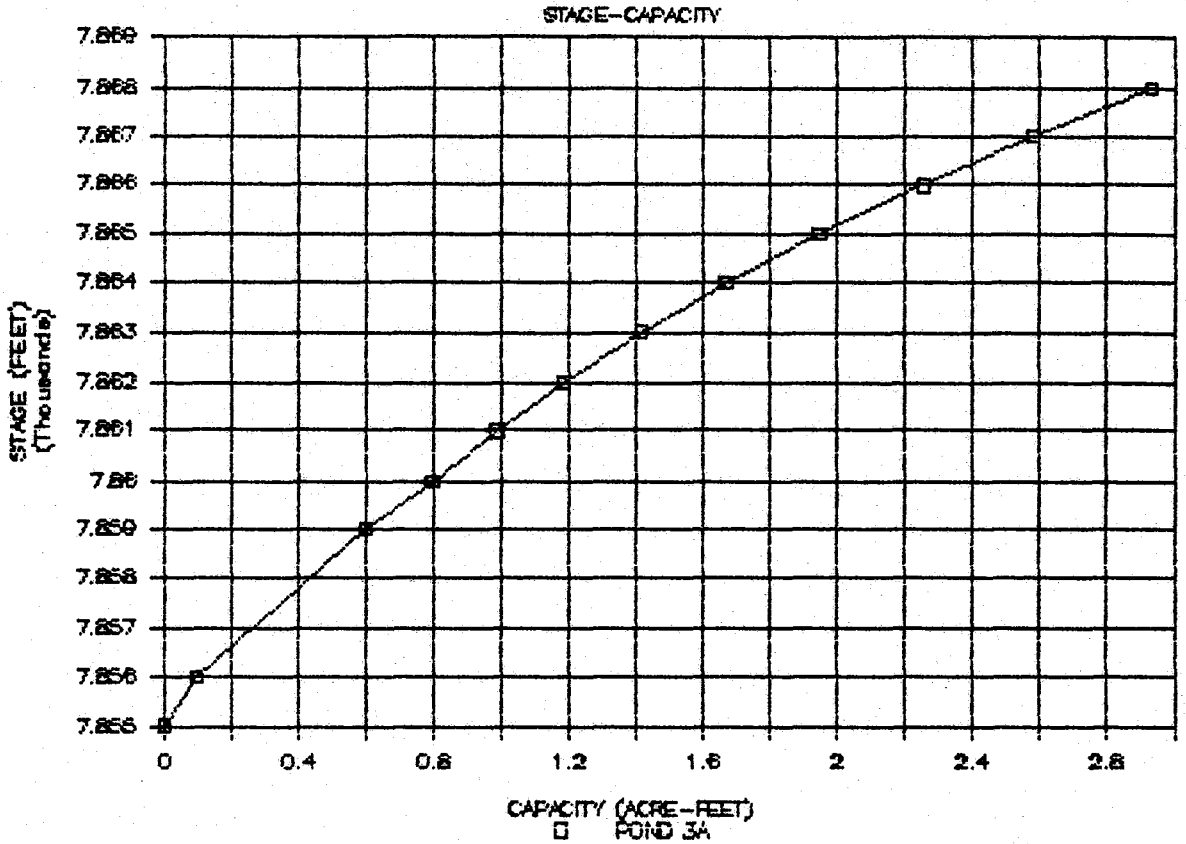
VALLEY CAMP SEDIMENT PONDS

STAGE-CAPACITY



STAGE VOLUME RELATIONSHIP
POND 00HA

8863		0	0	0	
8864	1	100	0.00	0.00	
8865	2	732	0.02	0.02	
8866	3	6169	0.09	0.11	
8867	4	13490	0.23	0.34	
8868	5	16050	0.36	0.70	— 1.09 - 60% SEDIMENT CLEANOUT
8869	6	21172	0.45	1.15	
8870	7	23860	0.52	1.67	
8871	8	25992	0.57	2.24	
8872	9	27930	0.62	2.86	
8873	10	30114	0.67	3.53	
8874	11	32320	0.72	4.24	
8875	12	34820	0.77	5.01	← Primary 8874.93
8876	13	36850	0.82	5.84	← Emergency 8875.9

— 8868.86

Storage at Primary Spillway = 4.96 ac-ft.
Required decant elev. to provide 3.15 ac-ft of runoff storage = 8870.25'

SEDIMENT POND DISCHARGE
CALCULATION CORRECTION NOTICE
(Prepared by Hansen, Allen & Luce)

June 8, 1990

Documentation of the re-survey of the four sediment ponds located at the Valley Camp of Utah facility was recently provided by Valley Camp to Hansen, Allen & Luce, Inc.. An calculation check was made to note what impact the newly obtained information has on work previously performed for the spillway rating curves. A comparison of the previous data with that of the new information is tabulated below.

Sedimentation Pond Number	Primary Spillway		Emergency Spillway	
	Prior Outfall Elev. (ft)	Updated Outfall Elev. (ft)	Prior Outfall Elev. (ft)	Updated Outfall Elev. (ft)
001A	7813.0	7809.39	7813.0	7809.41
002A	7828.0	7826.51	7828.0	7826.43
003A	7854.5	7854.60	7854.5	7854.22
004A	8859.0	8861.35	-	-

The only pipe outfall which was surveyed at a higher elevation than previously measured was for the primary spillway on Pond 003A. The small amount of decrease in head (totaling 0.1 feet) will not impact significantly the flow characteristics at the pipe inlet, or pipe flow itself for Pond 003A.

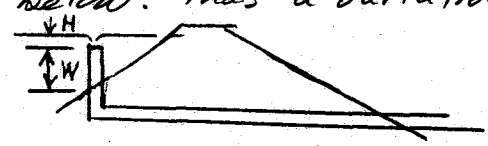
The updated information as shown in the table does however indicate larger variations in the outfall elevations for Ponds 001A, 002A, and 004A. The impact these elevation changes have on each pond is that the carrying capacity of each outfall pipe is increased, due to the increase in available head. As noted from the calculations based upon earlier information, the limiting factor for all ponds was orifice flow rather than pipe flow. Because of this, the earlier calculations remain valid as conservative estimates of each ponds ability to function during periods of high flow. No additional calculations will be required or are submitted. The calculations shown on the following pages are those prepared based upon the earlier, more conservative information.

I- Spillway Configurations:

All primary and emergency spillways for ponds 001A through 003A are 18" Diameter CMP riser standpipes, with a barrell through the embankment.

Summarized below are weir and orifice flow for an 18-inch diameter standpipe. These calculations are applicable to all primary and emergency spillways for all three ponds.

Note W in $C = 3.27 + 0.4H/W$ was assumed to be 4.7 ft for all of the ponds. The exact value for W for the ponds was not known. However, variation in W will make little difference in the prediction of weir flow over the region where weir flow governs. For example, even if W were 7 ft. at a head of 0.5 ft (where orifice flow begins to govern), the discharge over the 18" Diameter spillway would be 5.48 cfs instead of the 5.52 cfs value reported below. Thus a variation in W makes little difference in Q.



RISER TYPE SPILLWAY HYDRAULICS

WEIR FLOW: Q=CL(H)**3/2 C= 3.27+0.4H/W

ORIFICE FLOW: Q=C'A(2gH)**1/2 C'= 0.6

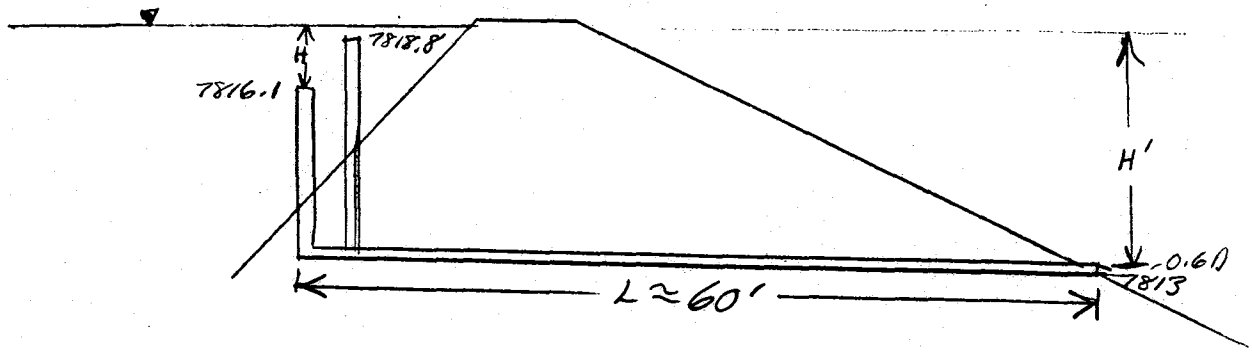
D= 1.5

A (FT2) 1.76625

STAGE H ft.	WEIR FLOW C	WEIR FLOW Q (CFS)	ORIFICE FLOW Q (CFS)	CONTOLLIN Q (CFS)
0	3.27	0	0	0
0.1	3.278	0.49	2.69	0.49
0.2	3.287	1.38	3.80	1.38
0.3	3.295	2.55	4.66	2.55
0.4	3.304	3.94	5.38	3.94
0.5	3.312	5.52	6.01	5.52
0.6	3.321	7.27	6.59	6.59
0.7	3.329	9.18	7.12	7.12
0.8	3.338	11.25	7.61	7.61
0.9	3.346	13.46	8.07	8.07
1	3.355	15.80	8.50	8.50
1.1	3.363	18.28	8.92	8.92
1.2	3.372	20.88	9.32	9.32
1.3	3.380	23.60	9.70	9.70
1.4	3.389	26.44	10.06	10.06
1.5	3.397	29.40	10.42	10.42
1.6	3.406	32.47	10.76	10.76
1.7	3.414	35.65	11.09	11.09
1.8	3.423	38.93	11.41	11.41
1.9	3.431	42.33	11.72	11.72
2	3.440	45.83	12.03	12.03
2.1	3.448	49.43	12.32	12.32
2.2	3.457	53.13	12.61	12.61
2.3	3.465	56.93	12.90	12.90
2.4	3.474	60.83	13.18	13.18
2.5	3.482	64.83	13.45	13.45
2.6	3.491	68.93	13.71	13.71
2.7	3.499	73.12	13.97	13.97
2.8	3.508	77.41	14.23	14.23
2.9	3.516	81.79	14.48	14.48
3	3.525	86.26	14.73	14.73
3.1	3.533	90.83	14.97	14.97
3.2	3.542	95.49	15.21	15.21
3.3	3.550	100.24	15.45	15.45
3.4	3.559	105.08	15.68	15.68

Orifice flow begins to govern

II - Check Pipe Flow from Pond 001A



Primary Spillway

$H' = H + [7816.1 - (7813 + 0.6(1.5))] = H + 2.2$

$L = 60'$

Emergency Spillway

$H' = H + [7818.8 - (7813 + 0.6(1.5))] = H + 4.9$

$L \approx 60'$

$$Q = \frac{a(2gH')^{1/2}}{(1 + K_e + K_b + K_c L)^{1/2}}$$

$K_e = 1$

$K_b = 0.5$

$K_c = \frac{5087(0.027)^2}{(18)^{4/3}} = 0.0786$

$a = 1.770\ ft^2$

$$Q = \frac{1.77(2 * 32.2\ H')^{1/2}}{(1 + 1 + 0.5 + 0.0786(60))^{1/2}} = \frac{14.20\ H'^{1/2}}{2.69} = 5.28\ H'^{1/2}$$

Primary Spillway

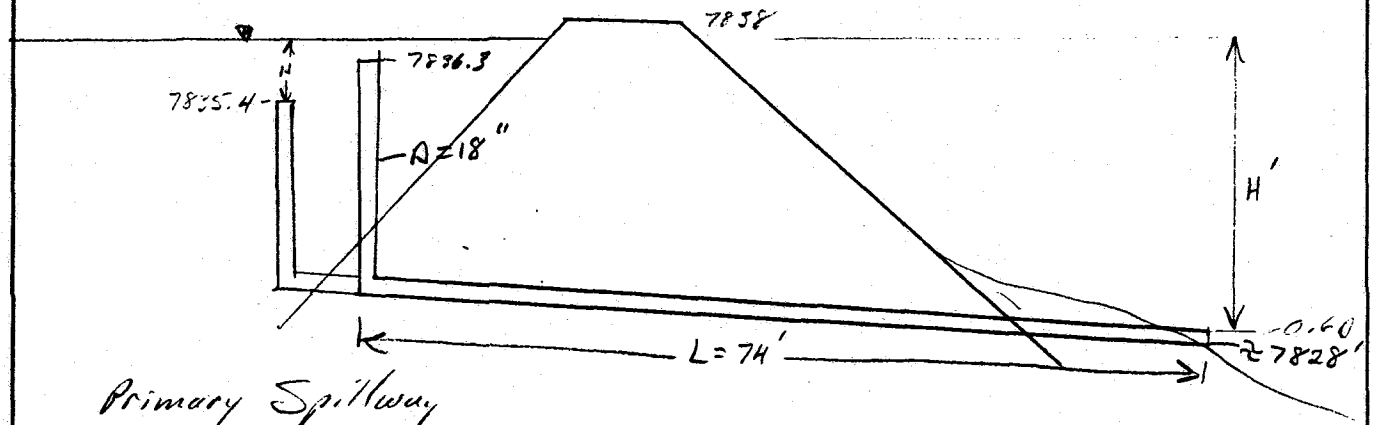
H ft	H' ft	Q cfs
0.2	2.4	8.18
0.4	2.6	8.51
0.6	2.8	8.83
0.8	3.0	9.14
1.0	3.2	9.45
1.2	3.4	9.73
1.4	3.6	10.02 ← Pipe flow becomes limiting at this point.
1.6	3.8	10.29
1.8	4.0	10.56
2.0	4.2	10.82
2.1	4.3	10.95
2.3	4.5	11.20
2.5	4.7	11.45
2.7	4.9	11.69
2.9	5.1	11.92
3.1	5.3	12.16
3.3	5.5	12.38
3.5	5.7	12.60
3.7	5.9	12.83

Emergency Spillway

H ft	H' ft	Q cfs
0.2	5.1	11.92
0.4	5.3	12.16
0.6	5.5	12.38
0.8	5.7	12.60
1.0	5.9	12.83
1.2	6.1	13.04
1.4	6.3	13.25
1.6	6.5	13.46
1.8	6.7	13.67
2.0	6.9	13.87 ↑

Orifice flow still limiting @ H = 2'.

III - Check Pipe Flow from 002A



Primary Spillway

$H' = H + [7835.4 - (7828 + 0.6(1.5))] = H + 6.5$

$L = 74'$

Emergency Spillway

$L = 74'$

$H' = H + [7836.3 - (7828 + 0.6(1.5))] = H + 7.4$

$$Q = \frac{a(2gH')^{1/2}}{(1 + K_e + K_b + K_c L)^{1/2}}$$

$K_e = 1.0$ $\quad n = 0.027$

$K_b = 0.5$

$K_c = \frac{5087 n^2}{d_i^{4/3}} = \frac{5087(0.027)^2}{(18)^{4/3}} = 0.0786$

$a = 1.770 \text{ ft}^2$

Principal Spillway & Emergency - (Since both are the same length & diameter)

$$Q = \frac{1.77[2(32.2)H']^{1/2}}{(1 + 1 + 0.5 + 0.0786(74))^{1/2}} = \frac{14.20 H'^{1/2}}{2.88H} = 4.92 H'^{1/2}$$

Primary Spillway - Pipe Flow

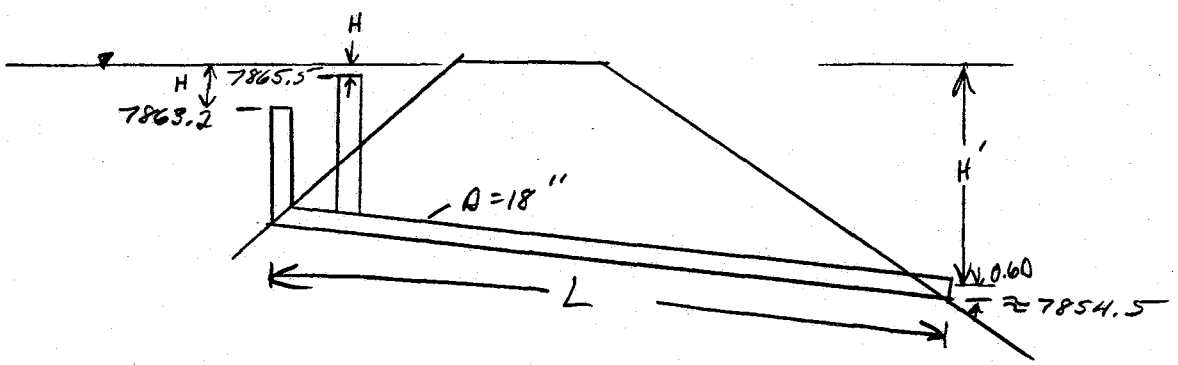
H (ft)	H' (ft)	Q (cfs)
0.2	6.7	12.7
0.4	6.9	12.9
0.6	7.1	13.1
0.8	7.3	13.3
1.0	7.5	13.47
1.2	7.7	13.65
1.4	7.9	13.83
1.6	8.1	14.00
1.8	8.3	14.17
2.0	8.5	14.34

Emergency Spillway - Pipe Flow

H (ft)	H' (ft)	Q cfs
0.2	7.6	13.56
1.0	8.4	14.26
2.0	9.4	15.08

← At this head orifice flow is still controlling, thus pipe flow is not controlling in Pond 002A

IV - Check Pipe Flow from Pond 003A



Principal Spillway

$$H' = H + [7863.2 - (7854.5 + 0.6(1.5))] = H + 7.8$$

$L \approx 74'$

Emergency

$$H' = H + [7865.5 - (7854.5 + 0.6(1.5))] = H + 10.1$$

$L \approx 74'$

$$Q = \frac{a(2gH')^{1/2}}{(1 + K_e + K_b + K_c L)^{1/2}}$$

$K_e = 1.0$ $\quad n = 0.027$

$K_b = 0.5$

$$K_c = \frac{5087 n^2}{d_c^{4/3}} = \frac{5087(0.027)^2}{(18)^{4/3}} = 0.0786$$

$$a = \text{area} = \frac{\pi D^2}{4} = \frac{\pi (1.5)^2}{4} = 1.77 ft^2$$

Principal and Emergency - (Since both have the same length; diameter).

$$Q = \frac{1.77[2(32.2)H']^{1/2}}{(1 + 1.0 + 0.5 + 0.0786(74))^{1/2}} = \frac{14.20 H'^{1/2}}{2.884} = 4.92 H'^{1/2}$$

Primary Spillway - Pipe Flow

H (ft)	H' (ft)	Q (cfs)
0.2	8.0	13.91
0.4	8.2	
0.6	8.4	
0.8	8.6	
1.0	8.8	14.60
1.2	9.0	
1.4	9.2	
1.6	9.4	
1.8	9.6	
2.0	9.8	15.40 cfs

Orifice Flow is still limiting at this head. Thus pipe flow is not controlling.

Emergency Spillway - Pipe flow

H (ft)	H' (ft)	Q (cfs)
0.2	10.3	15.8
1.0	11.1	16.4
2.0	12.1	17.1

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Spillway Rating Curves - Ponds 001A to 003A
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CHECKED
DATE 11/25/87

V - Discharge Rating Table - Pond 001A

Elev	Primary Spillway H ft	Primary Spillway Q cfs	Emergency Spillway H ft	Emergency Spillway Q cfs	Combined Q cfs
7816.1	0	0	0	0	0
7816.2	0.1	0.49			0.49
7816.4	0.3	2.55			2.55
7816.6	0.5	5.52			5.52
7816.8	0.7	7.12			7.12
7817.0	0.9	8.07			8.07
7817.2	1.1	8.92			8.92
7817.4	1.3	9.70			9.70
7817.6	1.5	10.16			10.16
7817.8	1.7	10.43			10.43
7818.0	1.9	10.69			10.69
7818.2	2.1	10.95			10.95
7818.4	2.3	11.20			11.20
7818.6	2.5	11.45			11.45
7818.8	2.7	11.69	0	0	11.69
7819.0	2.9	11.92	0.2	1.38	13.30
7819.2	3.1	12.16	0.4	3.94	16.10
7819.4	3.3	12.38	0.6	6.59	18.97
7819.6	3.5	12.60	0.8	7.61	20.21
7819.8	3.7	12.83	1.0	8.50	21.33

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Sediment Pond Analysis
FEATURE Spillway Rating Curves - Ponds 001A to 003A
PROJECT NO.

SHEET 6 OF 7
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CHECKED
DATE 1/25/89

VI - Discharge Rating Table - Pond 002A

Elev ft	Primary Spillway H ft	Primary Spillway Q cfs	Emergency Spillway H ft	Emergency Spillway Q cfs	Combined Q cfs
7835.4	0	0	0	0	0
7835.6	0.2	1.38			1.38
7835.8	0.4	3.94			3.94
7836.0	0.6	6.59			6.59
7836.2	0.8	7.61			7.61
7836.3	0.9	8.07	0	0	8.07
7836.4	1.0	8.50	0.1	0.49	8.99
7836.6	1.2	9.32	0.3	2.55	11.87
7836.8	1.4	10.06	0.5	5.52	15.58
7837.0	1.6	10.76	0.7	7.12	17.88
7837.2	1.8	11.41	0.9	8.07	19.48

HANSEN ALLEN & LUCE INC

CLIENT: Valley Camp
PROJECT: Sediment Pond Analysis
FEATURE: Spillway Rating Curves - Ponds 001A to 003A
PROJECT NO.:

SHEET 7 OF 7
COMPUTED: DMA
CHECKED:
DATE: 1/25/89

POND 003A

Elev ft.	Primary Spillway H ft	Primary Spillway Q cfs	Emergency Spillway H ft	Emergency Spillway Q cfs	Combined Q cfs
7863.2	0	0	0	0	0
7863.4	0.2	1.38			1.38
7863.6	0.4	3.94			3.94
7863.8	0.6	6.59			6.59
7864.0	0.8	7.61			7.61
7864.2	1.0	8.50			8.50
7864.4	1.2	9.32			9.32
7864.6	1.4	10.06			10.06
7864.8	1.6	10.76			10.76
7864.87					11.0 des. flow
7865.0	1.8	11.41			11.41
7865.2	2.0	12.03			12.03
7865.4	2.2	12.61			12.61
7865.5	2.3	12.90	0	0	12.9
7865.6	2.4	13.18	0.1	0.49	13.67
7865.8	2.6	13.71	0.3	2.55	16.26
7866.0	2.8	14.23	0.5	5.52	19.76
7866.2	3.0	14.73	0.7	7.12	21.85
7866.4	3.2	15.21	0.9	8.07	23.28

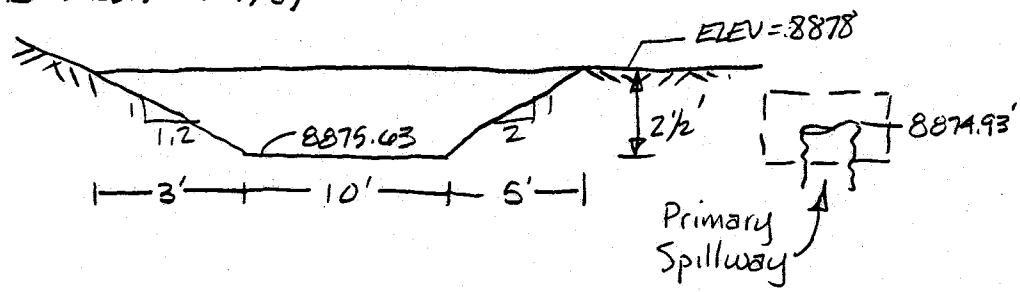
Min Embankment crest set 2.0' above Emergency Spillway Elevation. (7865.5+2.0=7867.5')

POND 004A Emergency Spillway

These calculations are a reworking of calculations completed earlier related to spillway requirements for Pond 004A. Calculations dated 23 MAY 89 are hereby replaced.

Emergency Spillway was reconstructed in May, 1989 subsequent to damage caused by animal boring.

Dimensions taken 5/17/89



Δh required between spillways = 1.0'
∴ must raise spillway (emergency) to 8875.9'

Embankment Elevation: EXISTING = 8877.5'

Crest Length = 20'

Previous Calculations use 25Yr, 24Hr Runoff Event = 52cfs

New Calculations allow for 25Yr, 6Hr Runoff Event = 13.2 cfs

by addition of ditches and culverts, Q = 14.5cfs + a small tributary area

C-34-24	Q = 2.52
C-33-24	Q = 11.96
	14.48

Estimated flow equal to 13.2 cfs is used because the estimate by ditch and culvert addition does not allow for routing.

Leaving pond in its "as is" state, no modifications are made to sediment volumes, etc as part of this hydraulic check.

Upstream Slope > 2:1

* Modified "C" values based upon decrease noted for Rect. weir w/20' Crest

PROJECT : VALLEY CAMP SEDIMENT POND 004A: 25-YR. 6-HR PEAK INFLOW

AREA= 37.1 ACRES
AVERAGE BASIN SLOPE= 40.0 PERCENT
CURVE NUMBER= 83.0
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2040. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0896 HOURS QPCFS= 312.99 CFS QPIN= 8.3664 INCHES
C3= 41.2379 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.04	.4068	.0000	.0000	.0	.00
2.06	.4273	.0002	.0002	15.7	.00
2.08	.4479	.0007	.0005	97.2	.02
2.10	.4685	.0016	.0009	207.8	.10
2.12	.4890	.0030	.0013	287.3	.27
2.13	.5096	.0046	.0017	313.0	.56
2.15	.5301	.0067	.0020	293.2	.96
2.17	.5507	.0091	.0024	247.5	1.46
2.19	.5713	.0118	.0027	193.6	2.04
2.21	.5918	.0149	.0031	142.8	2.68
2.22	.6124	.0183	.0034	100.7	3.34
2.24	.6330	.0220	.0037	68.4	4.03
2.26	.6535	.0260	.0040	45.0	4.72
2.28	.6741	.0302	.0043	28.9	5.41
2.29	.6947	.0348	.0046	18.1	6.09
2.31	.7152	.0397	.0049	11.2	6.76
2.33	.7358	.0448	.0051	6.8	7.41
2.35	.7563	.0502	.0054	4.0	8.04
2.37	.7769	.0558	.0057	2.4	8.67
2.38	.7975	.0617	.0059	1.4	9.27
2.40	.8180	.0679	.0061	.8	9.86
2.42	.8386	.0743	.0064	.5	10.44
2.44	.8592	.0809	.0066	.3	11.00
2.46	.8797	.0878	.0068	.1	11.54
2.47	.9003	.0948	.0071	.0	12.08
2.49	.9209	.1021	.0073	.0	12.59
2.51	.9331	.1065	.0044	.0	13.05
2.53	.9386	.1086	.0020	.0	13.21
2.55	.9442	.1106	.0021	.0	12.79
2.56	.9498	.1127	.0021	.0	11.82
2.58	.9553	.1148	.0021	.0	10.51
2.60	.9609	.1169	.0021	.0	9.14
2.62	.9664	.1190	.0021	.0	7.88
2.64	.9720	.1211	.0021	.0	6.86

HYDROGRAPH PEAK= 13.21 cfs
TIME TO PEAK= 2.53 Hours
RUNOFF VOLUME= 1.26 Acre-Feet

ELEV	HEAD	C	C*
8875.9	0	–	–
8876.1	0.2	2.82	2.23
8876.3	0.4	3.04	2.40

Broad Crested Weir eqn: $Q = CLH^{3/2}$ $L = 10'$

from Stage-Discharge hydraulic analysis - (sheet 4)

	Primary	Emergency	Σ
Head	1.29'	0.29	
Q	9.65	3.55	13.2

Spillway is adequate "as is".

REQUIREMENTS:

Primary Spillway Elev: 8874.93' ft

Emergency Spillway Elev: 8875.9 ft

Emergency Flow depth: 0.3 ft

Freeboard : 1.0 ft

Top of Embankment, min Elev: 8877.2 ft

RIPRAP CHECK

Primary Spillway - Q = 9.65 cfs Dia = 18 inch

Check velocity - $n = .024$; $S_f = 0.19$

$\frac{Qn}{1.49 S^{1/2}} = 0.349$, $y = 0.65'$, $V = 13.2$ fps

According to "Erosion + Sediment Control - Surface Mining in the Eastern U.S." - EPA, 1976

Fig 1-15

$D_{50} \sim 0.6$ ft, use 0.75' d_{50} (based upon Velocity)

$D_{max} = 1.5 d_{50} = 1.13$ use 1.25' d_{max}

Continue at least 10' downstream.

Client: Valley Camp Coal Company - Stage Discharge Hydraulic Analysis - Sediment Pond 004A
Project: 007.11.200
Date : August 17, 1990

Weir Flow Equation: Q=CL(H)**3/2 Where: C=3.27+0.4H/W
Orifice Flow Equation: Q=C'A(2gH)**1/2 C'= 0.6
Channel Weir Flow Equation: Q=C"L(H)**3/2

Pipe Dia : 1.5 feet
Weir Elev: 8874.9 feet (msl)
Chan Elev: 8875.9 feet (msl)
Chan Wdth: 10 feet
Weir Area: 1.77 feet squared

Weir Stage	Weir Head (ft)	C	Weir Flow Q(cfs)	Orifice Flow (cfs)	Governing Flow (cfs)	Channel Head (ft)	Channel Flow C"	Channel Flow Q(cfs)	Combined Flow (cfs)
8874.9	0.0	3.27	0.00	0.00	0.00			0.00	0.00
8875.0	0.1	3.28	0.49	2.69	0.49			0.00	0.49
8875.1	0.2	3.29	1.38	3.81	1.38			0.00	1.38
8875.2	0.3	3.30	2.55	4.66	2.55			0.00	2.55
8875.3	0.4	3.30	3.94	5.38	3.94			0.00	3.94
8875.4	0.5	3.31	5.52	6.02	5.52			0.00	5.52
8875.5	0.6	3.32	7.27	6.59	6.59			0.00	6.59
8875.6	0.7	3.33	9.18	7.12	7.12			0.00	7.12
8875.7	0.8	3.34	11.25	7.61	7.61			0.00	7.61
8875.8	0.9	3.35	13.46	8.07	8.07			0.00	8.07
8875.9	1.0	3.35	15.80	8.51	8.51	0.0	2.05	0.00	8.51
8876.0	1.1	3.36	18.28	8.92	8.92	0.1	2.14	0.68	9.60
8876.1	1.2	3.37	20.88	9.32	9.32	0.2	2.23	1.99	11.32
8876.2	1.3	3.38	23.60	9.70	9.70	0.3	2.32	3.81	13.51 ← 13.2
8876.3	1.4	3.39	26.44	10.07	10.07	0.4	2.40	6.07	16.14

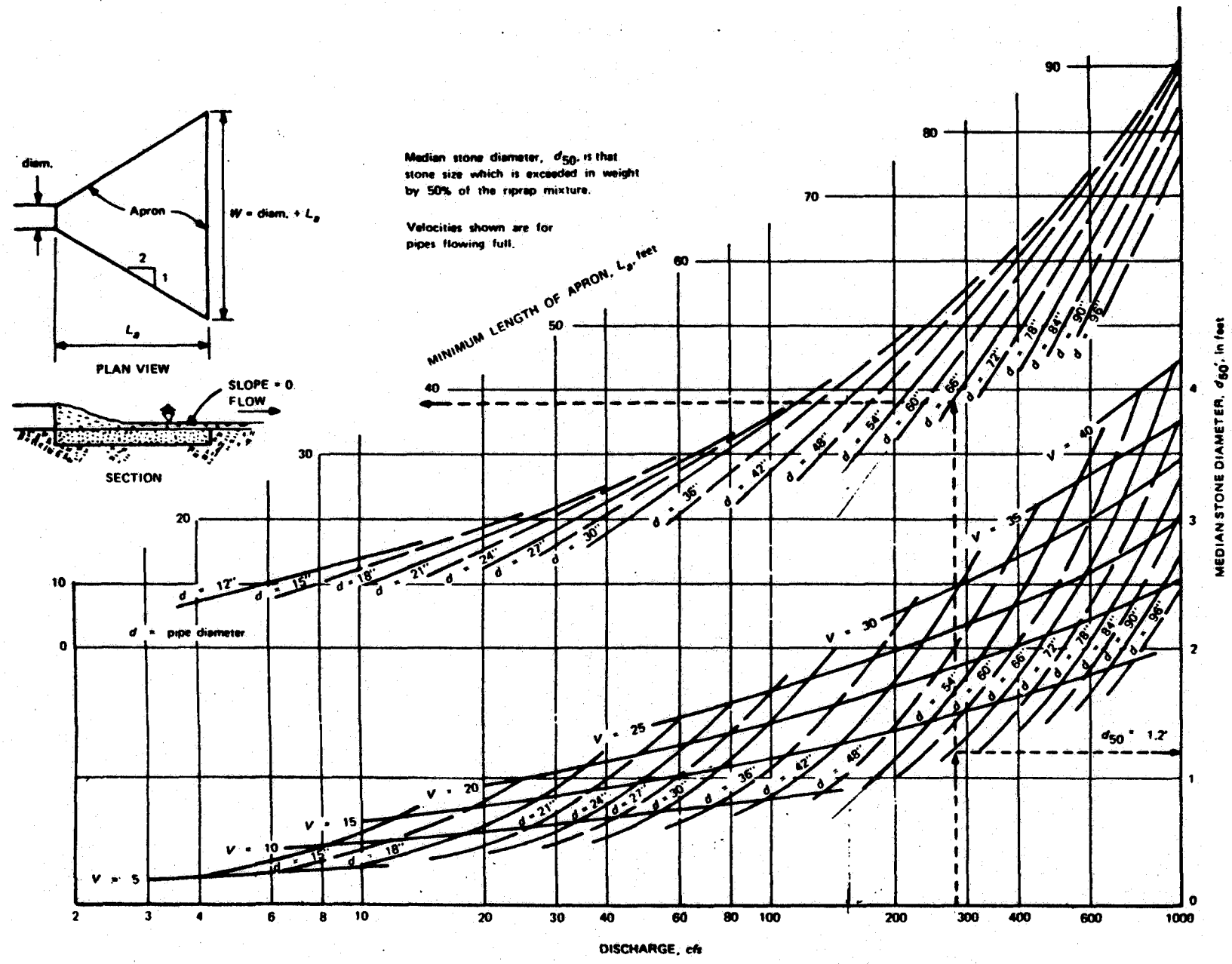


Figure I-15. Design of outlet protection — minimum tailwater condition ($T_W < 0.5$ diam.)[1]

Emergency Spillway

Use "OSM" Steep Channel design Method.

Slope down face of pond is approx. 35 to 40% (at max sections) using 40% grades, and a 6' base width (conservatively reducing the width from 10' to 6'), and a Q = 3.55 cfs.

$d_{50} < 0.5'$ $\quad d_{max} = 1.25(d_{50}) = 0.63$ use 0.75' d_{max}

$d_{10\text{-}20} = \frac{d_{50}}{2\text{-}3} = 0.17$ to $0.25'$ Use .25' $d_{10\text{-}20}$

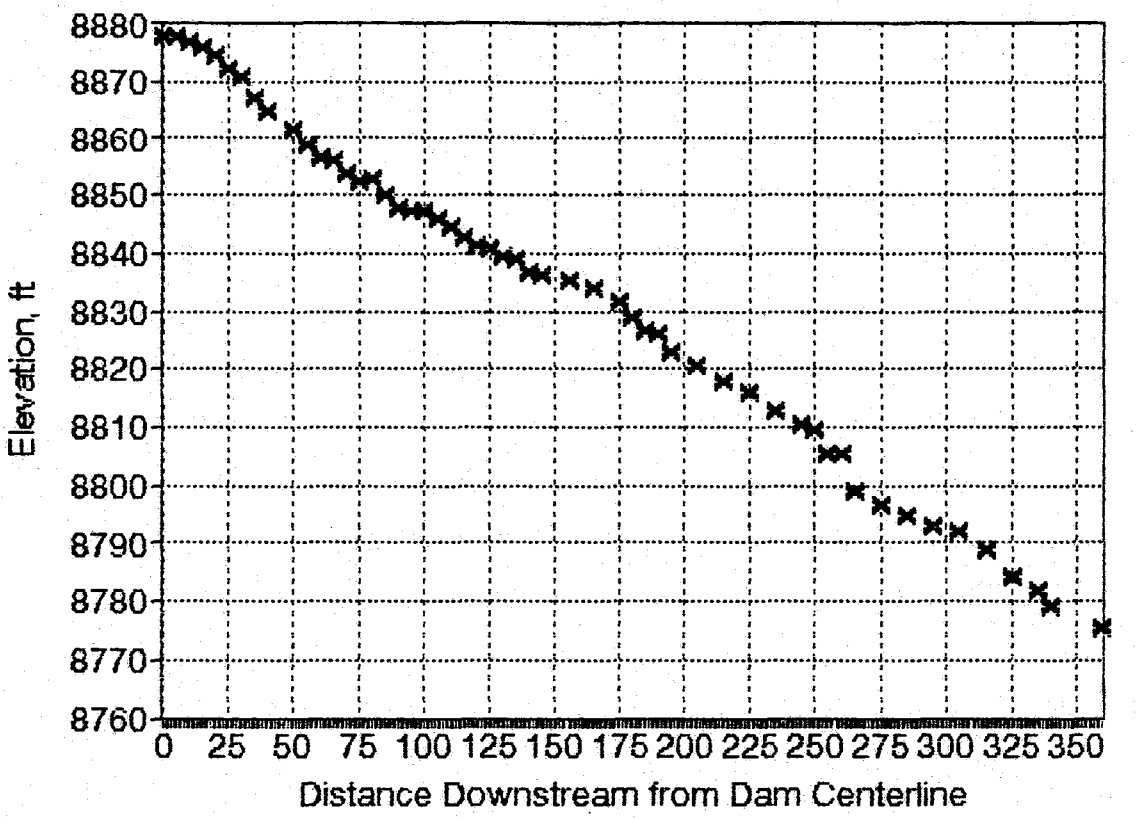
Use $d_{50} = 0.5'$

Valley Camp Coal - Pond 4 Survey Notes by Steve Tanner
Data Received 7/25/90
Top of Embankment: 8878

EMERGENCY SPILLWAY PROFILE

Distance	Relative Elev.	msl Elev.	Distance	Relative Elev.	msl Elev.
0	200.0	8878.0	205	142.6	8820.6
5	199.7	8877.7	215	139.9	8817.9
10	198.9	8876.9	225	137.9	8815.9
15	197.6	8875.6	235	134.8	8812.8
20	196.4	8874.4	245	132.2	8810.2
25	194.4	8872.4	250	131.5	8809.5
30	192.6	8870.6	255	127.3	8805.3
35	189.3	8867.3	260	127.5	8805.5
40	186.6	8864.6	265	120.8	8798.8
50	183.7	8861.7	275	118.9	8796.9
55	180.8	8858.8	285	116.9	8794.9
60	178.7	8856.7	295	114.8	8792.8
65	178.2	8856.2	305	113.9	8791.9
70	175.9	8853.9	315	110.9	8788.9
75	174.5	8852.5	325	106.1	8784.1
80	174.7	8852.7	335	103.8	8781.8
85	172.0	8850.0	340	101.0	8779.0
90	169.8	8847.8	360	97.5	8775.5
95	169.4	8847.4			
100	169.5	8847.5			
105	167.8	8845.8			
110	166.7	8844.7			
115	164.8	8842.8			
120	163.5	8841.5			
125	162.9	8840.9			
130	161.7	8839.7			
135	160.7	8838.7			
140	158.7	8836.7			
145	158.4	8836.4			
155	157.5	8835.5			
165	156.0	8834.0			
175	153.4	8831.4			
180	150.9	8828.9			
185	148.6	8826.6			
190	148.0	8826.0			
195	144.9	8822.9			

Pond 004A - Spillway Cross Section



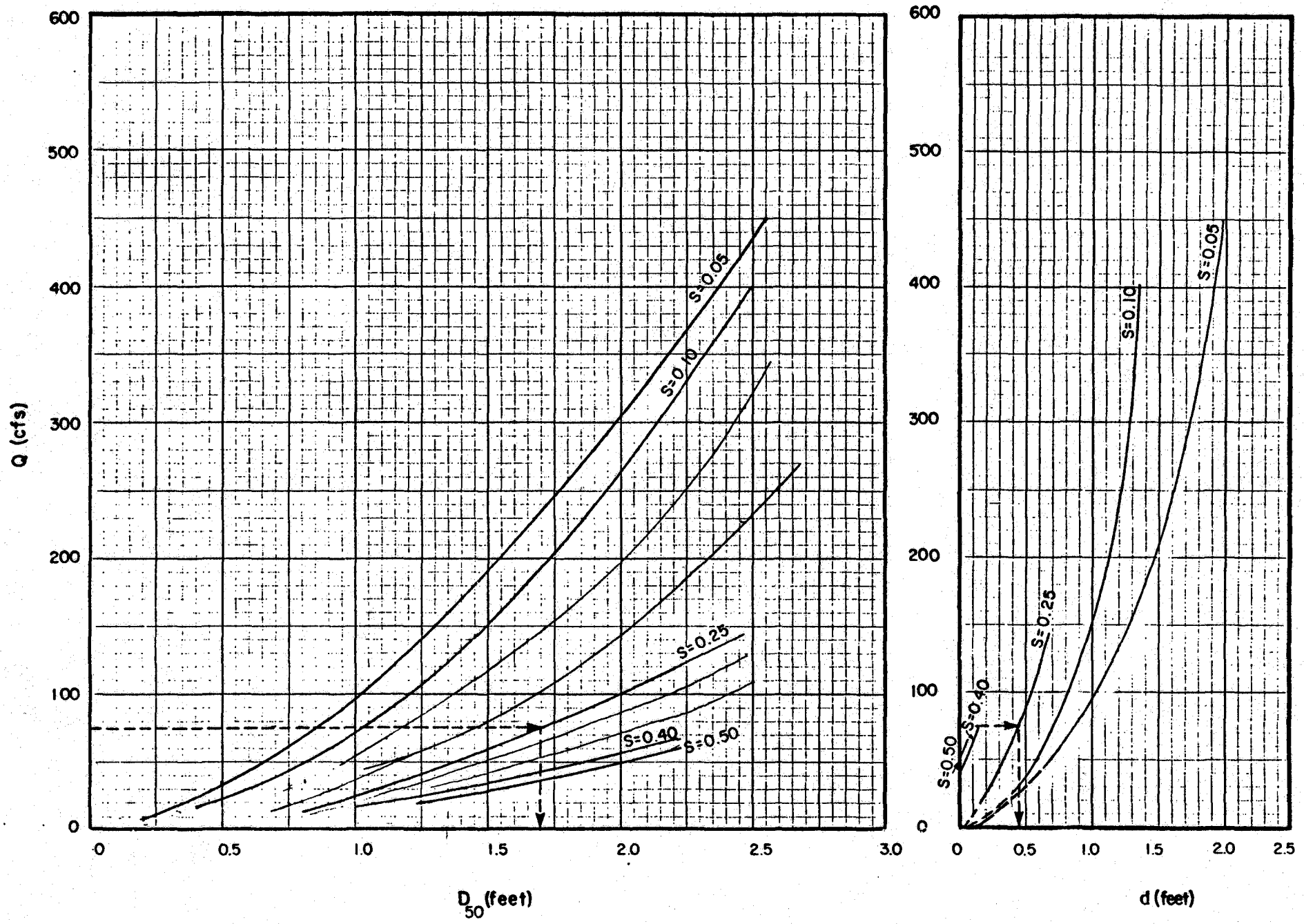
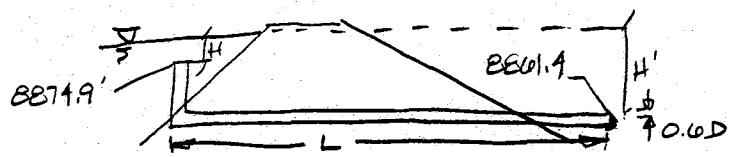


Figure 5.4. Steep slope riprap design, trapezoidal channels, 2:1 sideslopes, 6 ft base width.

Check of Pond 00AH (Pipe Flow Conditions)

PRIMARY SPILLWAY $L \approx 80'$

$$H' = H + [8874.9 - (8861.4 + 0.6(1.5))] = H + 12.65$$



$$Q = \frac{a(2gH')^{1/2}}{(1 + k_e + k_b + k_c l)^{1/2}} =$$

$K_e = 1$

$K_b = 0.5$

$K_c = \frac{5087(n)^2}{D^{4/3}} = \frac{5087(.027)^{1/2}}{(18)^{4/3}} = 0.0786$

$a = 1.77 ft^2$

$$Q = \frac{1.77(2(32.2)H')^{1/2}}{(1 + 1 + 0.5 + (.0786)80)^{1/2}} = \frac{14.2H'^{1/2}}{2.96}$$

UNDER MAX HEAD CONDITIONS

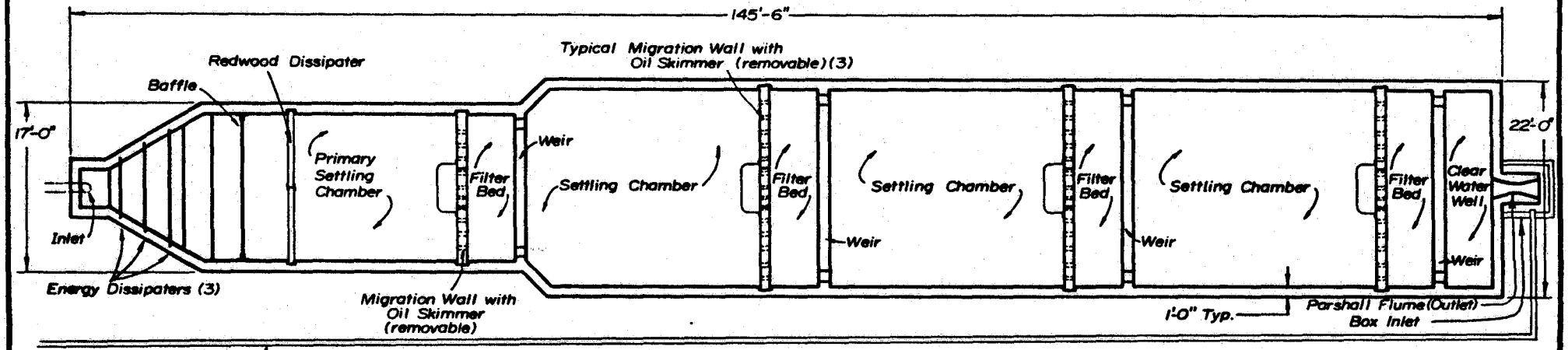
$H_{max} = 2.1'$ $\quad H' = 2.1' + 12.65 = 14.75'$

$$Q = \frac{14.2(14.75)^{1/2}}{2.96} = \underline{\underline{18.42 cfs}}$$

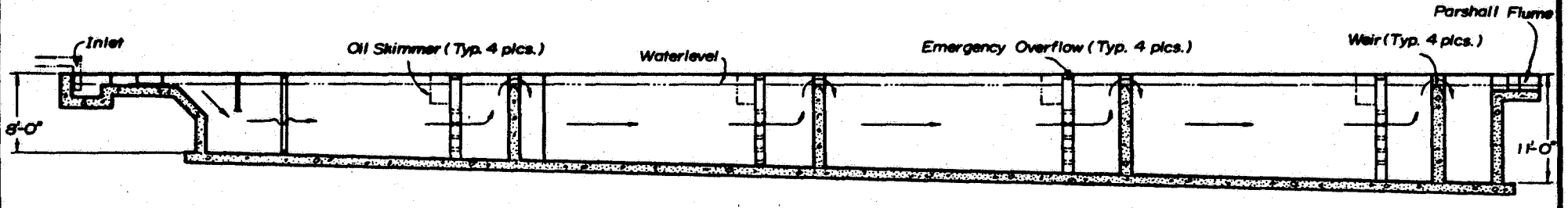
18.4 cfs > 12.22 cfs under Inlet Conditions,
∴ NOT GOVERNING

STAGE DISCHARGE RELATIONSHIP SHOWN ON SHEET 4

MONSEN ENG. • MC763



PLAN VIEW



SECTION VIEW & FLOW DIAGRAM

REGISTERED PROFESSIONAL ENGINEER
No. 5323
EDWIN BROOKSHIRE FOUST
STATE OF UTAH

DRAWN BY: Ed Sanderson	VALLEY CAMP OF UTAH, INC. SCOFIELD ROUTE HELPER, UTAH 84526
DATE: Sept. 12, 1983	
CHECKED BY:	
DATE:	
APPROVAL:	TITLE: CONCRETE FILTER POND
APPROVAL:	
APPROVAL:	SCALE: NONE — DRAWING NO. B4-0010

Fig. 3-6C

APPENDIX R614-301-742.221b

Sediment Pond Stability Reports

HANSEN ALLEN & LUCE inc

Chen-Northern, Inc.

350 West 2700 South
Salt Lake City, Utah 84115
801/487-3661

Billings
Boise
Casper
Colorado Springs
Denver
Elko
Evanston
Gilette
Glenwood Springs
Great Falls
Helena
Phoenix
Pocatello
Rock Springs
Salt Lake City
San Antonio
Tri Cities
Yakima

January 31, 1989

Subject: Geotechnical Consultation
Valley Camp Sedimentation
Ponds 1, 2 and 3,
Scofield, Utah

Job No. 515889

Hansen, Allen and Luce
6771 South 900 East
Midvale, Utah 84047

Attn: Mr. Marv Allen

Gentlemen:

As requested, Chen-Northern, Inc. performed stability analyses for the Valley Camp Sedimentation Ponds 1 through 3. The project site is located along the Scofield route near Helper, Utah.

Soil strength values for Pond 1 were taken from a report by Garco Testing Laboratories dated December 2, 1988. Soil strength values for Ponds 2 and 3 were taken from a report by Rollins, Brown and Gunnell, Inc. dated November 16, 1988.

The strength parameters, as well as the moist and saturated unit weights determined from the above-reference reports, are presented in the following table.

Pond No.	C psf	O Degrees	Moist Unit Weight, pcf	Saturated Unit weight, pcf
1	400	22	112	124
2	150	31	107	122
3	150	31	107	122

The geometry of the embankments and side slopes of the sedimentation ponds were taken from "Val-Cam Loadout Sediment Pond Analysis", Sheets 1, 2 and 3 by Hansen, Allen and Luce, Inc. dated September, 1988. In addition, we were provided with elevations of the bottom of the ponds as well as the top of the primary and emergency spillways for the ponds, which are presented in the following table.

Pond No.	Bottom of Pond feet	Top of Primary Spillway, feet	Top of Emergency spillway, feet
1	7803.0	7816.1	7818.8
2	7823.0	7835.4	7836.3
3	7855.0	7863.2	7865.5

In our analyses, we assumed that the soil profiles were homogenous.

The safety factors have been calculated for rapid-drawdown conditions. The soil beneath the elevation of the primary spillway was assumed to be saturated. The calculated safety factors and approximate slope angles for the sedimentation ponds are presented below:

Pond No.	Slope	Safety Factor
1	2.2H:1V (upstream)	2.1
2	1.9H:1V (upstream)	1.3
3	1.5H:1V (upstream)	1.3
1	2.8H:1V (downstream)	4.1
2	1.7H:1V (downstream)	1.5
3	1.8H:1V (downstream)	1.5

It was determined that flattening the upstream slopes of Sedimentation Ponds 2 and 3 to 2.5V:1V will result in safety factors of approximately 1.5 under rapid-drawdown conditions.

Analyses were performed to assess the stability of Sedimentation Pond 3 under static and dynamic steady-state seepage conditions. Analyses were performed for Pond 3 rather than Ponds 1 and 2 due to the steepness of Pond 3 slopes despite the fact that the embankments for Ponds 1 and 2 are 1 and 4 feet higher, respectively. For the dynamic analyses, a coefficient of horizontal acceleration of 0.1 was used based on the Uniform Building Code seismic zone map for the State of Utah. The safety factors calculated for Pond 3 for static and dynamic conditions are 2.0 and 1.6, respectively. These safety factors correspond to the upstream slopes. The safety factors for the downstream slopes are slightly higher because the downstream slopes are not as steep as the upstream slopes. Based on the rapid-drawdown analyses, the safety factors for Ponds 1 and 2 under steady-state static and dynamic conditions should be approximately equal to or greater than those calculated for Pond 3.

The safety factors calculated from the stability analyses are based on the strength values and topographic information mentioned previously. The geometry within the interior of the ponds was not

provided and was assumed based on the topography and elevation data provided.

The safety factors obtained appear to be suitable for the purpose of the ponds. Static and dynamic safety factors under steady state conditions should be greater than 1.5 and 1.1, respectively. Safety factors under rapid drawdown should be greater than 1.1. Each of the slopes evaluated have safety factors in excess of the minimum values. To maintain the minimum safety factors presented above, the upstream and downstream slopes should be no steeper than 1.5H:1V.

If you have questions, or if we may be of further service, please call.

Sincerely,

CHEN-NORTHERN, INC.

Dean B. Griswold

DBG/cs
Rev. by JEN, P.E.

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT Valley-Camp Sed. Ponds JOB NO. 515889

PURPOSE Stab. Analyses SHEET 1 OF 8

COMPUTED BY DBG CHECKED BY DATE 1/31/89

Steepest Permissible Slopes

Condition	Req'd Safety Factor
Steady State, Static	1.5
Steady State Dynamic	1.1
Rapid Drawdown	1.1

Parameters for Most Critical Slope

Highest Slope: 18 ft

Strength Parameters: $\phi = 31°$, $c = 150$ psf, $\gamma_{SAT} = 122$ pcf, $\gamma_{in} = 107$ pcf

Steepest Slope: 1.5H:1V

Highest Water level: 13 ft.

Narrowest Crest: 8 ft.

Condition	Safety Factor	Slope
Steady State, Static	1.51	0.8H:1V
Steady State Dynamic	1.21	0.72H:1V
Rapid Drawdown	1.15	1.5H:1V

Most Critical Case is Rapid Drawdown w/ FS = 1.15 and Slope = 1.5H:1V

NET-145

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

$H = 18$ ft
$\gamma_{SAT} = 122$ pcf
$\gamma_{dry} = 107$ pcf
$\phi = 31$
$c = 150$ psf

PROBLEM DESCRIPTION VAL-CAM SED. PONDS; GENERIC POND W/ STEE
PEST SLP & HIGHEST EMB & WEAK SOIL

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

RAPID DRAWDOWN
FS = 1.15
Slope = 1.5H : 1V

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	40.00	20.00	40.00	1
2	20.00	40.00	40.00	53.00	1
3	40.00	53.00	47.00	58.00	1
4	47.00	58.00	55.00	58.00	1
5	55.00	58.00	73.00	46.00	1
6	73.00	46.00	82.00	40.00	1

ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	40.00
2	20.00	40.00
3	40.00	53.00
4	50.00	53.00
5	73.00	46.00
6	82.00	40.00

A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 50.00 FT.
AND X = 80.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	16.67	40.00
2	24.10	37.04
3	32.10	36.77
4	39.71	39.22
5	46.05	44.10
6	50.36	50.84
7	52.00	58.00

*** 1.153 ***

4/8

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VAL-CAM SED. PONDS; GENERIC POND W/ STEE
PEST SLP & HIGHEST EMB & WEAK SOIL

$H_t = 18$ ft
$\gamma_{SAT} = 122$ pcf
$\gamma_m = 107$ pcf
$\phi = 31°$
$c = 150$ psf
Steady State, Static
FS = 1.5
Slope = 0.8H:1V

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	40.00	32.00	40.00	1
2	32.00	40.00	43.00	53.00	1
3	43.00	53.00	47.00	58.00	1
4	47.00	58.00	55.00	58.00	1
5	55.00	58.00	73.00	46.00	1
6	73.00	46.00	82.00	40.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	40.00
2	82.00	40.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM

TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 50.00 FT.
AND X = 80.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	20.00	40.00
2	27.84	38.41
3	35.80	39.20
4	43.18	42.30
5	49.31	47.43
6	53.67	54.14
7	54.76	58.00

*** 1.514 ***

6/8

1

GEOSLOPE
Version 2.0

Supplied by GEOCOMP Corp.
342 Sudbury Rd., Concord, MA. 01742
(617)369-8304

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$H = 18$ ft
$\gamma_{SAT} = 122$ pcf
$\gamma_m = 107$ pcf
$\phi = 31°$
$c = 150$ psf

Steady State, Dynamic
$a = 0.1g$
$F = 1.21$
Slope $= 0.72H:1V$

1

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VAL-CAM SED. PONDS; GENERIC POND W/ STEE
PEST SLP & HIGHEST EMB & WEAK SOIL

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	40.00	34.00	40.00	1
2	34.00	40.00	43.00	53.00	1
3	43.00	53.00	47.00	58.00	1
4	47.00	58.00	55.00	58.00	1
5	55.00	58.00	73.00	46.00	1
6	73.00	46.00	82.00	40.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

7/8

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	40.00
2	82.00	40.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .100 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE = .0 PSF
A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 50.00 FT.
AND X = 80.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL

8/8

FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	26.67	40.00
2	34.63	39.25
3	42.41	41.12
4	49.17	45.41
5	54.17	51.65
6	56.18	57.21

*** 1.211 ***

FAILURE SURFACE SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	20.00	40.00
2	27.84	38.41
3	35.80	39.20
4	43.18	42.30
5	49.31	47.43
6	53.67	54.14
7	54.76	58.00

*** 1.267 ***

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	26.67	40.00
2	34.32	37.66
3	42.22	38.90
4	48.78	43.48
5	52.66	50.48
6	53.06	58.00

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STANDARD COMPUTATION SHEET

PROJECT Valley Camp JOB NO. 515889

PURPOSE Stab. of Sed. Ponds - SUMMARY SHEET 1 OF ~~14~~ 13

COMPUTED BY DBG CHECKED BY DATE 1/16/89

Strength Values

Pond	γ_m	γ_{sat}	C	ϕ	From
1	112	124	400	22	GARCo
2	107	122	150	31	RBG
3	107	122	150	31	RBG

Slope Geometry from HAL Figures

Pond	Slope		FS/	
1	up/st	2H:1V	1.7	⇐ Not Embankment
1A	up/st	2.2H:1V	2.1	
2	up/st	1.9H:1V	1.25	
3	up/st	1.5H:1V	1.28	
1A	dn/st	2.8H:1V	4.1	
2	dn/st	1.7H:1V	1.5	
3	dn/st	1.4-1.8H:1V	~~1.26~~ 1.5	

Slopes Req'd for FS=1.5 under Rapid Drawdown

Pond	Slope	FS.
2	2.5H:1V	1.5
3	2.4H:1V	1.5

Steady State Seepage - Static & Dynamic Cases

Pond	Slope		Ground Acceleration	FS
3	u/s	1.5H:1V	0	2.0
3	u/s	1.5H:1V	0.1g	1.6
3	d/s	1.4-1.8H:1V	0	~~1.9~~ 2.2
3	d/s	1.4-1.8H:1V	0.1g	~~1.6~~ 1.7

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STANDARD COMPUTATION SHEET

PROJECT Valley Camp Sedimentation Ponds JOB NO. 515889

PURPOSE Stability Analyses SHEET 2 OF 19

COMPUTED BY DBG CHECKED BY DATE 1/12/89

Strength Values & Lab Results

Pond 1A - From GARCO

Pond Bottom - Lt. Brown Sandy Clay w/ Silt
South & West Sides - Gray Silty Sand

Sample #1 & #5; Sandy Clay

w_i	γ_{di}	w_f	γ_{df}
15.2	115.7	17.3	108.2
15.8	113.4	16.9	129.4
14.7	121	15.8	126.4

$w_{AVG} = 16\%$ $\gamma_{d_{AVG}} = 117.4$ pcf

$S_r = 95\%$ $\gamma_m = 136.2$ pcf

$\gamma_{SAT} = 137.1$ pcf

$c = 550$ psf
$\phi = 23.5°$

Sample #2; Very Fine Sand

w_i	γ_{di}	w_f	γ_{df}
42.7	76.4	44.6	75.4
40.6	78.8	47.6	80.4
32.0	83.8	38.5	85.9

$w_{AVG} = 40\%$

$\gamma_{d_{AVG}} = 80.1$ pcf $\gamma_m = 112.1$ pcf

$S_r = 100\%$ $c = 256$ psf

$\gamma_{SAT} = 112.2$ pcf $\phi = 21.5°$

$w_{AVG} = 28\%$

$\gamma_{d_{AVG}} = 99$ pcf

$\gamma_{SAT} \approx \gamma_m = 124$ pcf

If $S_r = 50\%$, $\gamma_m = 112$ pcf

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STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 3 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

Ponds 2A, 3A - From RBG, Inc.

Pond 2A

Test Pit	pcf γ_d	% w	tsf torvane	ft depth
TP-1	100.2	6	0.35	0
	90.3	11	0.45	3
	89.8	15.9	0.50	6
	100.7	16.8	0.35	9

$c = 1$ psi $= 144$ psf
$\phi = 31.5°$

$\gamma_{d_{Av}} = 95.2$
$w = 12.4$
$S_r = 92.5$
$\gamma_{SAT} = 123$

Pond 3A

TP-2	96.8	8.6	0.55	0
	93.5	10.8	0.55	3
	92.1	17.6	0.55	6
	91.6	21.5	0.55	9
	93.6	17.3	0.55	12
	91.1	17.9	0.55	15
	95.8	12.2	0.55	18

$\gamma_{d_{Avg}} = 93.5$
$w_{Av} = 15.1$
$S_r = 122$

$w_{Avg} = 14.0\%$
$\gamma_{d\,Avg} = 94.1$ pcf

$\gamma_m = 107.3$ pcf

$S_r = 46.7\%$
$\gamma_{SAT} = 122.3$ pcf

v	w
0.229	
0.226	14.1
0.545	93.5

NET-145

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 4 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

Geometry of Ponds

POND 1A

Elevations:

Bottom of Pond	7803
Top Primary Spillway	7816.1
Top Emergency Spill'w	7818.8

1A - 1A'

Dist.	Elev.
0	7840
9	7835
18	7830
30	7825
41	7824
43	7823
46	7822
47.5	7821
49.5	7820
56	7819
57	7818
59	7817
60	7816
62	7815
79	7815

Silted in

1B - 1B'

Dist.	Elev.
0	7835
24	7830
40	7825
42	7824
45	7823
47	7822
49	7821
52	7820
58	7819
118	7819
80	7803

1C - 1C'

Dist	Elev
0	7803
35	19
41	20
43	21
55	21
58	20
72	78.5

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 5 OF 19

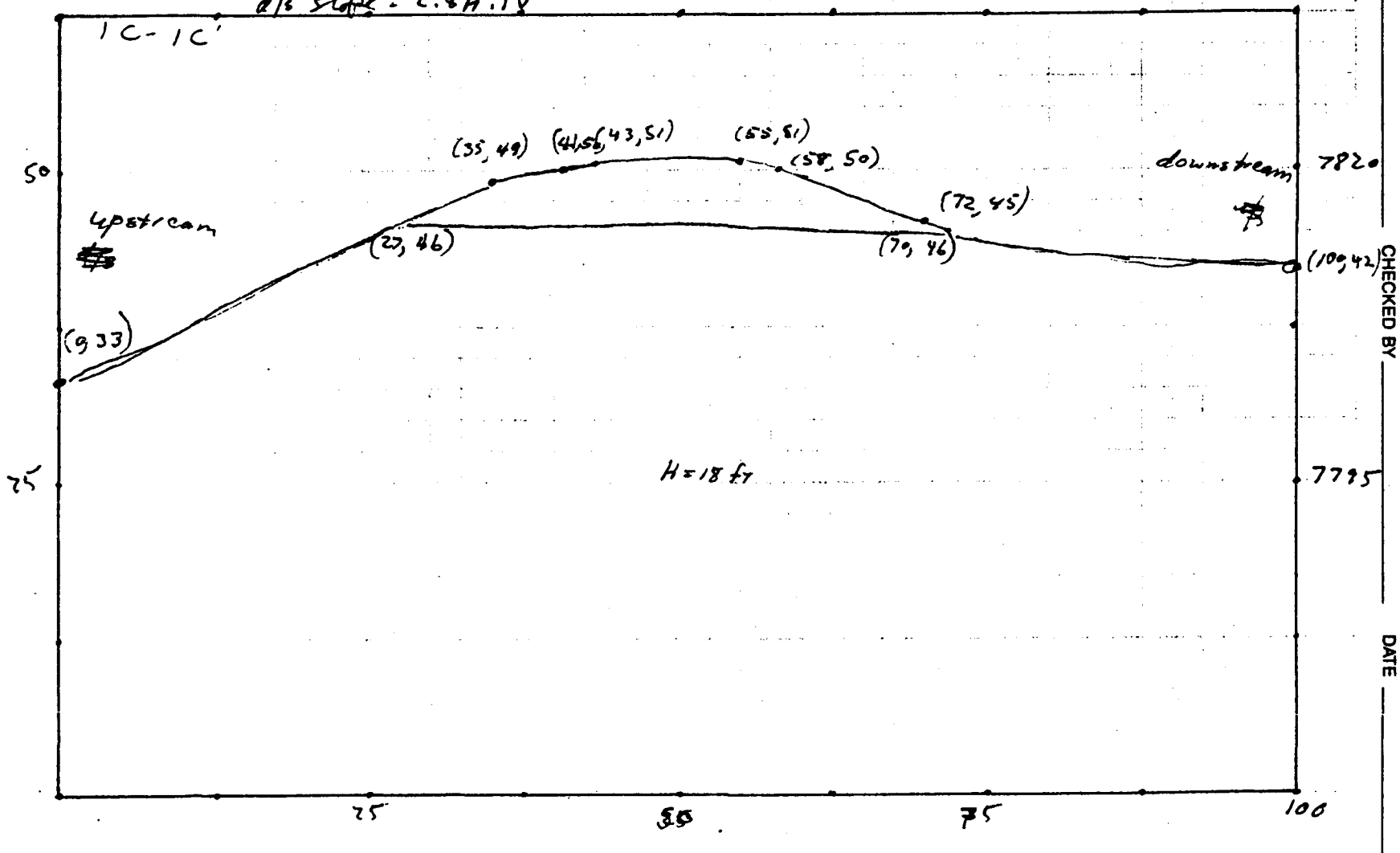
COMPUTED BY _____ CHECKED BY _____ DATE _____

~~Downstream slope~~

[illegible] Spillway, 16.1

u/s slope = 2.2H:1V

d/s slope = 2.8H:1V



NET 115

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 6 OF 18

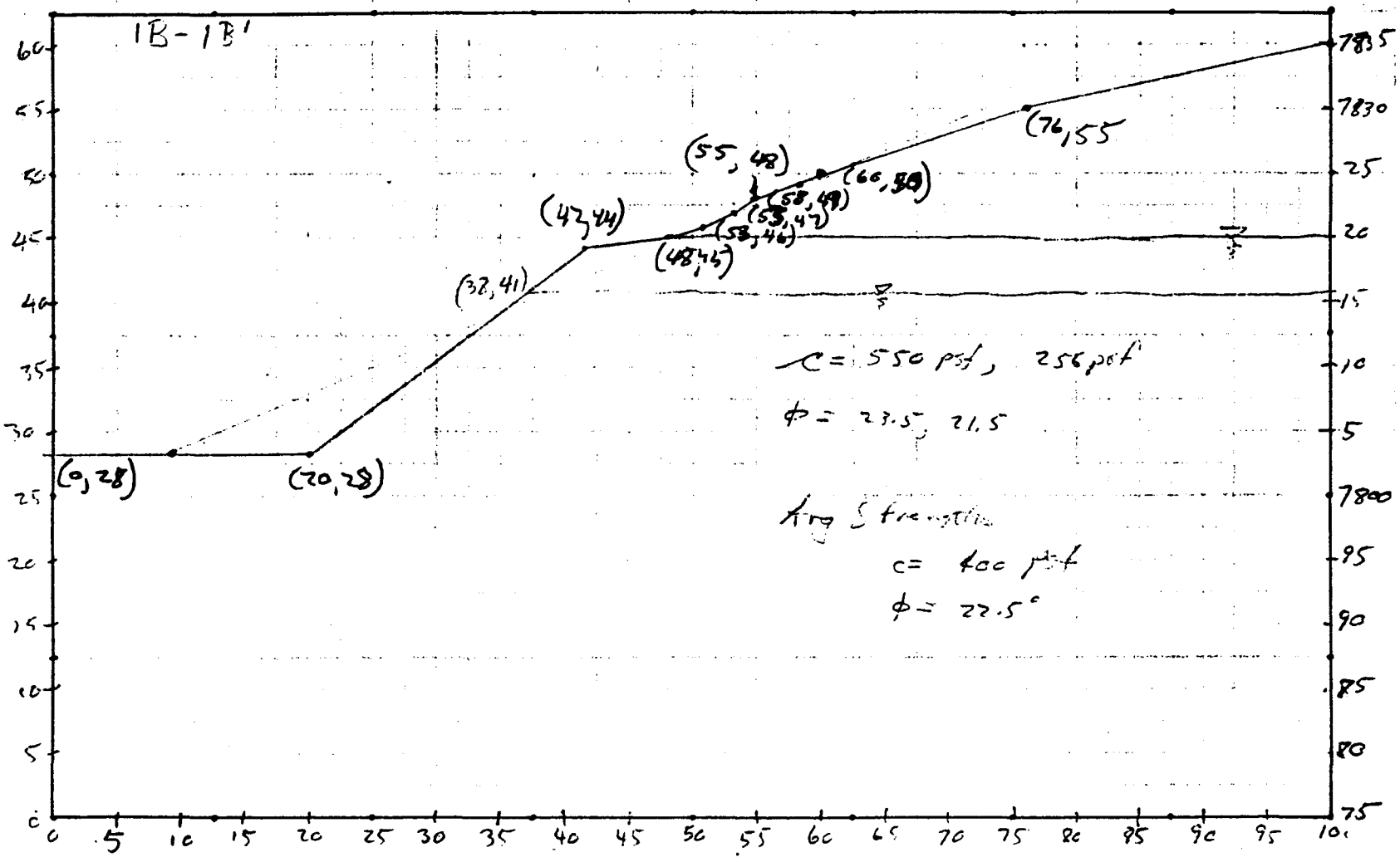
COMPUTED BY _____ CHECKED BY _____ DATE _____

$\frac{44-41}{44-38} = \frac{42-x}{42-30}$

$X = 38$

FS = 1.66

4/3 Slope: 1.8H:1V



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STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 7 OF 18

COMPUTED BY _____ CHECKED BY _____ DATE _____

Geometry

Pond 2A

Elevations:

Bottom of Pond	7823.6
Top of Primary Spillway	7835.4
Top of Emergency Spillway	7836.3

Section 2A-2A'

Dist.	Elev.
0	7836
48	7836
51	7837
53	7838
61	7838
63	7837
66	7836
68	7835
74	7830
83	7825

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STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 8 OF 18

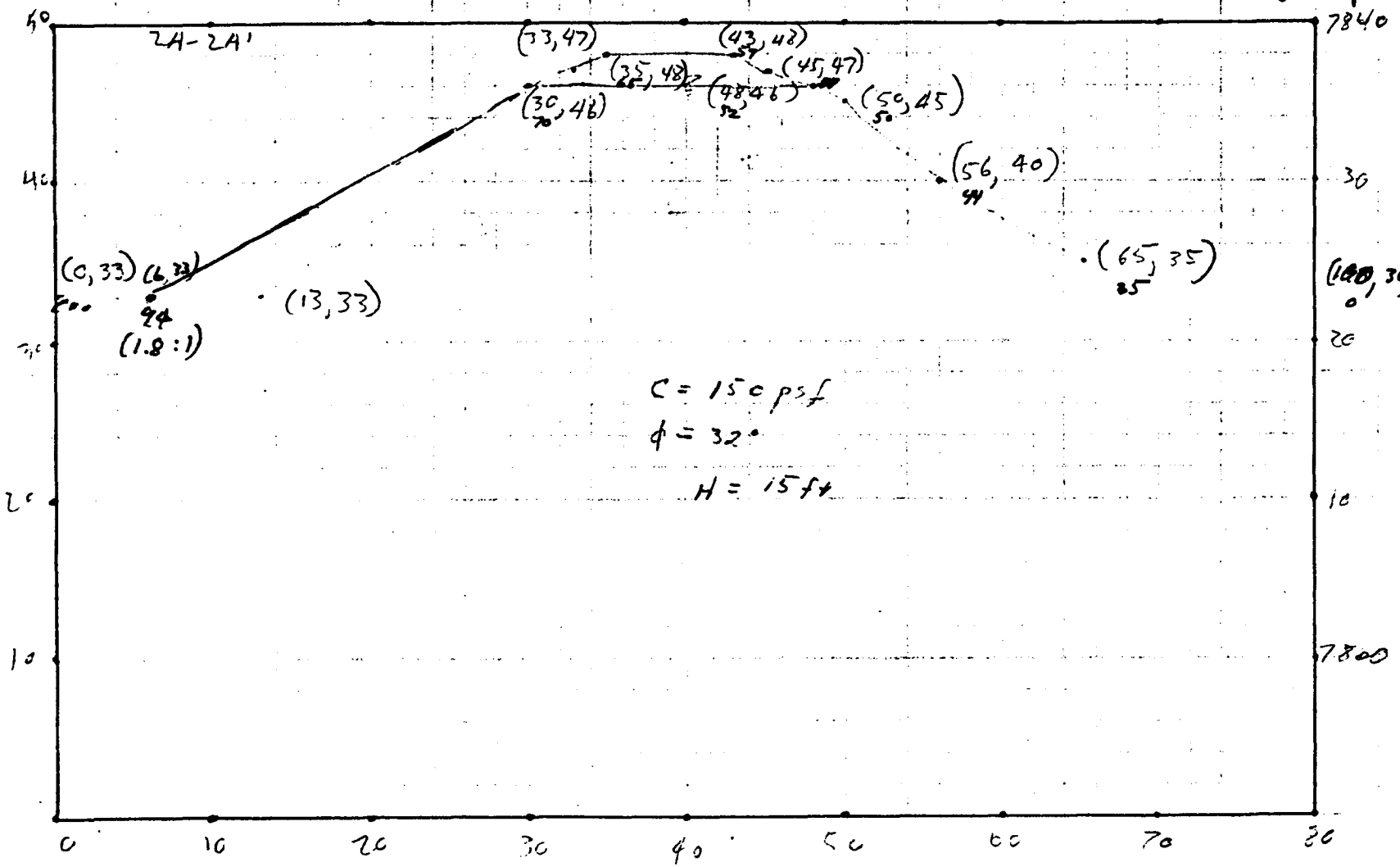
COMPUTED BY _____ CHECKED BY _____ DATE _____

1.515

~~1.47~~

~~1.148~~ ⇐ Orig Conf.

upstream slope



NFT 145

Northern Engineering and Testing, Inc.

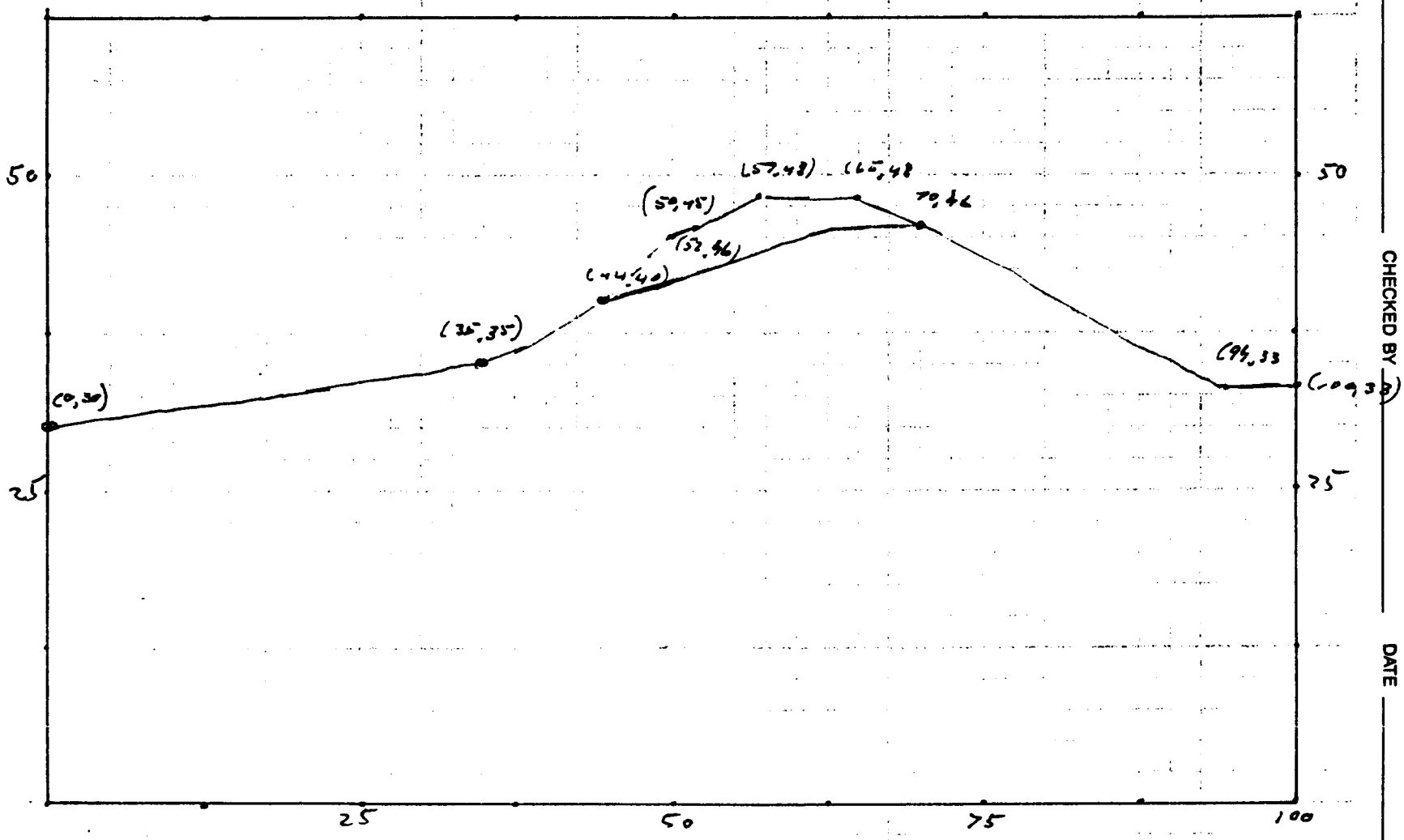
STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 9 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

2A-2A' downstream Slope (orig: 1.7H:1V)



NET 145

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 10 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

Geometry

POND 3A

Elevations:

Bottom of Pond	7855.0
Top of primary discharge	7863.2
Top of Emergency discharge	7865.5

Section 3A-3A'

Dist.	Elev		
0	7860	7859	0
79	7860	7859	25
80	7861	7860	26
82	7862	7861	28
84	7863	7862	30
86	7864	7863	32
89	7865	7864	35
92	7866	7868	38
110	7866	7868	56
114	7865	7865	60
121	7860	7860	67
133	7855	7855	79
155	7850	7850	100

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STANDARD COMPUTATION SHEET

PROJECT

PURPOSE

COMPUTED BY

CHECKED BY

DATE

JOB NO.

SHEET 11 OF 18

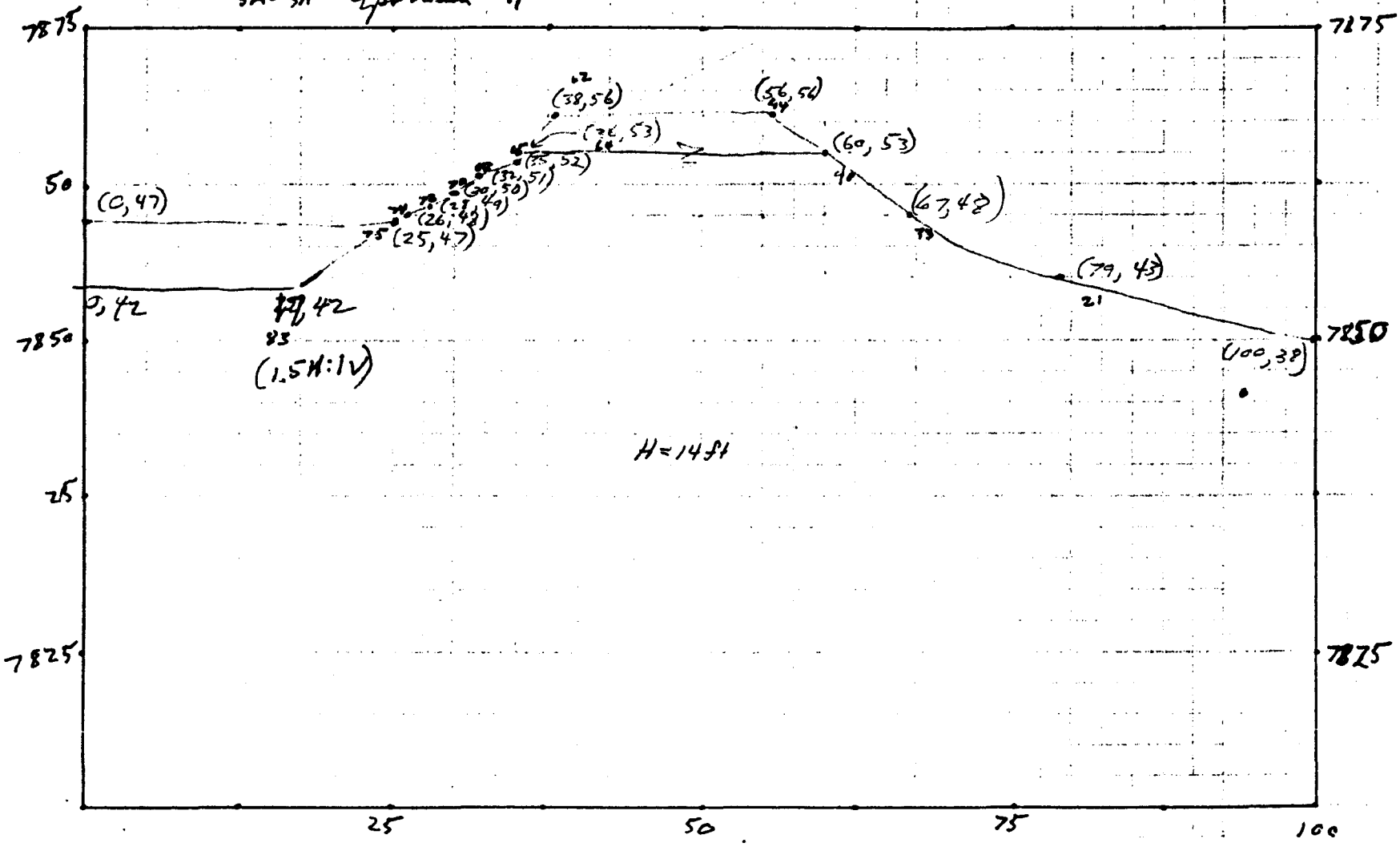
$\frac{56-53}{56-52} = \frac{38-x}{38-35}$

1.48

~~K = 1.497~~ } Re-sloped

FS = 1.284

3A-3A' upstream Slope



NET 145

Northern Engineering and Testing, Inc.

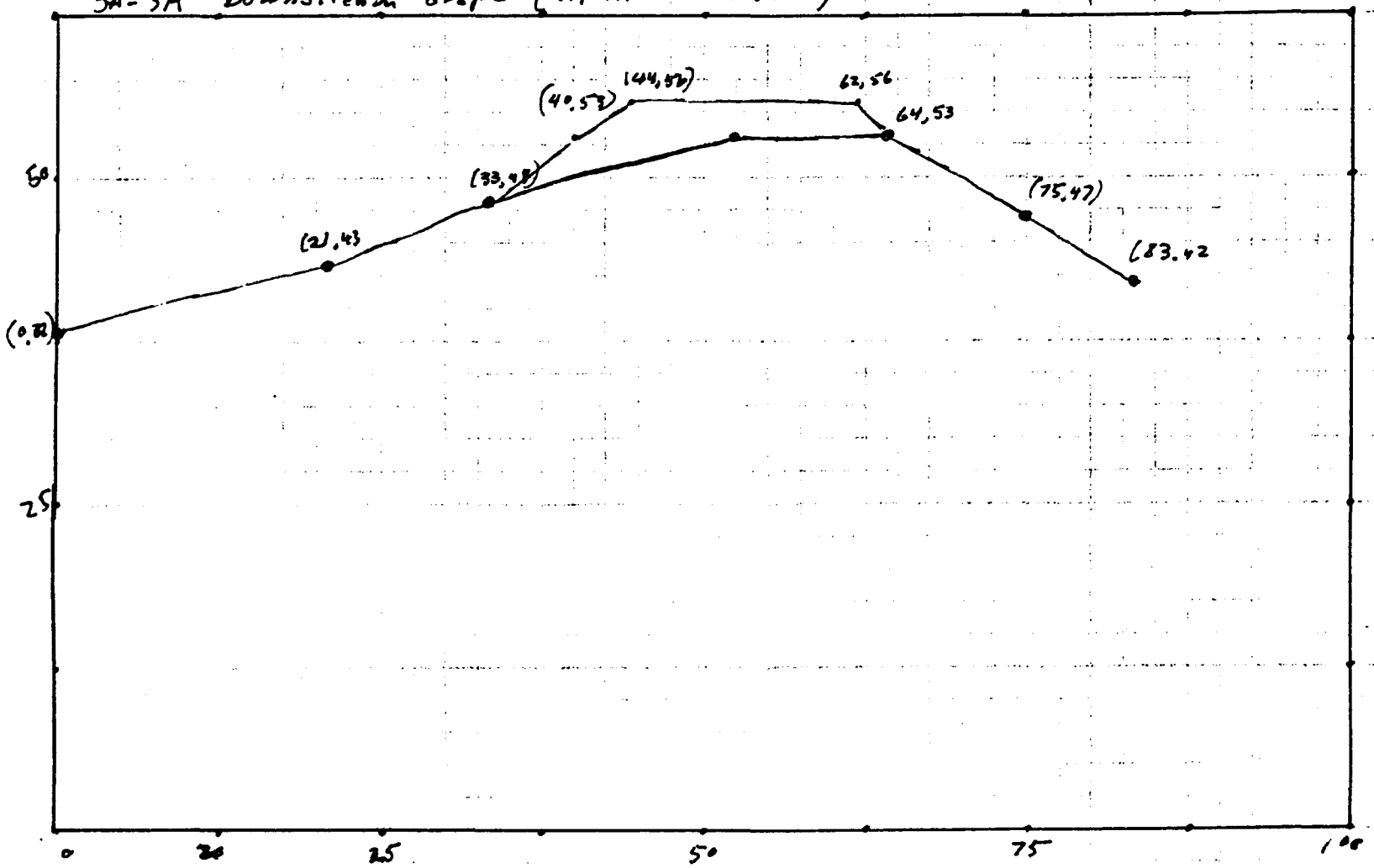
STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 12 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

3A-3A' Downstream Slope (1.4H:1V to 1.8H:1V)



HET-145

Northern Engineering and Testing, Inc.

STANDARD COMPUTATION SHEET

PROJECT _____ JOB NO. _____

PURPOSE _____ SHEET 13 OF 13

COMPUTED BY _____ CHECKED BY _____ DATE _____

Iteration I : Existing Slopes - Upstream Slopes

Pond	Slope	EQUAK	FS
1	2H:1V	No	1.7
1A	2.2H:1V	No	2.1
2	1.9H:1V	No	1.25
3	1.5H:1V	No	1.28

Iteration II : Slopes Req'd For FS=1.5, Upstream Slopes

1	2H:1V	No	1.7
2	2.5H:1V	No	1.5
3	2.4H:1V	No	1.5

Iteration III : Downstream Slopes, Static, Rapid Drawdown

1	2.8H:1V	No	4.1
2	1.7H:1V	No	1.5
3	1.4H- 1.8H:1V	No	~~1.26~~ 1.5

Iteration 3 : Steady State Seepage

3	u/s	1.5H:1V	No	2.0
3	u/s	1.5H:1V	0.1g	1.6
3	d/s	1.4-1.8H:1V	No	~~1.9~~ 2.2
3	d/s	1.4-1.8H:1V	0.1g	~~1.6~~ 1.7

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. POND STABILITY; RAPID DRAWDOWN; UPSTREAM; POND 1A

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	33.00	27.00	46.00	1
2	27.00	46.00	35.00	49.00	1
3	35.00	49.00	43.00	51.00	1
4	43.00	51.00	55.00	51.00	1
5	55.00	51.00	72.00	45.00	1
6	72.00	45.00	100.00	42.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	112.0	124.0	400.0	22.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 4 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	33.00
2	27.00	46.00
3	72.00	45.00
4	100.00	42.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 15.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 30.00 FT.
AND X = 70.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 8 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	.00	33.00
2	9.45	29.74
3	19.38	28.53
4	29.34	29.43
5	38.89	32.39
6	47.60	37.30
7	55.09	43.93
8	59.21	49.51

*** 2.132 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SEDIMENTATION POND NO. 2A;
RAPID DRAWDOWN; UPSTREAM

BOUNDARY COORDINATES

7 TOP BOUNDARIES
7 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	33.00	26.00	33.00	1
2	26.00	33.00	50.00	46.00	1
3	50.00	46.00	55.00	48.00	1
4	55.00	48.00	63.00	48.00	1
5	63.00	48.00	68.00	46.00	1
6	68.00	46.00	76.00	40.00	1
7	76.00	40.00	85.00	35.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	123.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 5 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	33.00
2	26.00	33.00
3	50.00	46.00
4	68.00	46.00
5	85.00	35.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.

AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 48.00 FT.
AND X = 85.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	23.33	33.00
2	30.94	30.54
3	38.94	30.92
4	46.28	34.10
5	52.02	39.67
6	55.42	46.91
7	55.50	48.00

*** 1.248 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SEDIMENTATION POND NO. 3A;
RAPID DRAWDOWN; UPSTREAM

BOUNDARY COORDINATES

8 TOP BOUNDARIES
8 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	42.00	17.00	42.00	1
2	17.00	42.00	25.00	47.00	1
3	25.00	47.00	36.00	53.00	1
4	36.00	53.00	38.00	56.00	1
5	38.00	56.00	56.00	56.00	1
6	56.00	56.00	60.00	53.00	1
7	60.00	53.00	67.00	48.00	1
8	67.00	48.00	79.00	43.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	108.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	42.00
2	17.00	42.00
3	25.00	47.00
4	36.00	53.00
5	60.00	53.00
6	67.00	47.00
7	79.00	43.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 20.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 32.00 FT.
AND X = 79.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	15.56	42.00
2	23.40	40.40
3	31.28	41.76
4	38.13	45.89
5	43.02	52.22
6	44.13	56.00

*** 1.284 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. PONDS; POND 1A DOWNSTREAM; RAP. DRAW.

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	42.00	28.00	45.00	1
2	28.00	45.00	45.00	51.00	1
3	45.00	51.00	57.00	51.00	1
4	57.00	51.00	65.00	49.00	1
5	65.00	49.00	73.00	46.00	1
6	73.00	46.00	100.00	33.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	112.0	124.0	400.0	22.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 4 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	42.00
2	28.00	45.00
3	73.00	46.00
4	100.00	33.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 28.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 42.00 FT.
AND X = 70.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

10.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 8 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	.00	42.00
2	7.07	34.93
3	15.99	30.41
4	25.87	28.88
5	35.74	30.50
6	44.62	35.11
7	51.62	42.25
8	55.95	51.00

*** 4.143 ***

^Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. POND 2A; DOWNSTREAM SLP; RAP. DRAW.

BOUNDARY COORDINATES

8 TOP BOUNDARIES
8 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	30.00	35.00	35.00	1
2	35.00	35.00	44.00	40.00	1
3	44.00	40.00	52.00	46.00	1
4	52.00	46.00	57.00	48.00	1
5	57.00	48.00	65.00	48.00	1
6	65.00	48.00	70.00	46.00	1
7	70.00	46.00	94.00	33.00	1
8	94.00	33.00	100.00	33.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	30.00
2	35.00	35.00
3	44.00	40.00
4	60.00	45.00
5	70.00	46.00
6	94.00	33.00
7	100.00	33.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 35.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 50.00 FT.
AND X = 80.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 8 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	23.33	33.33
2	30.71	30.23
3	38.65	29.24
4	46.56	30.43
5	53.85	33.72
6	59.98	38.86
7	64.49	45.47
8	65.31	47.88

*** 1.456 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. POND 3A; DOWNSTREAM; PID
DRAWDOWN
RAPID DRAWDOWN

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	38.00	21.00	43.00	1
2	21.00	43.00	33.00	48.00	1
3	33.00	48.00	44.00	56.00	1
4	44.00	56.00	62.00	56.00	1
5	62.00	56.00	64.00	53.00	1
6	64.00	53.00	83.00	42.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 7 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	38.00
2	21.00	43.00
3	33.00	48.00
4	52.00	53.00
5	64.00	53.00
6	75.00	47.00
7	83.00	42.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED

ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 41.00 FT.
AND X = 75.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 8 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	10.00	40.38
2	17.54	37.69
3	25.52	37.12
4	33.36	38.71
5	40.49	42.34
6	46.38	47.75
7	50.61	54.54
8	51.05	56.00

*** 1.484 ***

2

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SEDIMENTATION POND NO. 3A;
UPSTREAM; STEADY-STATE; DYNAMIC

BOUNDARY COORDINATES

8 TOP BOUNDARIES
8 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	42.00	17.00	42.00	1
2	17.00	42.00	25.00	47.00	1
3	25.00	47.00	36.00	53.00	1
4	36.00	53.00	38.00	56.00	1
5	38.00	56.00	56.00	56.00	1
6	56.00	56.00	60.00	53.00	1
7	60.00	53.00	67.00	48.00	1
8	67.00	48.00	79.00	43.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	108.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	42.00
2	79.00	43.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .100 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE = .0 PSF
1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM

TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 20.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 32.00 FT.
AND X = 79.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	15.56	42.00
2	23.40	40.40
3	31.28	41.76
4	38.13	45.89
5	43.02	52.22
6	44.13	56.00

*** 1.623 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. POND 3A; DOWNSTREAM;
STEADY-STATE; DYNAMIC, A=0.1g

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	38.00	21.00	43.00	1
2	21.00	43.00	33.00	48.00	1
3	33.00	48.00	44.00	56.00	1
4	44.00	56.00	62.00	56.00	1
5	62.00	56.00	64.00	53.00	1
6	64.00	53.00	83.00	42.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	38.00
2	83.00	42.00

A HORIZONTAL EARTHQUAKE LOADING COEFFICIENT
OF .100 HAS BEEN ASSIGNED

A VERTICAL EARTHQUAKE LOADING COEFFICIENT
OF .000 HAS BEEN ASSIGNED

CAVITATION PRESSURE = .0 PSF
1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 41.00 FT.
AND X = 75.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION
AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL
FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL
FIRST.

FAILURE SURFACE SPECIFIED BY 10 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	.00	38.00
2	7.24	34.59
3	14.97	32.55
4	22.95	31.95
5	30.91	32.79
6	38.58	35.06
7	45.71	38.67
8	52.08	43.52
9	57.46	49.44
10	61.54	56.00

*** 1.665 ***

Z

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SEDIMENTATION POND NO. 3A;
UPSTREAM; STEADY-STATE; STATIC

BOUNDARY COORDINATES

8 TOP BOUNDARIES
8 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	42.00	17.00	42.00	1
2	17.00	42.00	25.00	47.00	1
3	25.00	47.00	36.00	53.00	1
4	36.00	53.00	38.00	56.00	1
5	38.00	56.00	56.00	56.00	1
6	56.00	56.00	60.00	53.00	1
7	60.00	53.00	67.00	48.00	1
8	67.00	48.00	79.00	43.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	108.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	42.00
2	79.00	42.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 20.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 32.00 FT.
AND X = 79.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	15.56	42.00
2	23.40	40.40
3	31.28	41.76
4	38.13	45.89
5	43.02	52.22
6	44.13	56.00

*** 2.013 ***

-2

--SLOPE STABILITY ANALYSIS--
SIMPLIFIED JANBU METHOD OF SLICES
IRREGULAR FAILURE SURFACES

PROBLEM DESCRIPTION VALLEY CAMP SED. POND 3A; DOWNSTREAM;
STEADY STATE; STATIC

BOUNDARY COORDINATES

6 TOP BOUNDARIES
6 TOTAL BOUNDARIES

BOUNDARY NO.	X-LEFT (FT)	Y-LEFT (FT)	X-RIGHT (FT)	Y-RIGHT (FT)	SOIL TYPE BELOW BND
1	.00	38.00	21.00	43.00	1
2	21.00	43.00	33.00	48.00	1
3	33.00	48.00	44.00	56.00	1
4	44.00	56.00	62.00	56.00	1
5	62.00	56.00	64.00	53.00	1
6	64.00	53.00	83.00	42.00	1

1 ISOTROPIC SOIL PARAMETERS

1 TYPE(S) OF SOIL

SOIL TYPE NO.	TOTAL UNIT WT. (PCF)	SATURATED UNIT WT. (PCF)	COHESION INTERCEPT (PSF)	FRICTION ANGLE (DEG)	PORE PRESSURE PARAMETER	PRESSURE CONSTANT (PSF)	PIEZOMETRIC SURFACE NO.
1	107.0	122.0	150.0	31.0	.00	.0	1

1 PIEZOMETRIC SURFACE(S) HAVE BEEN SPECIFIED

UNITWEIGHT OF WATER = 62.40

PIEZOMETRIC SURFACE NO. 1 SPECIFIED BY 2 COORDINATE POINTS

POINT NO.	X-WATER (FT)	Y-WATER (FT)
1	.00	38.00
2	83.00	42.00

1 A CRITICAL FAILURE SURFACE SEARCHING METHOD, USING A RANDOM
TECHNIQUE FOR GENERATING CIRCULAR SURFACES, HAS BEEN SPECIFIED.

100 TRIAL SURFACES HAVE BEEN GENERATED.

10 SURFACES INITIATE FROM EACH OF 10 POINTS EQUALLY SPACED
ALONG THE GROUND SURFACE BETWEEN X = .00 FT.
AND X = 30.00 FT.

EACH SURFACE TERMINATES BETWEEN X = 41.00 FT.

AND X = 75.00 FT.

UNLESS FURTHER LIMITATIONS WERE IMPOSED, THE MINIMUM ELEVATION AT WHICH A SURFACE EXTENDS IS Y = .00 FT.

8.00 FT. LINE SEGMENTS DEFINE EACH TRIAL FAILURE SURFACE.

THE FACTOR OF SAFETY FOR THE TRIAL FAILURE SURFACE DEFINED BY THE COORDINATES LISTED BELOW IS MISLEADING.

FAILURE SURFACE DEFINED BY 7 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	30.00	46.75
2	36.95	42.79
3	44.57	40.34
4	52.52	39.52
5	60.48	40.35
6	68.09	42.81
7	74.88	46.70

FACTOR OF SAFETY FOR THE PRECEDING SPECIFIED SURFACE =*******
1 FOLLOWING ARE DISPLAYED THE TEN MOST CRITICAL OF THE TRIAL FAILURE SURFACES EXAMINED. THEY ARE ORDERED - MOST CRITICAL FIRST.

FAILURE SURFACE SPECIFIED BY 6 COORDINATE POINTS

POINT NO.	X-SURF (FT)	Y-SURF (FT)
1	20.00	42.76
2	27.82	41.06
3	35.73	42.25
4	42.69	46.20
5	47.78	52.37
6	48.99	56.00

*** 2.173 ***

Z

Golder Associates
CONSULTING MINING AND GEOTECHNICAL ENGINEERS

SURFACE FACILITIES GRADING PLAN
BELINA MINE AREA

PREPARED FOR

VALLEY CAMP OF UTAH
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8 March 1983

Golder Associates
CONSULTING GEOTECHNICAL AND MINING ENGINEERS

OCT 7 1982
VALLEY CAMP OF UTAH

G/82/298
September 30, 1982

Valley Camp Coal Company
Scofield Route
Helper, Utah 84526

Attention: Mr. Trevor Whiteside

Subject: Belina Mine Area
Surface Facilities Grading Plan

Gentlemen:

With reference to our engineering report entitled, "Surface Facilities Grading Plan, Belina Mine Area" dated June 1980, this letter will confirm that the engineering analyses and recommendations presented in the report were made under my supervision, and that the drawings presented in the report (list attached) are correct as shown.

Sincerely,

GOLDER ASSOCIATES

A. Allen Gass, Principal, P.E.
State of Utah # 4890

AAG/CWL:hd
Attachment
793-1156

REGISTERED PROFESSIONAL ENGINEER
No. 4390
A. ALLEN GASS
STATE OF UTAH

GOLDER ASSOCIATES, INC • 2950 NORTHUP WAY, BELLEVUE (SEATTLE), WASHINGTON 98004-1486, U.S.A. • TELEPHONE (206) 827-0777 • TELEX 32-1014

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1.0 INTRODUCTION

This report presents the results of continuing geotechnical studies carried out by Golder Associates to date at the Belina Mine Site, operated by Valley Camp of Utah, Helper, Utah. The work was initiated based on a proposal to Valley Camp dated July 19, 1979 and accepted per letter from Valley Camp dated August 2, 1979. Subsequent to the proposal, the work has been extended through verbal agreements.

The work has involved the development of the Belina Mine Area surface facilities in regard to the operational requirements of Valley Camp and the regulatory requirements of the State of Utah Division of Oil, Gas and Mining (DOGM), and the Office of Surface Mining (OSM) of the U.S. Department of the Interior.

At the start of the work (August 1979), the Belina No. 1 Mine was in operation under an approved 30CFR211 and Utah Division of Oil Gas and Mining Mine Plan and Permit. Valley Camp's operational plan called for the continued development of the Belina No. 1 Mine, including the development of surface facilities for the No. 1 Mine and the construction of a coal carrying conveyor system and load storage facility for the No. 1 Mine, as set out in general in the originally approved plans. Golder Associates has provided engineering services relative to the geotechnical engineering and hydrologic control structures relative to the proposed development and the current OSM Regulations. This work has included the design of cut and fill slopes and pads, drainage measures, and a sedimentation pond. In addition, construction control and periodic inspection of the work have been carried out at appropriate intervals when required.

2.0 GENERAL SITE CONDITIONS

The Belina Mine Area is located in the upper reaches of Whisky Gulch, a tributary to Eccles Canyon and eventually Pleasant Valley Creek located in the Scofield Drainage of western Carbon County, Utah. Whisky Gulch in the vicinity of the Belina Mine area is an intermittent stream under the definitions provided by the OSM in that it obtains its flow from both surface runoff and ground water discharge at least some part of the year.

The surface facilities area for the Belina mines ranges from an elevation of about 8800 to 9200 feet above sea level. The terrain is steep, and heavily forested with stands of aspen and conifers. Soil cover is thin, in the range of 2 to 10 feet maximum depth, and is in general colluvial material derived from the parent sandstones and siltstones which comprise the rock exposed in the area.

Average annual precipitation at the mine site is about 25-30 inches. Of this, approximately 8 inches occurs as rainfall, generally from May through September.

The general vicinity of the Belina Mine is shown on Figure 2-1. As can be seen, the mining area is on what is known as the Connelville Block, located between the Connelville fault and the O'Connor Fault. The rock units which comprise the Whisky Gulch Area are members of the Blackhawk Formation.

As shown in Figure 2-1, the Belina No. 1 facilities are located on the western flank of Whisky Gulch. The coal handling facilities, first phase, as approved will occupy the gulch itself and portions of the eastern flank. Existing natural slopes in the area are in the range of 20° to 25°.

At the time Golder Associates became involved with the Belina Mine Area (August 1979), a large earthfill had been placed in Whisky Gulch to provide access and work area for the coal storage area. The fill pad was at approximate elevation 8940 at its upper end, sloping down to an existing sedimentation pond constructed at the downstream crest of the fill. At this time, the sediment pond area was exhibiting some local signs of instability in the form of tension cracks, and water was observed seeping out along the toe of the fill. Also, the slopes extending from the Belina No. 1 mine area to the fill pad were steep and exhibiting signs of sloughing.

In order to stabilize the sediment pond area, as well as the Belina slopes, and to provide areas for the surface facilities, Golder Associates recommended that the downstream portion of the existing valley fill be removed and replaced in an engineered manner with a rock toe buttress and a graded filter. The sediment pond was then located at the top of the fill. Additionally, recommendations were made concerning stabilizing the existing slopes from the No. 1 mine area down to the coal storage area. This work resulted in a surface facilities grading plan as presented in Figure 2-2.

To the extent possible, all grading was carried out in conjunction with the requirements of the OSM regulations. In the few areas where this was not possible, engineering analyses of variation were carried out to insure the overall stability and satisfactory performance of the structures.

3.0 DRAINAGE CONTROL

3.1 INTRODUCTION

Control of drainage, both surface water and groundwater, is critical to the overall development of the surface facilities. This includes surface precipitation water, snowmelt, ground water intercepted in cuts seeps or springs, and ground water intercepted and collected in the mine workings and pumped to the surface for ultimate discharge. This section presents the methodologies used to predict the flows expected for the various watersheds encountered in the Belina Mine Area.

3.2 DRAINAGE QUANTITIES

Peak flow on the small watersheds in this study area were estimated using the Rational Method (Grey, 1973). This method is based on the criteria that for storms of uniform intensity, evenly distributed over the watershed, the maximum rate of runoff occurs when the entire watershed is contributing at the outlet and that this rate of runoff, or flow, is proportional to the rainfall intensity. The equation is:

$$Q = ciA$$

where:

- c = Runoff coefficient, determined empirically
- i = Maximum rainfall intensity, in/hr, whose duration is equal to the time of concentration of the watershed
- A = Area of watershed, acres
- Q = Peak flow, cfs.

TABLE 3-1

ESTIMATED PRECIPITATION DEPTHS FOR VARIOUS RETURN PERIODS AND DURATIONS AT CLEAR CREEK, SUMMIT, UTAH (FROM RICHARDSON, 1971)

	DURATION									
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.10	.16	.20	.28	.35	.46	.57	.84	1.08	1.33
2	.12	.19	.25	.34	.43	.57	.70	1.04	1.34	1.65
5	.16	.24	.31	.43	.54	.72	.90	1.34	1.73	2.14
10	.19	.29	.37	.51	.65	.86	1.06	1.55	1.99	2.45
25	.24	.38	.48	.66	.84	1.08	1.31	1.88	2.39	2.92
50	.25	.38	.48	.67	.85	1.13	1.40	2.07	2.67	3.29
100	.27	.42	.53	.73	.93	1.24	1.54	2.29	2.96	3.65

Maximum rainfall intensities for specified return periods were calculated using precipitation data from a gauging station at Clear Creek, Summit, Utah, as reported by Richardson (1971), (Table 3-1). The duration time used when choosing the rainfall intensity was equal to the time of concentration of the particular section of the drainage basin. A 10 year return period was used when choosing the rainfall intensity, as specified by the OSM regulation for permanent diversions.

The time of concentration was determined using the Upland Flow Method (Kent, 1972). Types of flow considered in the Upland Method are: overland, through grassed waterways, over paved areas, and through small upland gullies. Upland flow employed in this method can be a combination of these various surface runoff conditions. The velocity of flow is determined using Figure 3-1. The time of concentration of the drainage basin then is equal to the sum of the times required for water falling on the farthest point of the watershed, flowing over various types of terrain, to reach the outlet.

The runoff coefficient in the Rational Method is dependent on the topography, soil type and vegetation of the watershed. Values of the runoff coefficient can be found in Grey (1973). An estimate of the coefficient was made from these charts assuming:

Topography - Hilly Land
Soil - Open Sandy Loam $c = 0.03$
Cover - Woodlands

The drainage area of concern at the Belina Mine site was divided into six sections (see Figure 3-2). The drainage characteristics and outflow were calculated for each section. Table 3-2 lists the values used and outflow for each section. Outflow from section I and VI is to be diverted outside the area of

concern while the outflow from sectins II-V is to be carried via a culvert under the mine site and out of the drainage area. Water falling within the mine area is to be treated in a settling pond and then discharged.

TABLE 3-2
CALCULATION OF FLOW RATES BY SECTION

SECTION NUMBER	I	II	III	IV	V	VI
AREA (acres)	23.6	28.3	29.8	36.5	36.1	8.3
tc (min)	8.6	14.7	15.3	24.3	24.2	17.4
i (in/hr)	1.74	1.48	1.48	1.25	1.25	1.48
c RUNOFF COEFFICIENT	0.3	0.3	0.3	0.3	0.3	0.3
Q =ciA (cfs)	12.3	12.6	13.2	13.7	13.5	3.7

4.0 SEDIMENT POND

4.1 HYDROLOGIC DATA

Hydrologic analyses were made to calculate the runoff volume, from the disturbed area, that would be treated in the sediment pond (see Figure 3-2). The calculated storage volume was based on the 10-year 24-hour storm as required by OSM Regulations. Previous experience has indicated that a 48-hour detention time is required to insure adequate settling.

Hydrographs for the Belina Mine Area were obtained from a report to Valley Camp from Vaughn Hansen Associates. The Hydrograph presented in the report was for a 25-year 24-hour storm. The Hydrograph for the 10-year 24-hour storm is assumed to be of similar shape. The peak inflow into the pond was calculated assuming the hydrograph could be represented by a triangle. The peak runoff for the 10-year 24-hour hydrograph, shown in Figure 4-1, is determined according to the equation:

$$q_p = \frac{484\ AQ}{T_p}$$

where,

q_p = Peak Runoff, cfs
A = Area, sq. miles
Q = Net storm rain, inches (Runoff only)
Tp = Time to peak runoff, hours.

The volume of runoff is then defined as the area under the hydrograph.

4.2 SEDIMENT POND DESIGN

The outflow hydrograph of the sediment pond for the design storm was constructed based on a constant outflow from the principal spillway (spillway design will be discussed in Section 4.3). Since the outflow from the sediment pond will be constant and the detention time has been specified as 48 hours, a direct determination of the outflow hydrograph for this design storm is possible, the constant outflow can be determined from the equations:

$$Vi = Vo$$
$$Vi = q_o \times tb$$

where,

Vi = Volume Inflow
Vo = Volume Outflow
q_o = Constant Outflow Rate, cfs
tb = Duration of hydrograph, hours.

The calculated outflow hydrograph is shown with the inflow hydrograph in Figure 4-1. The constant outflow required for the design storm and a 48 hour detention time is 0.94 cfs.

The storage required for the 10-year 24-hour storm can be determined from the inflow and outflow hydrographs. The difference between the area under the inflow hydrograph and that under the outflow hydrograph during the time of inflow is the storage volume; this is shown as the shaded area in Figure 4-1. The pond storage volume necessary to hold the runoff for 48 hours from the design storm is 6.7 acre-feet. The pond must be of sufficient volume to hold both the accumulated sediment and the design storm runoff.

Estimates of the volume of sediment were made from the 0.1 acre-feet per acre of disturbed land value recommended by the OSM Regulations. This results in a total sediment volume of 3.6 acre-feet. The total volume required for the storage pond would be the water volume plus the sediment volume which is 10.3 acre-feet.

The stage-storage curve for the existing pond is shown in Figure 4-2. The volume required for both the sediment and the water storage are marked on the curve as well as the spillway elevation. The existing pond with a storage volume of 11.2 acre-feet is sufficient to hold both the design sediment load and design runoff for a 48 hour detention time. It is understood that the existing decant is at elevation 8856. This is satisfactory for retention of 60% of the maximum allowable sediment level in addition to retaining the design storm for 48 hours. In accordance with the regulations, the accumulated sediment must be removed when it reaches 60% of maximum. It should be noted that, with the fixed decant system, it will be necessary to dewater the pond by pumping after precipitation events in order to provide storm surcharge capacity.

In accordance with Mine Safety and Health Administration Regulation (30 CFR 77.216-1) a permanent identification marker should be located on or immediately adjacent to the sediment pond. This marker should be at least 6 feet high and show the identification number of the impounding structure as assigned by the District Manager, the name associated with the impounding structure and name of the person owning, operating, or controlling this structure.

It is understood that the sediment pond may be used to collect and discharge mine water, as well as surface runoff. If this should be required, we recommend that the fixed elevation decant be replaced by a floating orifice type decant (see Figure

4-3). This design provides a constant discharge at an easily controlled rate and also skims water from the top of the pond, which tends to increase the effectiveness of settling pond performance (Skelly and Loy, 1979). Further, this type of decant will dewater the pond, thereby eliminating the need for pumping, and constantly providing storm surcharge capacity.

The collapsible pipe, shown in Figure 4-3, must extend from maximum pool level to permanent pool level (i.e., from elevation 8849.5 feet to 8856.6 feet). The inlet of the floating orifice is attached to floats so that it is submerged to give a desired outflow rate. The required outflow rate of 0.94 cfs can be accomplished using a variety of combinations of pipe diameter and head (see Figure 4-4). Any pipe diameter that is available could be used for the inlet so long as the decant is constructed with the appropriate head. The top of the pipe must be protected from logging by a trash rack or other suitable device.

4.3 SPILLWAY DESIGN

The emergency spillway was designed to handle the OSM Regulation's design storm of 25-years 24-hours. The emergency spillway elevation is at 8856.0 feet and the crest of the embankment if at 8860.0 feet. OSM regulations require at least 1.0 feet of clearance between the maximum elevation of water in the emergency spillway and the crest of the embankment. This limits the maximum depth of flow in the emergency spillway to 1.0 feet. At this flow depth the flow rate in the emergency spillway calculated by the Manning equation would be 85.0 cfs (see Figure 4-5). The flow rate required to handle the 25-year 24-hour storm was calculated to be 15.9 cfs, from equation 4-1. In conclusion, the emergency spillway can handle flow rates in excess of the 25-year 24-hour storm while still satisfying the OSM Regulations concerning freeboard.

4.4 CONSTRUCTION

Construction of the valley fill and sediment pond structure was commenced in September 1979 and essentially completed in December 1979. The construction of the valley fill was carried out under the full-time inspection of a geotechnical field engineer, who was responsible for insuring that the construction was carried out in conjunction with standard engineering practices and the requirements of the Division of Oil, Gas, and Mining. This consisted of material inspection, stripping and removal of topsoil and organic debris, and placement of the fill materials, including the rock buttress. The granular filter and underdrains, and the general embankment fill.

4.4.1 Construction Materials

An as-built schematic longitudinal section of the valley fill and sediment pond is shown in Figure 4-6. There are essentially five different earth materials involved in the structure.

These materials are as follows:

I. Rock Toe Buttress, consisting of boulders and cobbles ranging from approximately 4 to 1 in mean dimension.

II. Graded Filter, consisting of sand and gravel blended to provide protection against piping of the general embankment fill.

III. Rockfill Drain, consisting of small boulders and cobbles ranging from approximately 2 feet to 6 inches in mean dimension.

IV. General Embankment Fill

V. Foundation Material, consisting of the undisturbed material soil and rock materials comprising the valley floor and walls.

The location and geotechnical strength characteristics of the various materials are presented on Figure 4-6. Particle size gradations of the general embankment fill and the graded filter are shown on Figure 4-7.

4.4.2 Construction Control

The construction of the earthfill and sediment pond was inspected in the field by a qualified geotechnical field engineer. The inspections consisted of determination of material suitability, material placement techniques, and in-place density testing to determine the degree of compaction obtained in the materials. Observation of the sequential phases of the construction are discussed individually below:

1. Rock Toe Buttress

   Prior to placement of the rock toe buttress, all areas to receive fill were stripped of topsoil and loose or surficial materials. The rock toe buttress was then placed by end dumping or moving with dozers in order to insure interlocking and proper resting of the individual boulders.

2. Graded Filter

   Upon completion of the rock toe buttress, the graded filter material was placed. Placement was carried out

in 6 inch to 12 inch lifts, with the material spread by dozers and compacted in place. No compaction tests were carried out in the filter materials due to their granular nature, but careful observations were made throughout the filter placement to insure continuity of material and adequacy of the placement techniques.

3. General Embankment Fill

After the graded filter material was placed, general embankment fill was compacted in lifts to form the valley fill and sediment pond embankment. Compaction testing was carried out as the fill was placed to insure that the material was compacted to at least 95% of Modified Proctor Density, per ASTM D-1557. Results of the compaction testing are presented in summary form on Figure 4-8.

4.4.3 Stability Analyses

Stability analyses of the as-built section have been carried out to insure that the fill has an adequate static factor of safety against failure. The analyses were carried out using the Bishop Method, which essentially assumes a circular failure arc through the mass and computes the stability of forces along that arc. This is then carried out on a variety of failure arcs until the minimum factor of safety is determined. The analysis was carried out using the impact parameters and fill geometry shown on Figure 4-6, and various water seepage paths through the embankment and fill. The results of the analyses are presented in Figures 4-9 through 4-12. Critical surfaces are shown on the figure, with the computed Factor of Safety. As can be seen, the minimum computed static factor of safety for the composite embankment is 1.8; the critical failure arc is located in the lower portion of the embankment. This is within the requirements set forth by OSM.

A cross-section of the valley fill (Figure 4-13) shows the valley walls, the embankment fill, and the emergency spillway for the sediment pond.

4.5 POND MAINTENANCE

Sediment ponds must be periodically maintained to remove deposited sediments so that trap efficiency can be preserved. The Federal Regulation require that this occur when the design sediment storage volume has been 60 percent displaced (see Figure 4-2).

Since the required volume is 3 years of sediment or 0.1 acre-feet per acre of disturbed land, the maintenance schedule should require cleaning the pond at least every 21 months (60 percent of 3 years). It is advisable, however, to reduce this to a maximum of every 12 to 18 months because the sediment will not be deposited evenly over the 3 year period (Skelly and Loy, 1979).

A thorough inspection of the sediment pond and embankment should be undertaken at least once per year. When examining for stability and general inspection the inspector should be looking for any of the following conditions:

- Seepage from anywhere on the down-stream side of the embankment but especially around the discharge pipe
- Erosion of embankment slopes
- Continuity of emergency spillway
- Erosion around entrance or exit of discharge pipe
- Clogged principal or emergency spillway
- Check slope stakes for obvious slope movement

- Level of sediment
- Placement of wave erosion protection
- Erosion at spillway discharges
- Clogging of dewatering device.

Monitoring for embankment movement (Skelly and Loy, 1979) should also be a part of this inspection. This can be performed by setting stakes in the embankment, along the toe and several rows proceeding up from the toe. The original position and elevation should be recorded with reference to a permanent landmark. These positions should be checked during inspection. If unstable or potentialy unstable conditions exist, corrective measures should be taken immediately.

5.0 DIVERSION DITCHES AND CULVERTS

5.1 LOCATION

Figure 3-2 shows the drainage area of concern. In an effort to minimize further land disturbance, it is recommended that existing roads be incorporated into the diversion ditch scheme. This will result in the construction of only one additional ditch that must be constructed on undisturbed land. This ditch is labeled on the plan as I-J.

5.2 ANALYSIS AND DESIGN

Diversion channels were conceptually designed using the Chezy-Manning equation:

$$Q = \frac{1.486\ AR^{2/3}S^{1/2}}{n}$$

where:

- Q = flow, cfs
- n = Mannings Roughness Coefficient
- A = Cross sectional area
- R_h = Hydraulic radius
- S = Slope

Typical values of the rougness coefficient ranges between 0.022 and 0.030 for excavated or dredged earth channels, straight and uniform with short grass and few weeds (Grey, 1973). An average value of 0.027 was used for the calculations.

Channels were designed, where possible, for ease of construction and maintanence. A trapezoidal cross section with sides sloping at 45 degrees and a base width of 1.0 feet was

used. A typical cross section of ditch along side of the existing road and an enlarged cross section through a ditch are shown on Figures 5-1 and 5-2, respectively.

Table 5-1 shows the quantity of flow that will be carried by each ditch and the particular drainage sections that contribute to the flow. Table 5-2 summarizes the calculated depths of flow in each channel. OSM regulation requires a minimum free board of 0.3 feet for diversion ditch design. This criteria would be satisfied if all ditches were constructed to a depth of 1.0 feet.

It is recommended that the channels be seeded to aid in the prevention of erosion during peak flows.

Velocities, calculated from Mannings equation, indicate that they would be in the range of 10-12 feet per second at peak flow. Measures needed to reduce the velocities (cutting new ditches in undisturbed areas with reduced slopes) or to prevent erosion (lining the channels with rocks sufficient to resist erosion) are felt to be too destructive to justify the gains.

OSM regulation require energy dissipators at ditch stream interfaces if velocities of entering ditches are greater than that of the receiving stream. A situation of this nature occurs at only one place in the study area. This is where the outflow of Ditch A-B enters an existing stream. Here we would recommend that a rock check-dam be placed at the interface to be used as an energy dissipater.

5.3 CULVERT DESIGN

Existing culverts were checked to see if they could carry the required 10-year storm. The 24-inch culvert at a road/stream

TABLE 5-1
FLOW RATES TO BE CARRIED BY PROPOSED DITCHES

Ditch	CD	EF	GH	FH	AB	IJ
Drainage Section	I	I	I	I	II	VI
Q (cfs)	12.3	12.3	12.3	12.3	12.6	3.7

TABLE 5-2
SUMMARY OF CALCULATED FLOW DEPTHS
IN PROPOSED DITCHES

Ditch	Cross Section	Depth of Flow (ft.)
CD	Trapezoidal*	0.64
EF		0.64
GF		0.64
FH		0.64
AB		0.70
IJ		0.67

* Base = 1.0 ft, sides slope 45°

intersection southwest of the mine area will receive outflow from sections II and III resulting in a total flow of 25.8 cfs. The 24-inch culvert will carry this flow if the head water elevation is equal to 2.0 times the culvert diameter. Culvert flow quantities were determined using monographs for inlet controlled culverts in the Handbook of Steel Drainages and Highway Construction Products, 1971. The maximum flow through this culvert will depend on the type of entrance inlet of the culvert. Our design was based on an end section conforming to the fill. A cross section of this inlet is shown in Figure 5-3.

The other culvert which carries flow underneath the mine area is 42 inches in diameter. Outflow from sections II, III, IV and V would be carried by this culvert. Computations by others indicate that the culvert has a total capacity of approximately 52 cfs, and that the maximum flow as a result of a 100-year, 24-hour storm and 100 year, 6-hour storm are approximately 17 cfs and 19 cfs, respectively.

Respectfully submitted,

GOLDER ASSOCIATES

CWLockhart

Charles W. Lockhart

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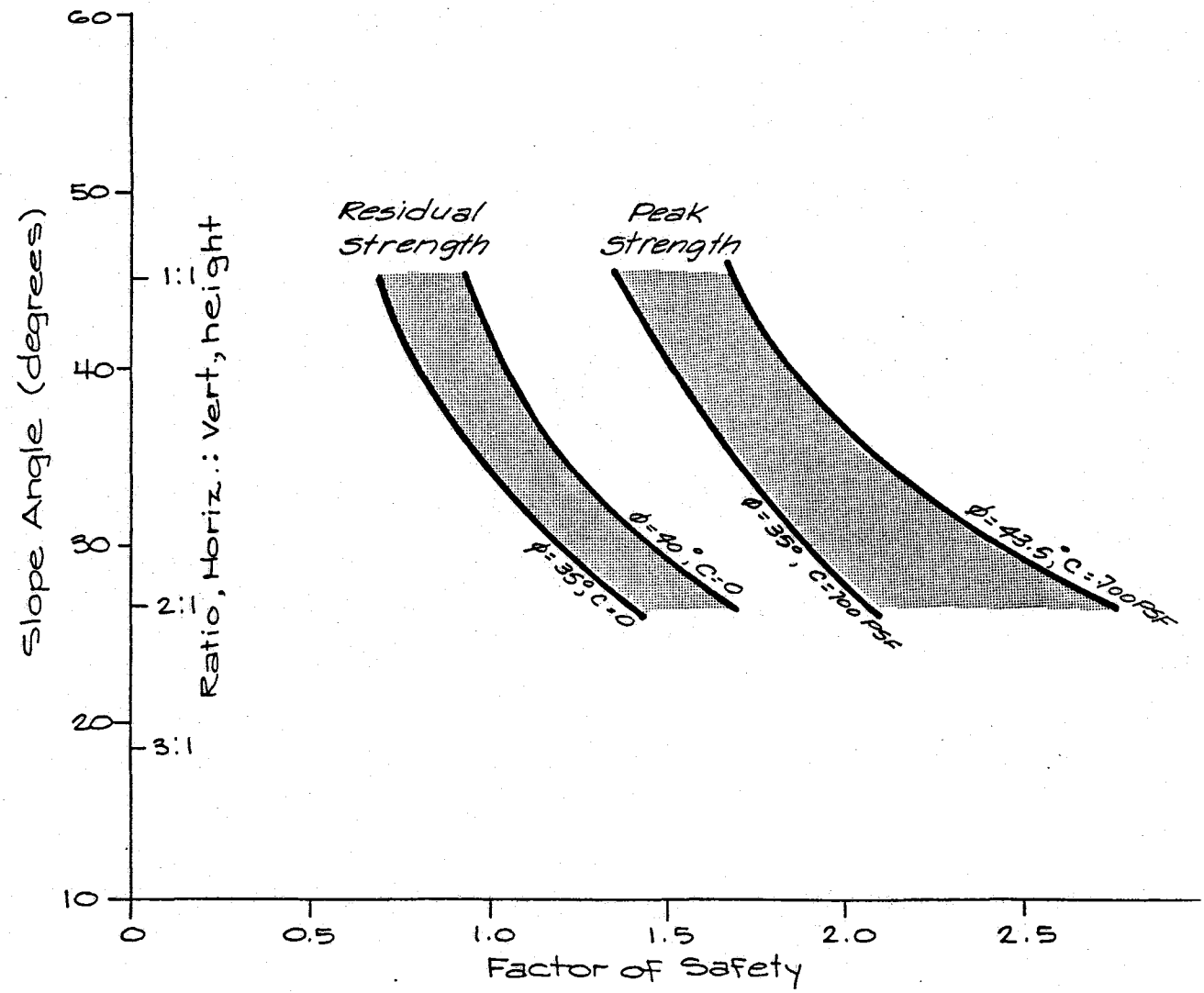
INFLUENCES OF SLOPE ANGLE AND STRENGTH PARAMETERS ON THE FACTOR OF SAFETY

Figure 1



Project No. Reviewed Date

FIGURE 2-1

GENERAL SITE LOCATION
BELINA MINE AREA

This figure is not reporduced herein because of it's large size and quality of the original. For further information see original copy on file at Valley Camp of Utah, Inc.

FIGURE 2-2

SURFACE FACILITIES PLAN
BELINA MINE AREA

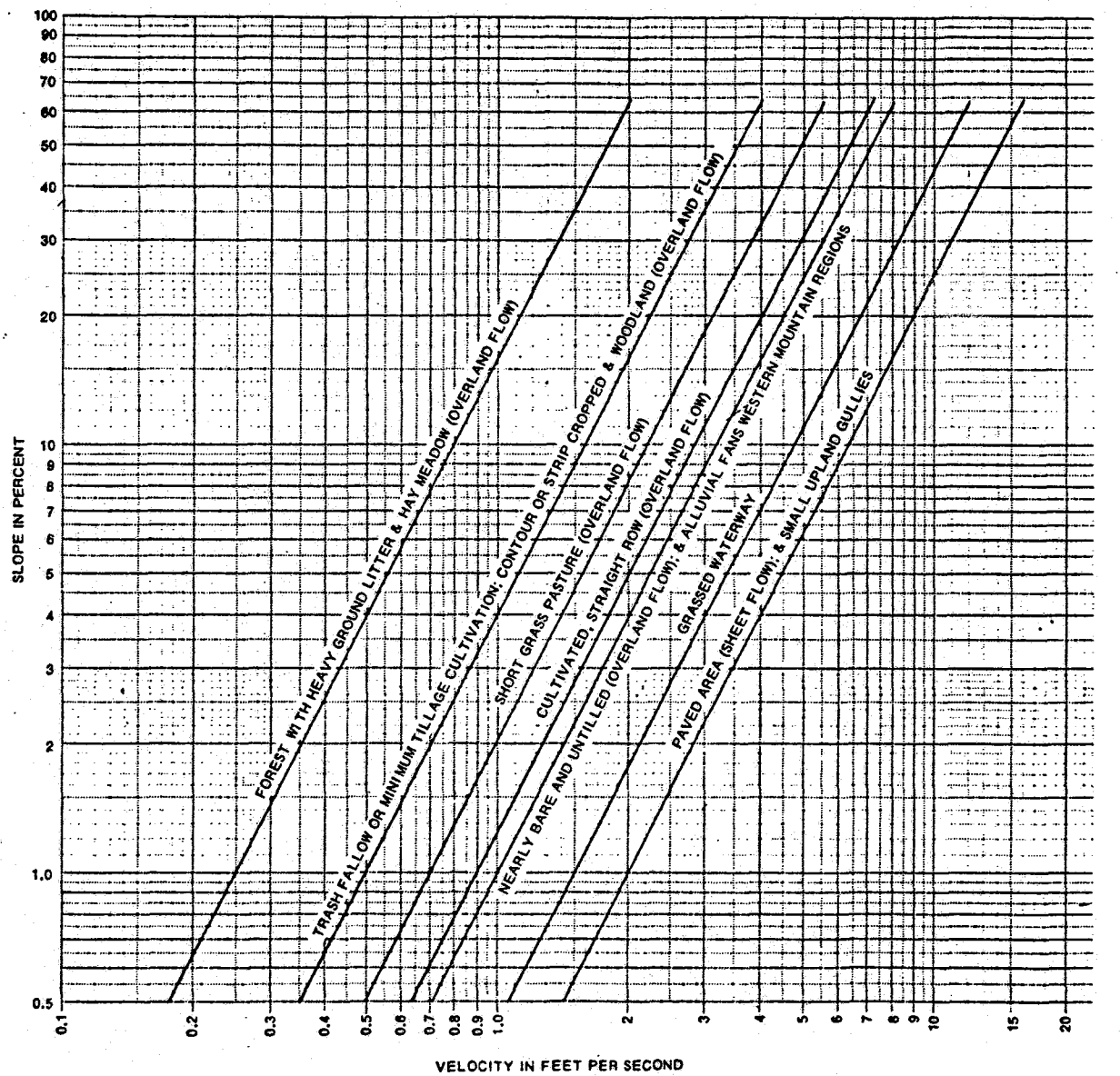
This figure is not reporduced herein because of it's large size and quality of the original. For further information see original copy on file at Valley Camp of Utah, Inc.

FIGURE 3-2

DRAINAGE AREAS
BELINA MINE AREA

This figure is not reporduced herein because of it's large size and quality of the original. For further information see original copy on file at Valley Camp of Utah, Inc.

VELOCITIES FOR UPLAND METHOD OF ESTIMATING T_c Figure 3-1

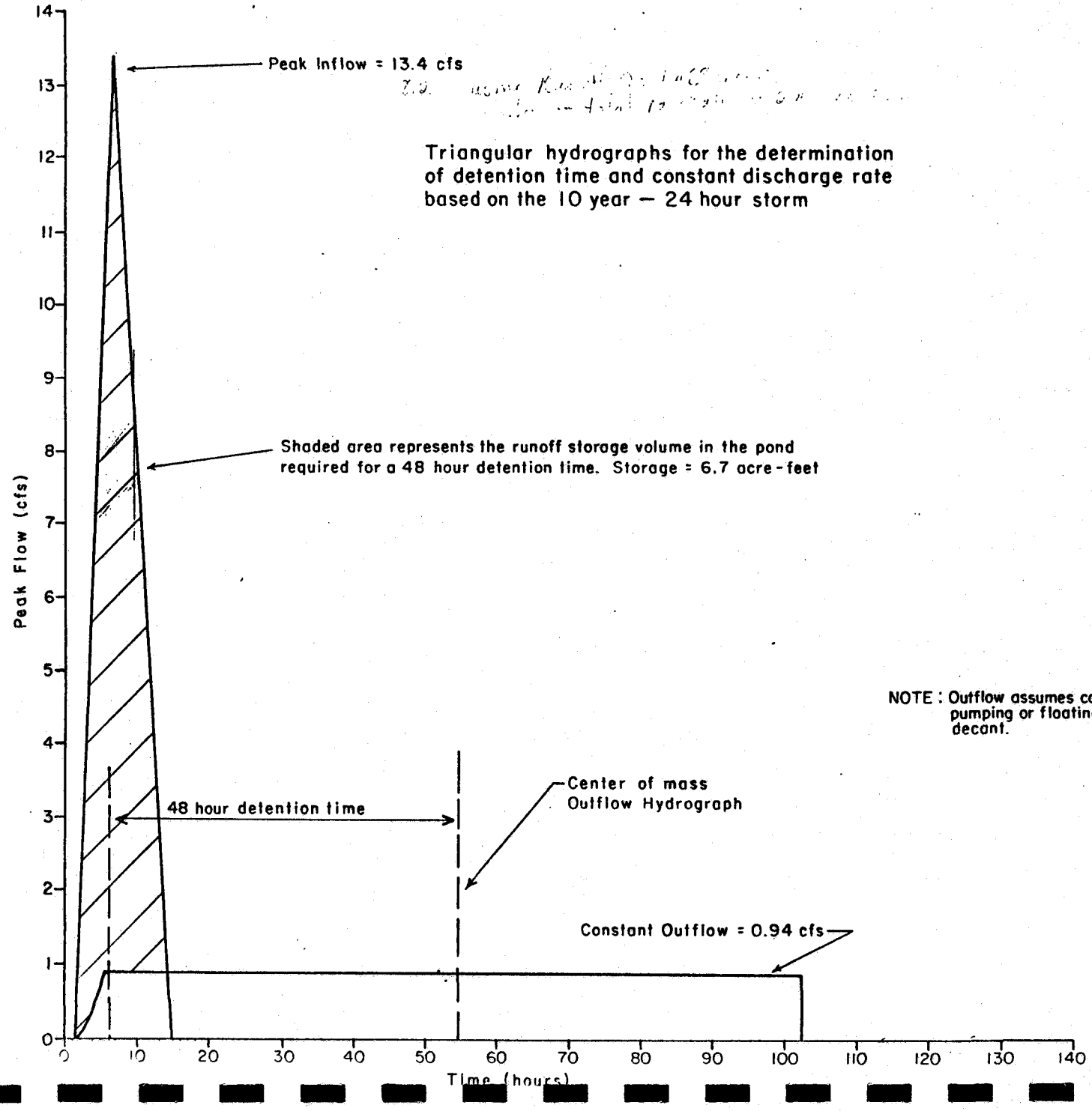


after Kent, 1972

Project No. 783-1184 Reviewed CML Date 4-80

Project No. 792-1156A Reviewed RC Date 5/80

Golder Associates



Triangular hydrographs for the determination of detention time and constant discharge rate based on the 10 year — 24 hour storm

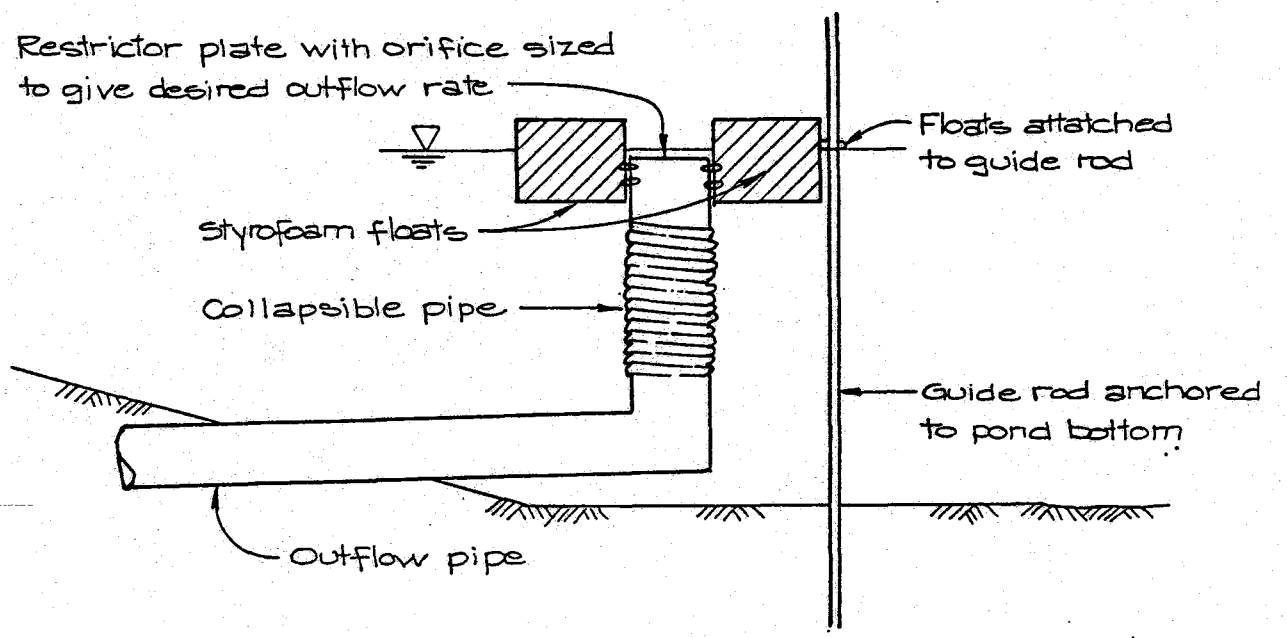
NOTE : Outflow assumes continuous pumping or floating orifice decant.

INFLOW AND OUTFLOW HYDROGRAPHS OF SEDIMENT POND

Figure 4-1

FLOATING ORIFICE TYPE DECANT

Figure 4-3

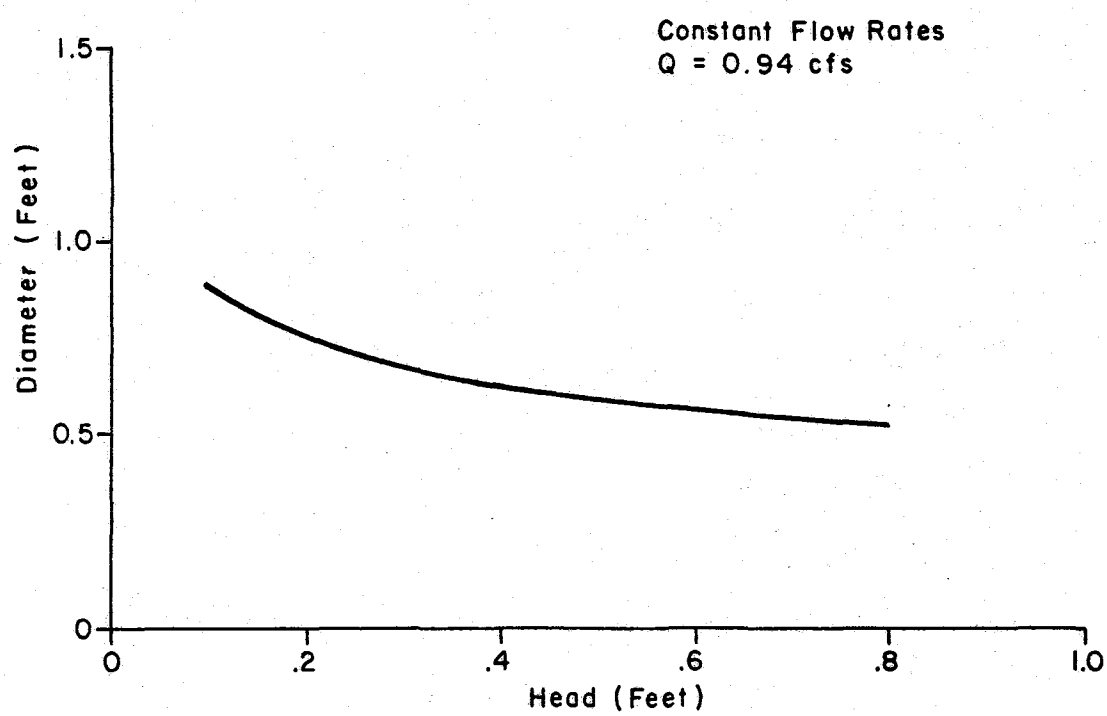


NOTES

1. Collapsible pipe must extend from maximum pool level to permanent pool level.
2. Inlet is attatched to floats so that it is submerged to give desired outflow rate.

Project No. 773-1122 Reviewed [illegible] Date 5/[illegible]

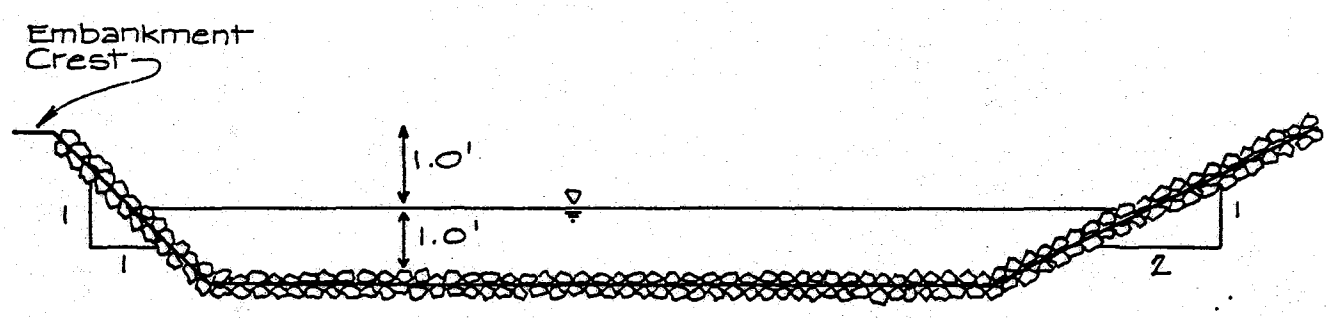
PIPE DIAMETER Vs. HEAD FOR CONSTANT FLOW RATE Figure 4-4



Project No. 7[illegible] Reviewed: [illegible] Date 5[illegible]

CROSS SECTION EMERGENCY SPILLWAY

Figure 4-5



$n = 0.040$
$S = 0.05$
$R = 0.84$ Ft.
$A = 11.50$ Ft.^{2}
$P = 13.65$ Ft.

$$Q_{max.} = \frac{1.486}{n} R^{0.67} S^{0.5} A$$

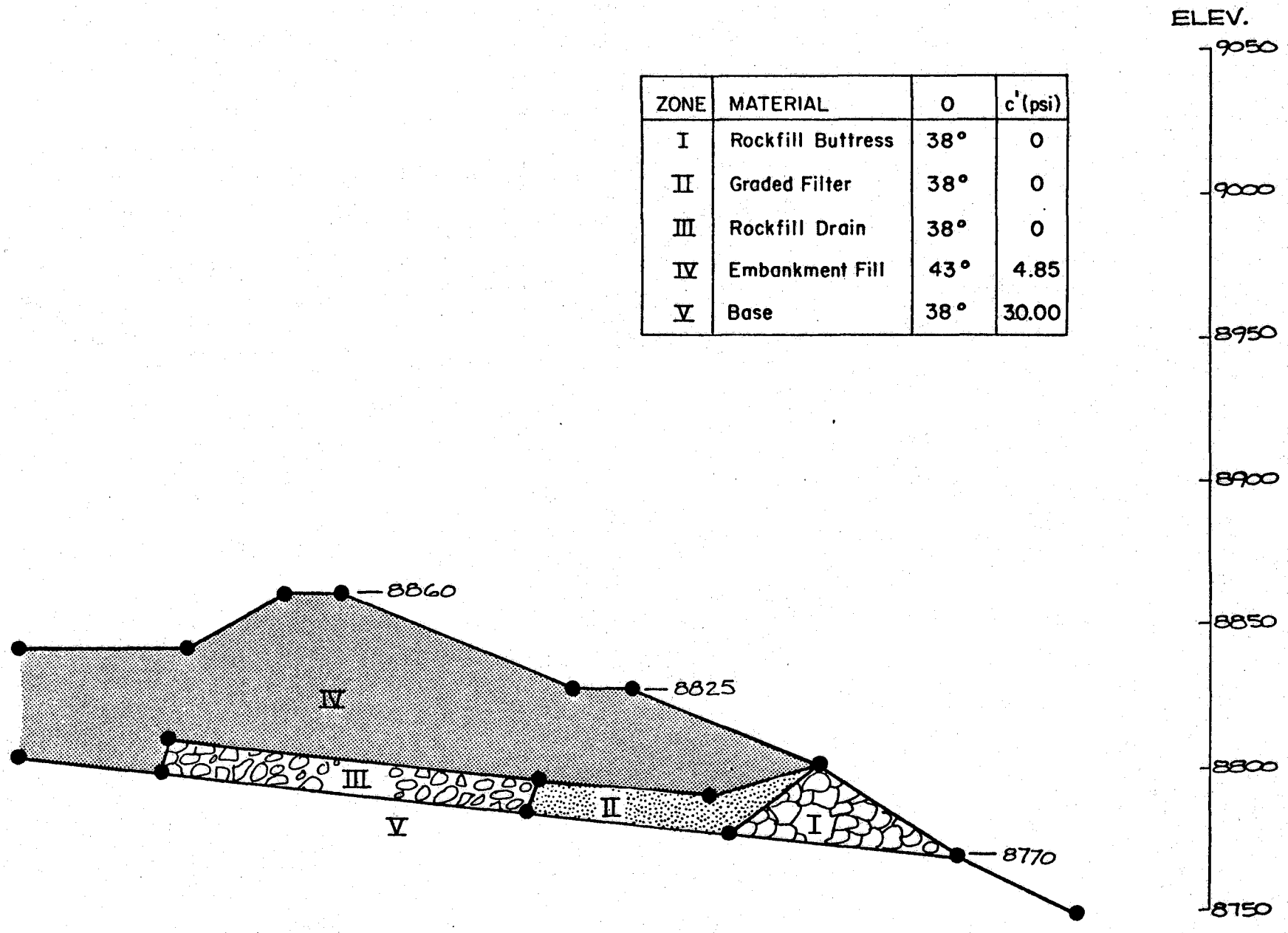
$Q_{max} = 85.0$ cfs

Scale: 1" = 2.5'

Project No. ___ Reviewed ___ Date ___

Project No. 793-1156A Reviewed CWL Date 2-80

ZONE	MATERIAL	ϕ	c' (psi)
I	Rockfill Buttress	38°	0
II	Graded Filter	38°	0
III	Rockfill Drain	38°	0
IV	Embankment Fill	43°	4.85
V	Base	38°	30.00



LONGITUDINAL GEOMETRY FOR VALLEY FILL

Figure 4-6

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Project No. 783-1156a
Project Valley Camp
Golder Associates
Date 6-1980
By HC

GRAIN SIZE DISTRIBUTION

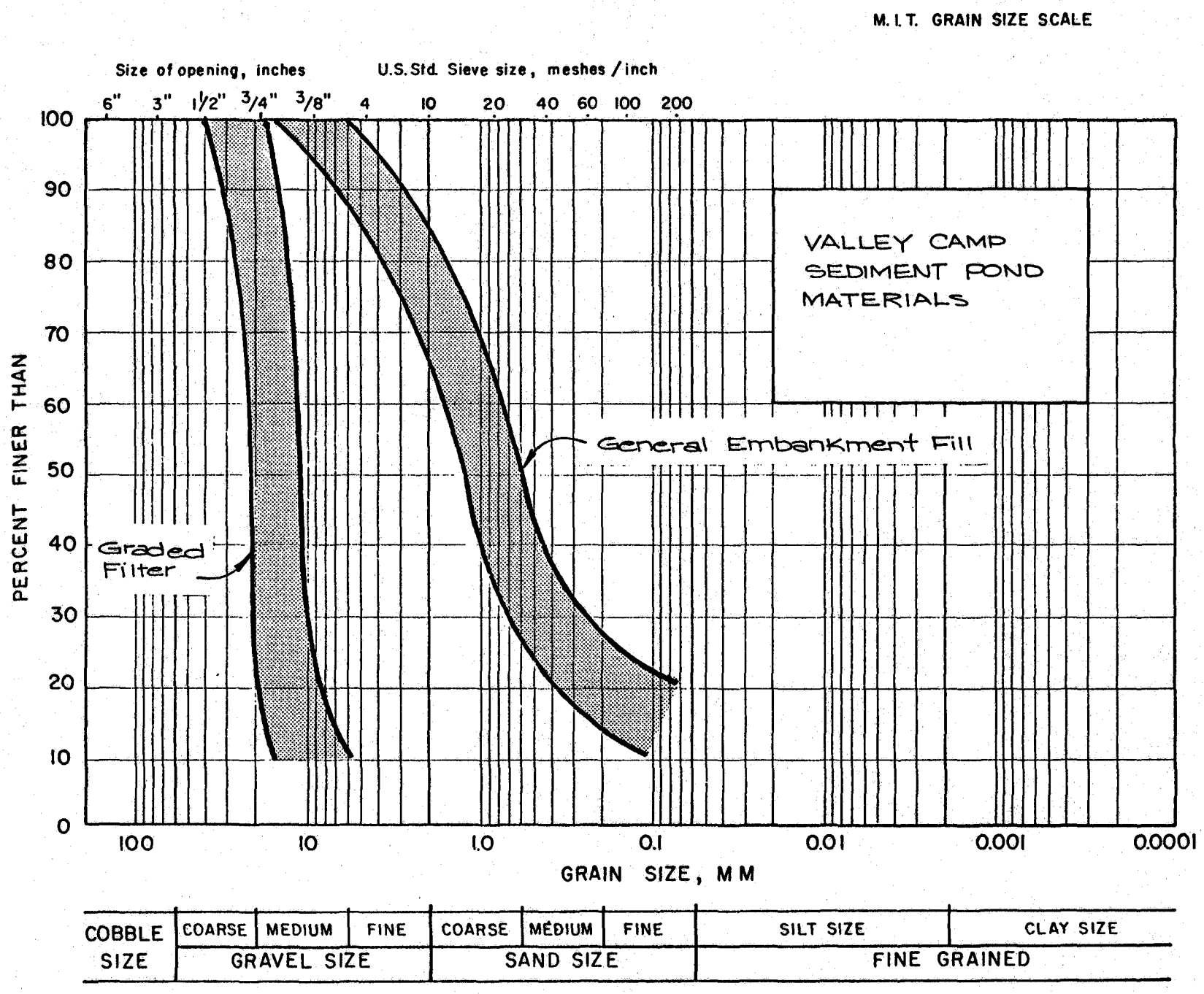
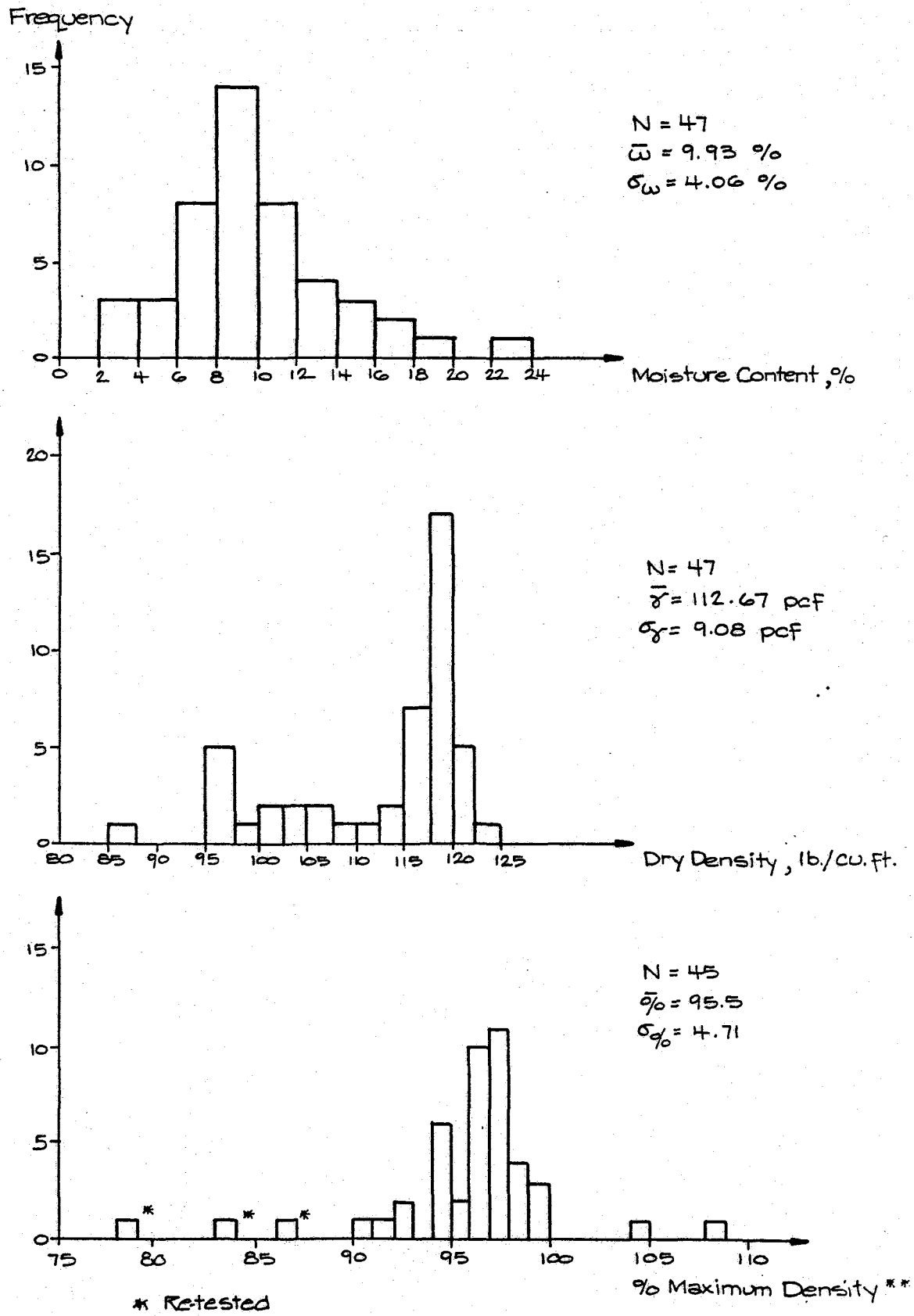


FIGURE 4-7

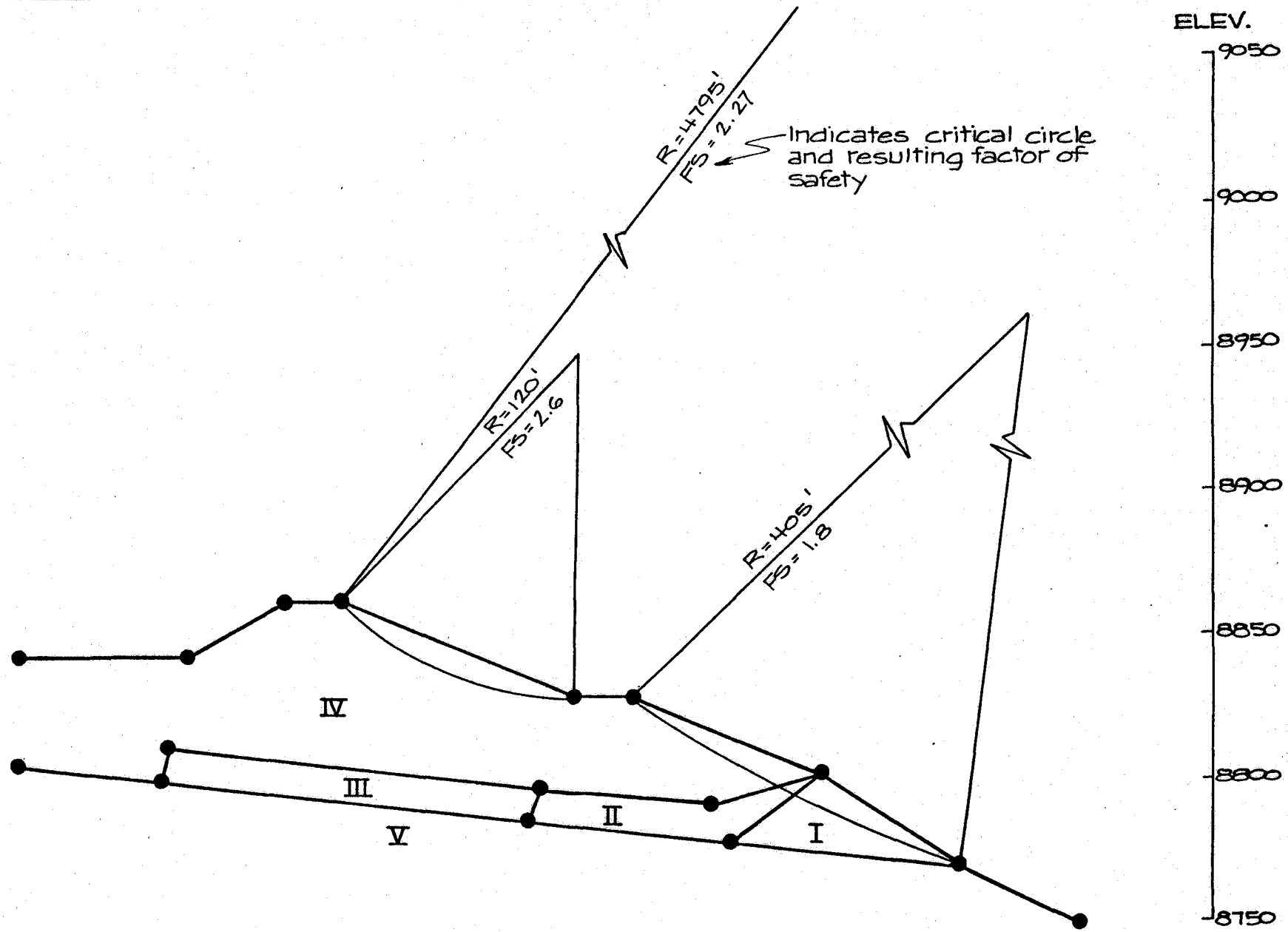
COMPACTION TEST RESULTS

Figure 4-8



Project No. 773-1156A Reviewed CMC Date 2-80

Project No. 793-1156A Reviewed CWL Date 2-80

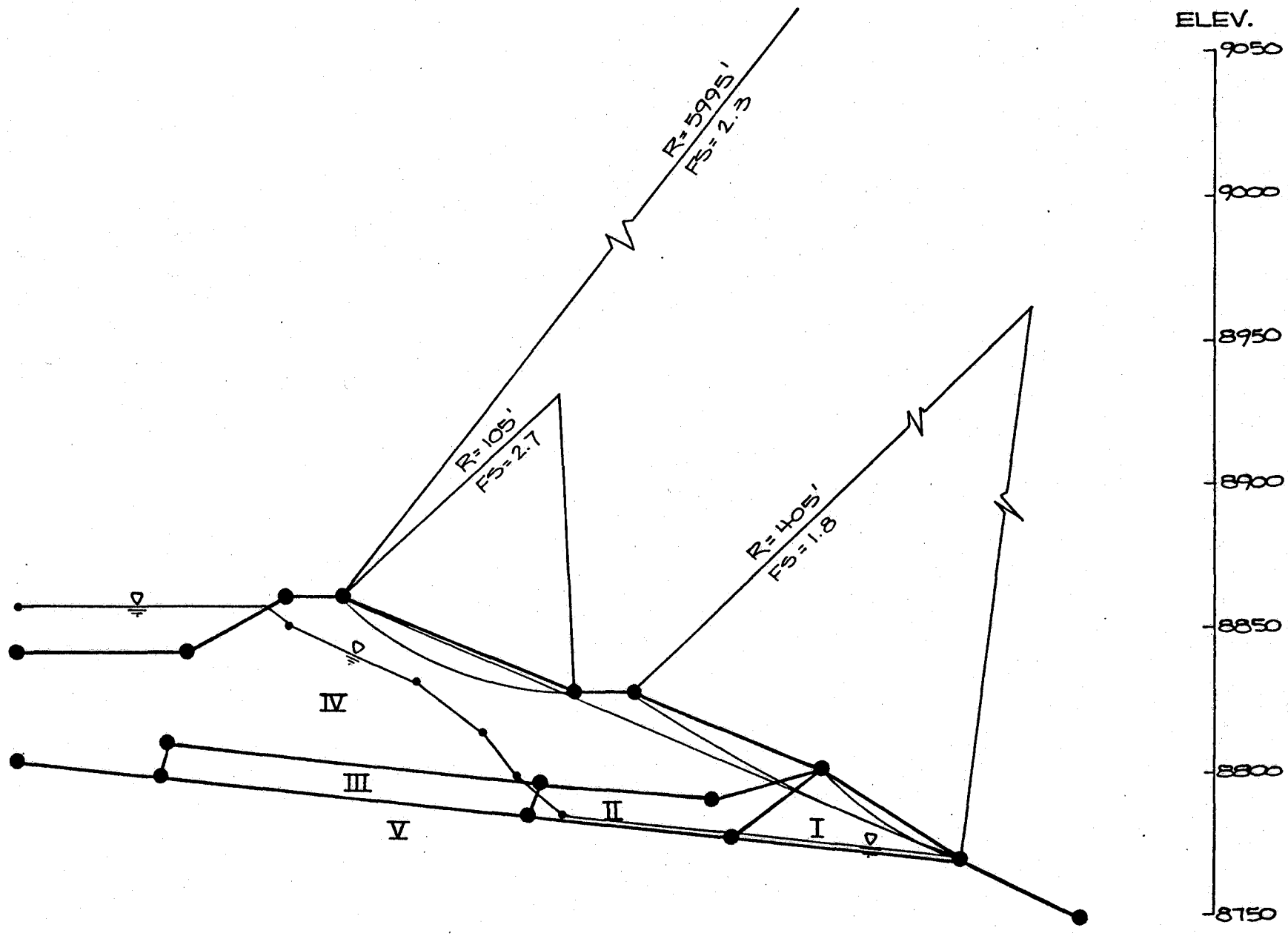


EMBANKMENT STABILITY ANALYSES
DRAINED

Figure 4-9

Golder Associates

Project No. 793-1156A Reviewed CWL Date 2-80



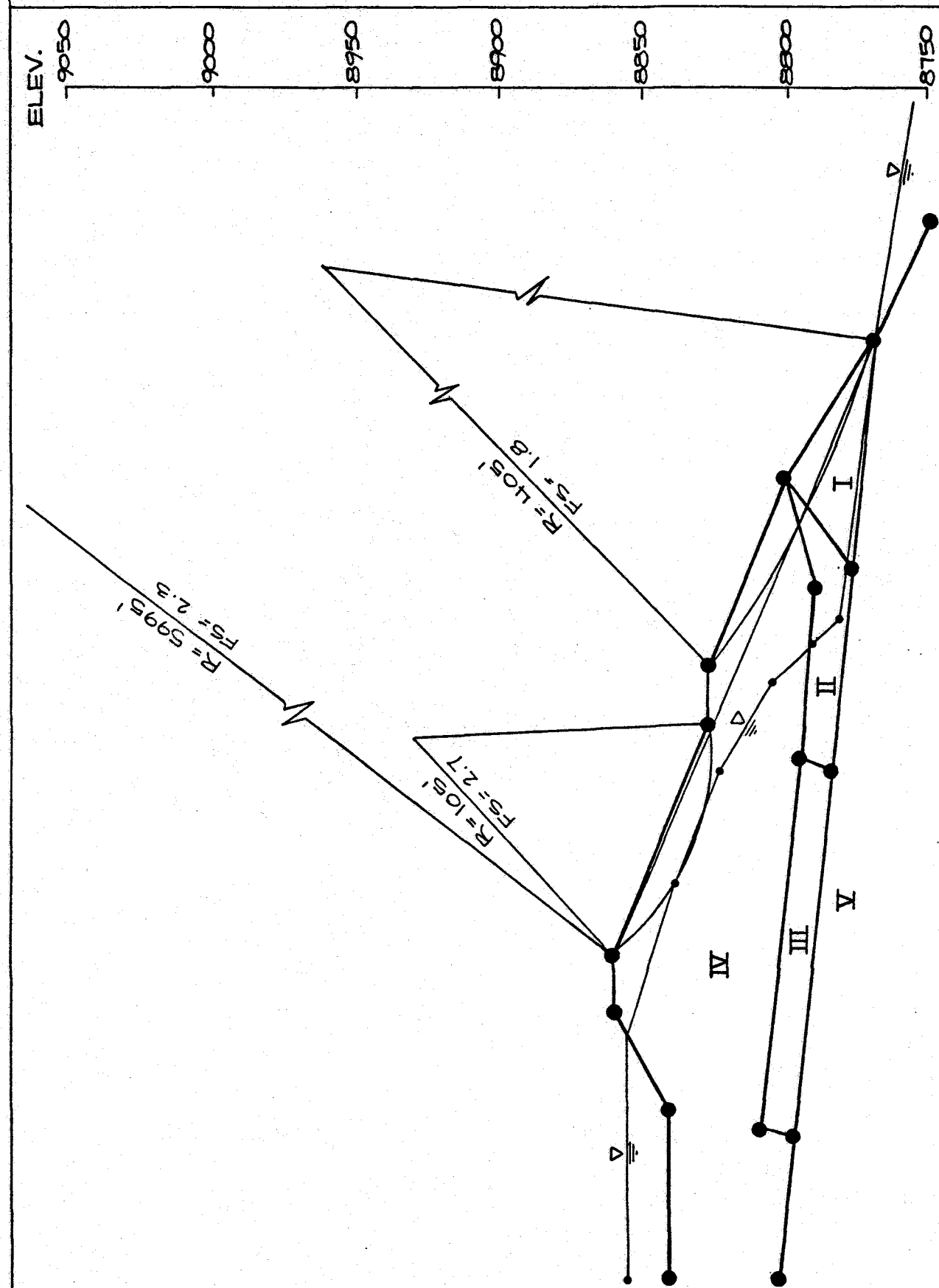
EMBANKMENT STABILITY ANALYSES
STEADY STATE — FULL DRAINAGE INTO II

Figure 4-10

Golder Associates

EMBANKMENT STABILITY ANALYSES
STEADY STATE — II PARTIALLY CLOGGED

Figure 4-11

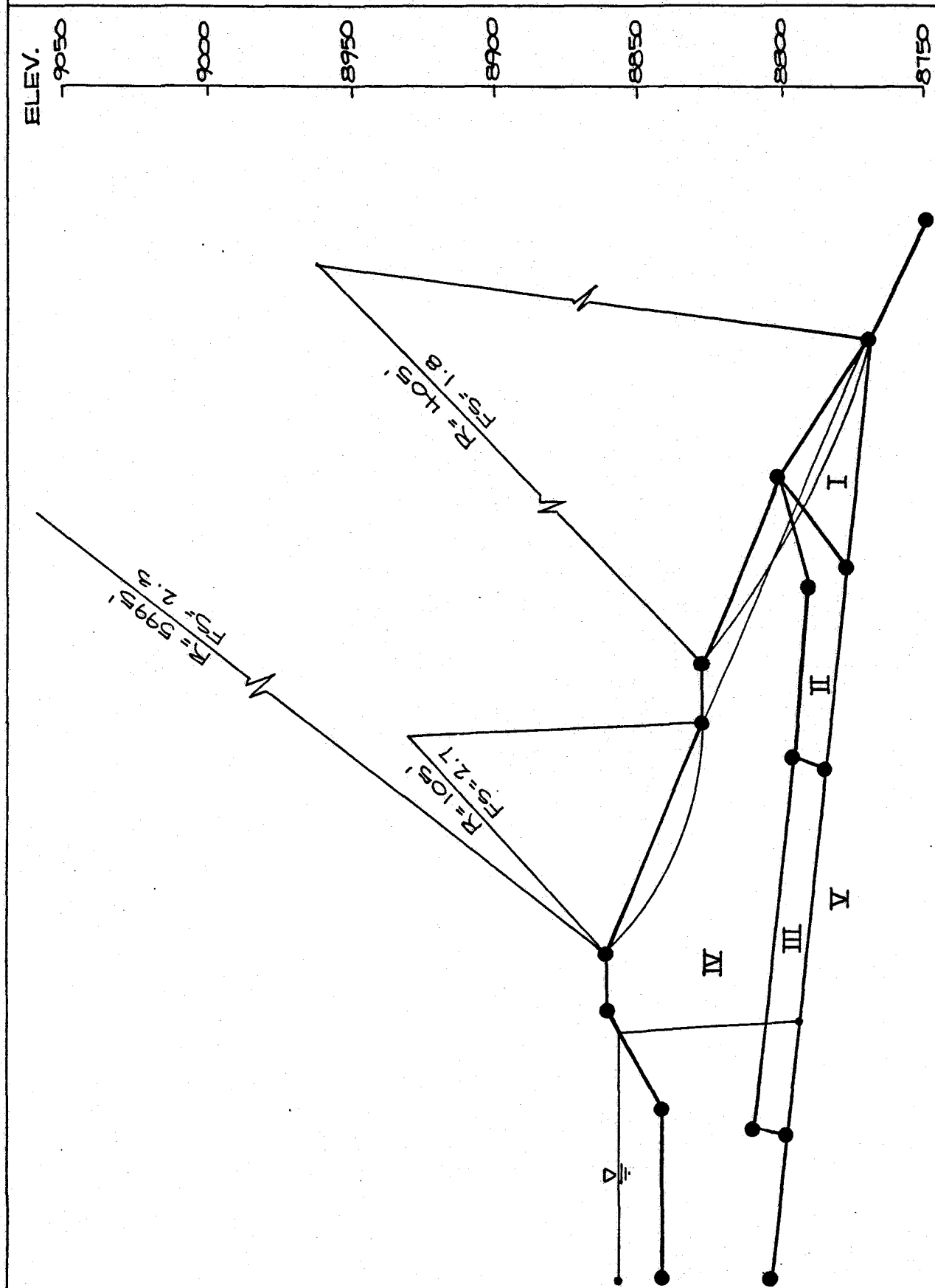


Project No. 783-1156A Reviewed CML Date 2-80

Golder Associates

EMBANKMENT STABILITY ANALYSES
STEADY STATE — FULL DRAINAGE INTO III

Figure 4-12

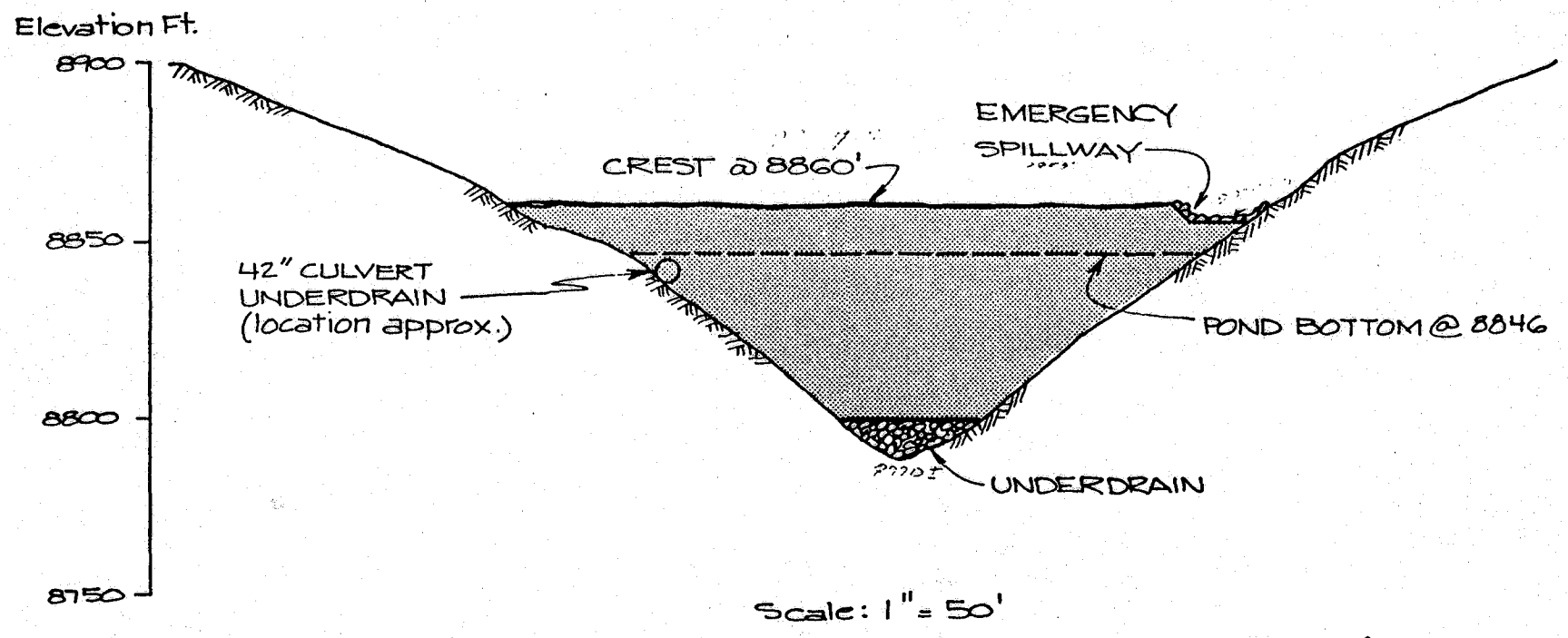


Project No. 783-1156A Reviewed CWL Date 2-80

Golder Associates

Project No. 73-1156A Reviewed CWL Date 6-80

VALLEY FILL CROSS-SECTION



Scale: 1" = 50'

TOP = 8874.93

5' loop of pipe

NOTE: Data from field survey undertaken by Valley Camp

Figure 4-13

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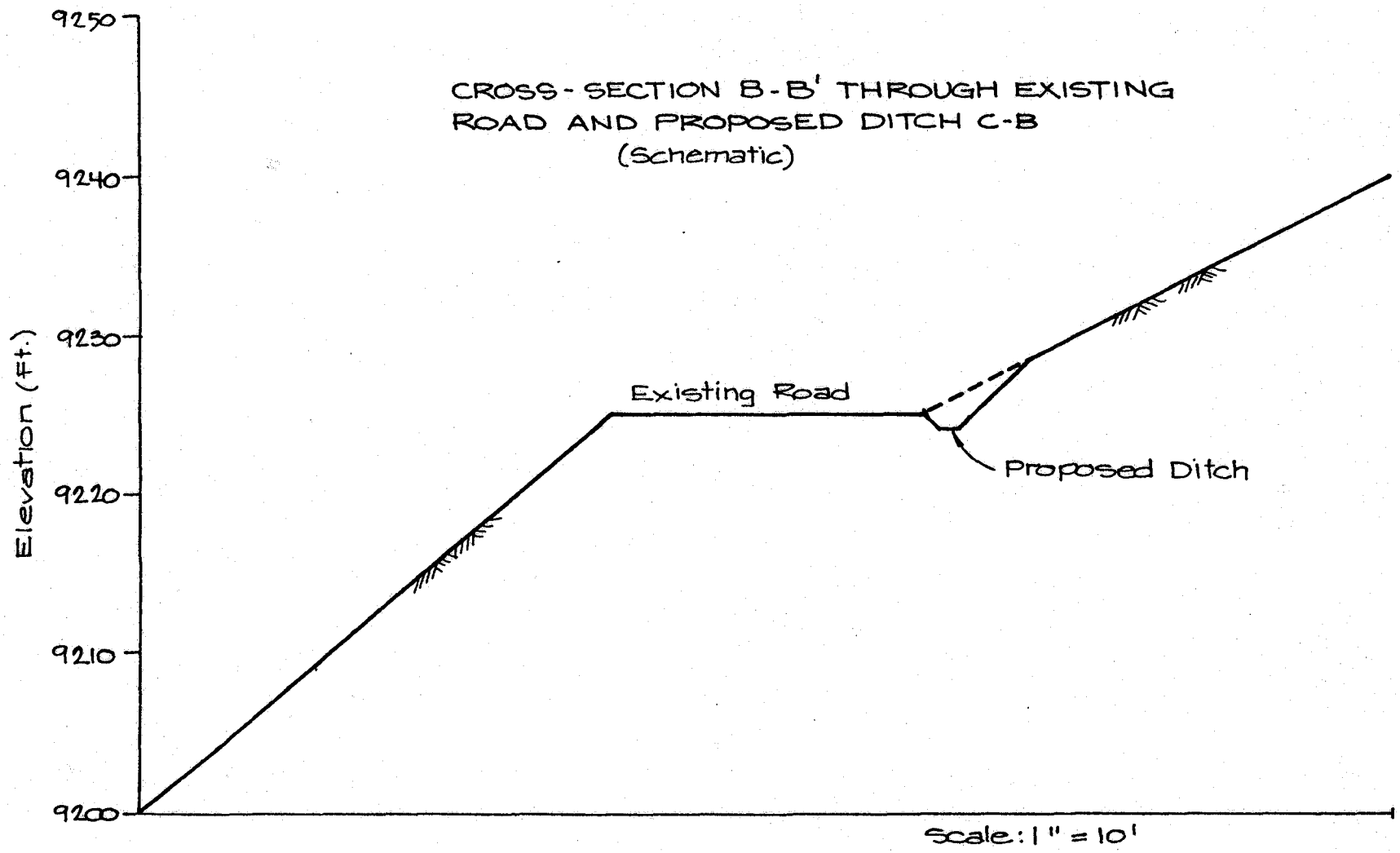
Project No 783-1561 Reviewed CWL Date 4-80



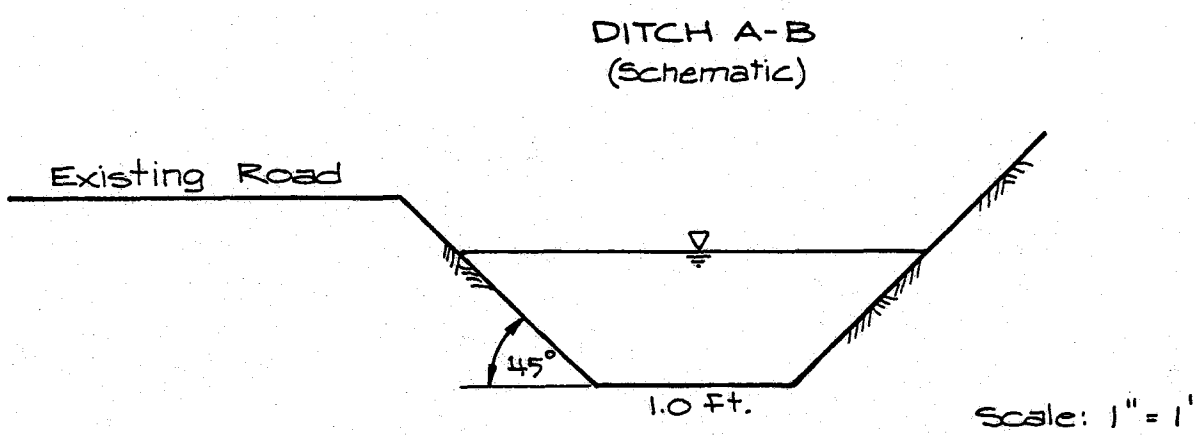
TYPICAL CROSS-SECTION THROUGH
EXISTING ROAD AND PROPOSED DITCH C-B

Figure 5-1

Golder Associates

CROSS-SECTION THROUGH DITCH AND TYPICAL DESIGN CONFIGURATIONS

Figure 5-2



$$Q = \frac{1.49}{n} R_h^{2/3} s^{1/2}$$

$n = 0.027$
$s = 0.089$
$Q = 12.6$ cfs

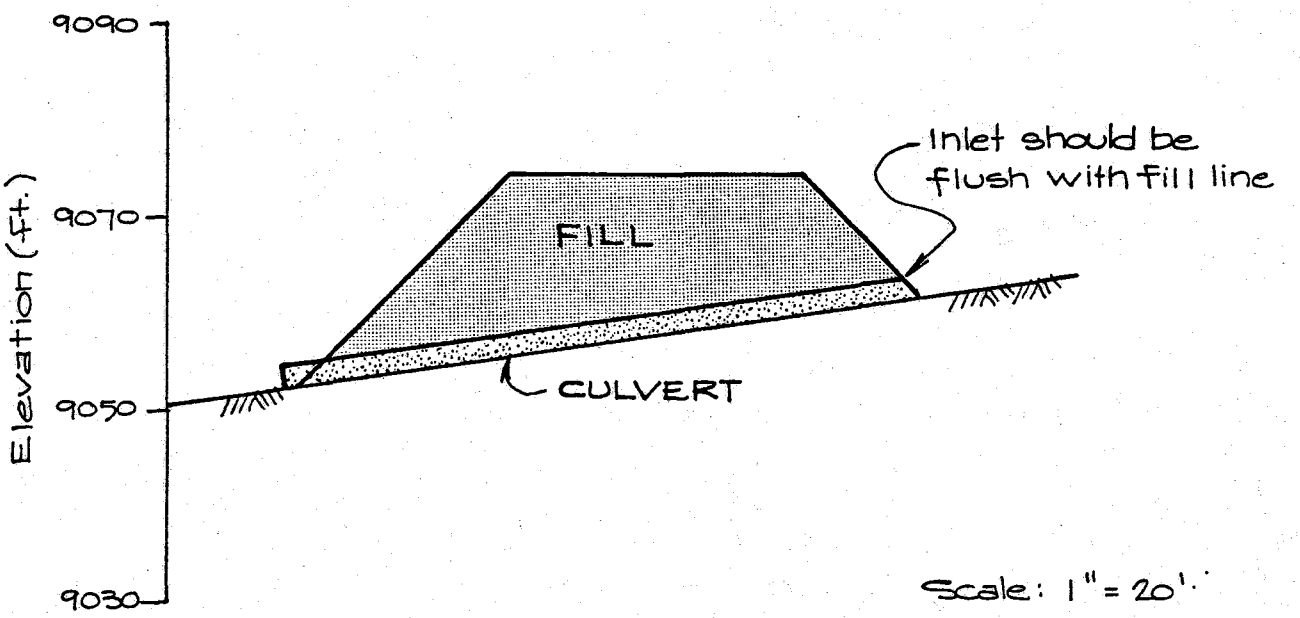
Depth of flow y was solved for by trial and error.

$\therefore y = 0.70$ ft.

Project No. 783-1164 Reviewed OLL Date 8-80

INLET CONFIGURATION FOR CULVERT

Figure 5-3



Project No. 783-1562 Reviewed CHL Date 4-80

APPENDIX R614-301-742.310

Hydrologic Calculations

HANSEN ALLEN & LUCE inc

POND ENERGY DISSIPATION - CHECK OF RIPRAP SIZES IN PLACE

FLOW DATA - 25 Yr - 24 Hr Event

Pond	Q(cfs)	Outfall Dia. (in) PRIMARY	EMERGENCY
001A	18.3	18	18
002A	13.7	18	18
003A	11.0	18	18
004A	52.0	18	N/A

POND 001A

Q=18.3cfs Discharge Elev = 7819.35';

	Head	Q
Primary Spillway	3.25	12.33
Emergency Spillway	0.55	5.93
		18.3 cfs

$\eta = 0.024$ for Corrugated Metal Pipe $Q = \frac{1.49}{\eta} A R^{2/3} S^{1/2}$

Check velocity ranges for discharge pipes

Principal: 1% Slope

$\frac{Q\eta}{1.49 S^{1/2}} = 1.986$

y	$AR^{2/3}$
1.5	0.220
1.0	0.720
1.3	0.9600
1.4	0.988

PIPE FLOW IS FULL UNDER HEAD

$V = Q/A$ $A = \pi D^2/4 = 1.767 ft^2$

$V = 7.0 fps$

from: "Erosion and Sediment Control - Surface Mining in the Eastern U.S." - EPA - Oct 1976 - pp 34-40

Requirements for Riprap Apron (for Check of Riprap Sze)

- Grade = 0%
- Side Slopes (m) = 2:1 or flatter

d_{50} = Median Rock Size
L_a = Apron Length

from Figure 1-15,

$Q = 12.33$ cfs gives $d_{50} \sim 0.4$ ft, $L_a = 12$ ft - Primary Spillway

$Q = 5.9$ cfs gives $d_{50} \sim 0.2$ ft, $L_a \sim 5$ ft - Emergency Spillway

Existing Rock estimated to have existing riprap size approximately 4-6" diameter. Rock heavily sedimented in, and no erosion is present. Current installation appears to be close to design requirements for riprap size.

$$d_{max} = 1.5\, d_{50} = 1.5(0.4\text{ft}) = 0.6\text{ft}$$

Recommend holding to existing structure with replacement if erosion is noted. This is not believed likely due to occurance of large runoff events in 1983 and 1984 without pond discharges. Ponds are believed to be overdesigned due to new precipitation return period changes made in the regulations.

POND 002A

$Q = 13.7$ cfs

Spillway	Head (ft)	Q (cfs)	d_{50} (ft)	d_{max} ($1.5 d_{50}$)	L_a (ft)
Primary	1.3	9.7	0.3	0.45	10
Emergency	0.4	4.0	0.1	0.15	2
		13.7 cfs			

Assumed Riprap Size of 0.5' existing is adequate - OK

POND 003A

$Q = 11.0$ cfs

Spillway	Head (ft)	Q (cfs)	d_{50} (ft)	d_{max} ($1.5 d_{50}$)	L_a (ft)
Primary	1.67	11.0	0.3	0.45	10
Emergency	—	—			

Assumed Riprap = 0.25' - OK - Replace if erosion Noted.

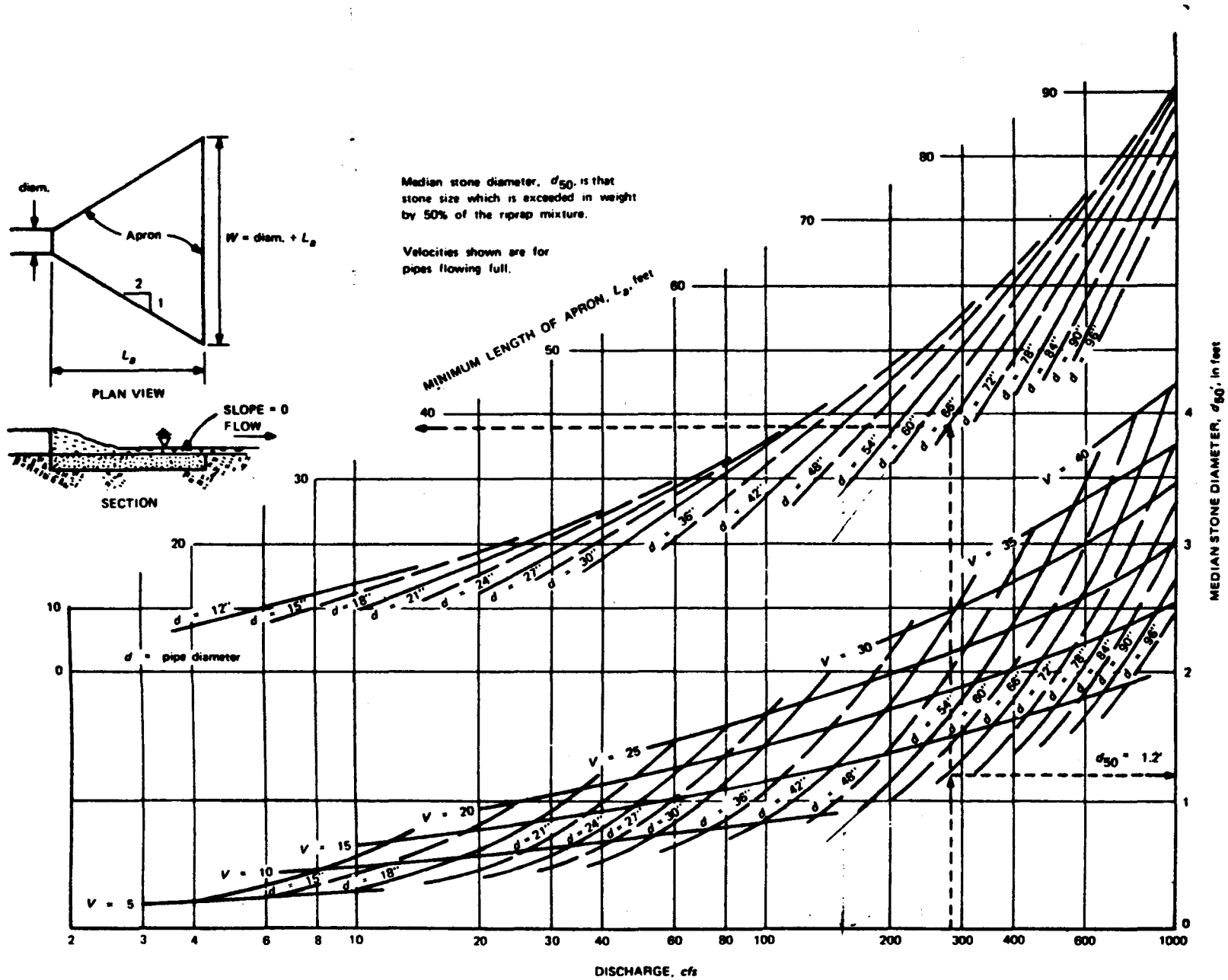


Figure I-15. Design of outlet protection – minimum tailwater condition ($T_W < 0.5$ diam.)[1]

3/5

Pond 005A

Design Q = 250 gpm = 0.56 cfs flow rate is below charts

Velocity is small

discharge pipe slope ~ 10'/60' = 0.17
$\eta = 0.024$
$d = 12''$

$\frac{Qn}{1.49 S^{1/2}} = 0.022$, $y = 0.18'$, $V = 5.83$ fps

USE OSM Steep Channel design Methodology. Fig 5.3.

Triangular Channel, $m = 2$

with low flow, $d_{50} \sim 0.5'$ max

$d_{max} \sim 0.75'$

$d_{10-20} \approx 0.25'$

Install Riprap if erosion is noted on site.

Since discharge is constant, erosion would be present (and growing) if it were a problem.

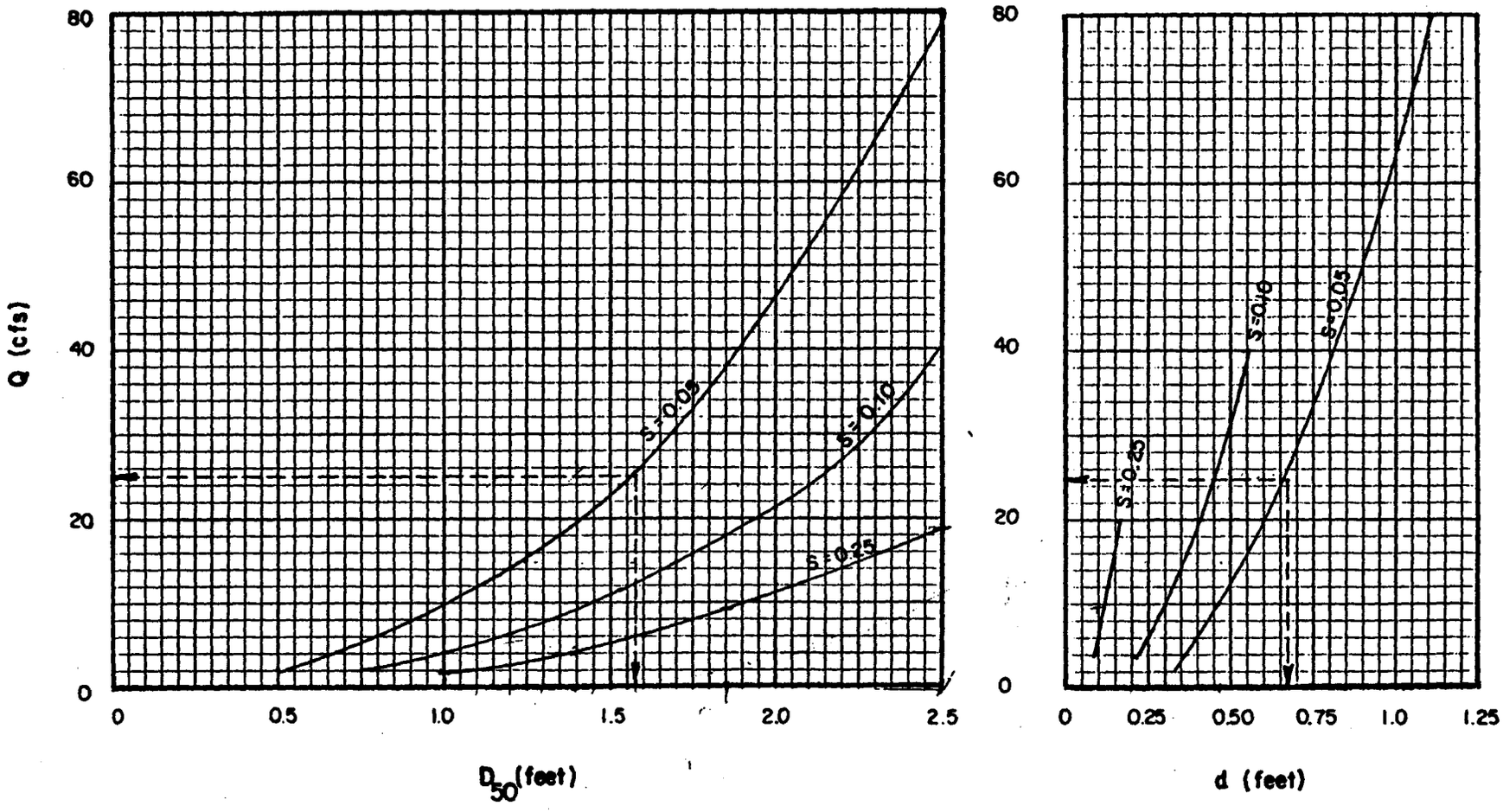


Figure 5.3. Steep slope riprap design, triangular channels, 2:1 sideslopes.

HANSEN ALLEN & LUCE INC

CLIENT: VALLEY CAMP
PROJECT:
FEATURE:
PROJECT NO.: 007.11.200

SHEET 2 OF 2
COMPUTED: DEH
CHECKED:
DATE: 2 NOV 89

RUNOFF VOLUME INCLUDING TRIBUTARY AREAS

ASCA#	PAVED	DISTURBED	SAGE/GRASS	FOREST/ASPEN	TOTAL
1	—	1.13	—	—	1.13
2	0.60	1.00	—	—	1.60
3	0.75	0.87	—	0.50	2.12
4	—	0.19	—	0.37	0.56
5	—	1.16	—	—	1.16
6	—	0.95	1.00	1.34	3.29

ASCA#	PAVED		DISTURBED		SAGE/GRASS		FOREST/ASPEN		TOTAL RUNOFF VOLUME
	Q (in)	Vol (ac-ft)	Q (in)	Vol (ac-ft)	Q (in)	Vol (ac-ft)	Q (in)	Vol (ac-ft)	ac-ft
1	2.22	—	1.49	0.14	0.62	—	0.43	—	0.14
2	"	0.11	"	0.12	"	—	"	—	0.23
3	"	0.14	"	0.11	"	—	"	0.02	0.27
4	"	—	"	0.02	"	—	"	0.01	0.03
5	"	—	"	0.14	"	—	"	—	0.14
6	"	—	"	0.12	"	0.05	"	0.05	0.22

PERCENT AREA AS ASCA = $\frac{6.66\ ac}{62.83\ ac} * 100 = 10.6\%$

TOTAL POND RUNOFF VOLUME

POND #	10-YR, 24-HR VOLUME (ac-ft)
001A	1.18
002A	0.94
003A	0.67
004A	3.15
TOTAL	5.94

ASCA Runoff Vol. as a % of Total Pond Runoff Vol = $\frac{0.83}{5.94} * 100 = 14.0\%$

PROJECT : Valley Camp Coal - C-26A-24 Runoff Calculation

AREA= 9.2 ACRES
AVERAGE BASIN SLOPE= 52.5 PERCENT
CURVE NUMBER= 75.0
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 1780. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0898 HOURS QPCFS= 77.52 CFS QPIN= 8.3562 INCHES
C3= 41.1878 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.26	.6567	.0000	.0000	.0	.00
2.28	.6773	.0000	.0000	3.9	.00
2.30	.6979	.0003	.0003	24.1	.00
2.32	.7184	.0008	.0005	51.5	.00
2.33	.7390	.0015	.0007	71.2	.03
2.35	.7596	.0025	.0010	77.5	.07
2.37	.7802	.0037	.0012	72.6	.12
2.39	.8008	.0052	.0014	61.3	.20
2.41	.8214	.0069	.0017	47.9	.29
2.42	.8420	.0088	.0019	35.4	.38
2.44	.8626	.0109	.0021	24.9	.49
2.46	.8832	.0132	.0023	16.9	.60
2.48	.9037	.0157	.0025	11.2	.71
2.50	.9243	.0185	.0027	7.2	.82
2.51	.9340	.0199	.0014	4.5	.93
2.53	.9396	.0207	.0008	2.8	1.00
2.55	.9452	.0215	.0008	1.7	1.01
2.57	.9507	.0223	.0008	1.0	.96
2.58	.9563	.0232	.0008	.6	.88
2.60	.9619	.0240	.0009	.3	.78
2.62	.9674	.0249	.0009	.2	.69
2.64	.9730	.0258	.0009	.1	.62
2.66	.9785	.0267	.0009	.0	.56

HYDROGRAPH PEAK= 1.01 cfs
TIME TO PEAK= 2.55 Hours
RUNOFF VOLUME= .14 Acre-Feet

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE C-27-24
PROJECT NO. 007.11.100

SHEET 1 OF
COMPUTED DEH
CHECKED
DATE 10/30/89

C-27-24

Tributary Area: Ditch UDD-2 $Q = 0.57$ cfs (See HA&L, 1989 Sed. Pond Report)

Check Pipe Flow

- 24" CMP
- $n = 0.024$
- $S = 43\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 80.6$ cfs ok

Pipe flow not limiting

Check Inlet Control

- HW depth = 4.5'

$\frac{HW}{D} = 2.25$ Use Chart 5

Assume Projecting entrance

$Q_{HW} = 24$ cfs ok

Inlet Control not limiting

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.014$, $Y = 0.12'$, $V = 7.4$ fps > 5.0

Since $Q < 2$ cfs, data is off EPA Fig 1-15, Use d_{50} min = 1.0'
$L_{min} = 10'$

(If severe erosion exists - High flow events of 1983-84 exceeded 10yr event)

HANSEN ALLEN & LUCE inc

CLIENT Valley Camp
PROJECT Permit
FEATURE
PROJECT NO. 007.16.100

SHEET 1 OF
COMPUTED DEH
CHECKED
DATE 6/30/89

C-28-24

Tributary Area: UDD-2 14.7ac ← from 1989 HA&L Report on Sed. Ponds
Tributary Area 20.3ac
35.0 ac

CN	A	Q
75	33.6	0.19
70	1.4	0.10
	35.0	

$\bar{Q} = 0.18$ $\bar{CN} = \underline{74.8}$

Slope from Tributary Area = $\frac{6800(80)100}{43560(20.3)} = 61.5\%$

Avg. Slope = $\frac{61.5(20.3)+37(14.7)}{35} = \underline{51.2\%}$

Precipitation = 1.55 inches (10yr, 6hr)

Hydraulic Length = 2350 feet

Calculated Runoff $Q_{TOT} = \underline{3.3\ cfs}$ WSGM: 3.6 cfs

Check Pipe Flow

- 24" CMP
- $n = 0.024$
- $S = 30\%$

$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$

$Q_{allow} = \underline{67.12\ cfs}$ OK

Pipe Flow not limiting

Check Inlet Control

- HW depth = 8.5'

Use Chart 5 - Assume Projecting Inlet

$\frac{HW}{D} = 4.25$

$Q_{HW} = 35.0\ cfs$

Inlet control not limiting

Riprap Check EPA Fig 1-15 shows $d_{50} = 0.25'$ - USE: $d_{50min} = 1.0'$; $L_a = 10'$
(If active erosion exists - 83-84 flows were > 10 yr Event)

PROJECT : Valley Camp Coal - C-28-24 Runoff Calculation

AREA= 35.0 ACRES
AVERAGE BASIN SLOPE= 51.2 PERCENT
CURVE NUMBER= 74.8
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2350. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1142 HOURS QPCFS= 231.87 CFS QPIN= 6.5699 INCHES
C3= 32.3828 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.26	.6550	.0000	.0000	.0	.00
2.28	.6812	.0000	.0000	11.6	.00
2.31	.7074	.0003	.0003	72.0	.00
2.33	.7336	.0010	.0007	153.9	.03
2.35	.7597	.0021	.0011	212.9	.12
2.37	.7859	.0036	.0015	231.9	.28
2.40	.8121	.0055	.0018	217.2	.52
2.42	.8383	.0077	.0022	183.3	.85
2.44	.8645	.0102	.0026	143.4	1.25
2.47	.8907	.0131	.0029	105.8	1.71
2.49	.9169	.0164	.0032	74.6	2.19
2.51	.9335	.0186	.0022	50.6	2.69
2.53	.9406	.0196	.0010	33.4	3.09
2.56	.9477	.0206	.0010	21.4	3.30
2.58	.9548	.0216	.0010	13.4	3.27
2.60	.9618	.0227	.0011	8.3	3.07
2.63	.9689	.0238	.0011	5.0	2.80
2.65	.9760	.0249	.0011	3.0	2.52
2.67	.9831	.0260	.0011	1.8	2.28
2.69	.9901	.0272	.0012	1.0	2.10
2.72	.9972	.0283	.0012	.6	1.97
2.74	1.0043	.0295	.0012	.3	1.88
2.76	1.0114	.0307	.0012	.2	1.84
2.79	1.0185	.0320	.0012	.1	1.82
2.81	1.0255	.0333	.0013	.0	1.82

HYDROGRAPH PEAK= 3.30 cfs
TIME TO PEAK= 2.56 Hours
RUNOFF VOLUME= .53 Acre-Feet

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE C-29-16
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED Paul
CHECKED
DATE 5/9/89

C-29-16

Tributary Flow D-29A, D-29B

$Q_{TOT} = 1.14$ cfs

Check Pipe Flow

- 16" cast iron
- $n = 0.014$
- $S = 5\%$

$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$

$Q_{allow} = 15.9$ cfs

Since $Q_{allow} > Q_{TOT}$ Culvert is acceptable OK

Check Inlet Control

HW depth = 16" $\frac{HW}{D} = 1$

Evaluate inlet based on 2 know charts from "Hydraulic Charts for the selection of Highway Culverts"

Chart 7 - assume type A $\Rightarrow Q_{HW_7} = 5$ cfs

Chart 5 - assume proj. entrance $Q_{HW_5} = 4.2$ cfs ←

Select lower value

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting OK

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.018$, $y = 0.24$, $V = 6.7$ fps

EPA fig 1-15 show data off Chart - $Q < 2$ cfs

If active erosion exists, Install $d_{50} = 0.25'$
$L_0 = 5'$

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE Culvert Calculations
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED Paul
CHECKED ___
DATE 5/9/89

C-30-24

Receives flow from D-30

flow D-30 = 4.5 cfs = Q_{TOT}

Check Pipe Characteristics

- 24" CMP
- $n = 0.024$
- $S = 4\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 24.5$ cfs

Since $Q_{allow} > Q_{TOT}$ Culvert is Acceptable

Check Inlet Control

HW depth = 48" $\frac{HW}{D} = 2$

assume projecting entrance - Based on Chart 5 of Federal Highway Administration

$Q_{HW_5} = 21.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ inlet control does not limit

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.362$, $y = 0.58'$, $V = 5.95$ fps > 5.0

Culvert discharges to Concrete ditch

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE C-31-18
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED paul
CHECKED
DATE 5/9/89

C-31-18

Tributary Flow - D-31 1.48 cfs

Check Pipe Flow

- 18" CMP
- $n = 0.024$
- $S = 18\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 24.14$ cfs

Since $Q_{allow} > Q_{TOT}$ Pipe Flow not limiting ok

Check Inlet Control

- HW depth = 21.6" $\frac{HW}{D} = 1.2$

from Chart 5 assume projecting entrance

$Q_{HW_5} = 7.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting ok

Riprap Check

Culvert discharges to Concrete ditch

HANSEN ALLEN & LUCE inc

CLIENT Valley Camp
PROJECT Permit
FEATURE C-32-36
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED Paul
CHECKED
DATE 5/9/89

C-32-36

Tributary Flow D-32A, D-32B

$Q_{TOT} = 0.58$ cfs

Check Pipe Flow

- 36" CMP
- $n = 0.024$
- $S = 4\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 72.26$ cfs

Since $Q_{allow} > Q_{TOT}$ Pipe Flow not limiting OK

Check Inlet Control

HW depth = 7.67' (drop box w/grate. Assume headwall entrance conditions to model inlet.)

$\frac{HW}{D} = 2.56$

from Chart 5 $Q_{HW_5} = 80.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet control not limiting OK

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.047$, $y = 0.19'$, $V = 3.1$ fps < 5.0 OK

NO LINING REQ'd

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE Culvert Analysis
PROJECT NO. 007.11-100

SHEET ___ OF ___
COMPUTED paul
CHECKED ___
DATE 5/9/89

C-33-24

Receives flow from D-33A, D-33B

flow D-33A = 5.36 cfs
D-33B = 6.6 cfs

Q_{TOT} = 11.96 cfs

Check Pipe Characteristics

- 24" CMP
- $n = 0.024$
- $S = 17\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Q_{allow} = 50.5 cfs

Since $Q_{allow} > Q_{TOT}$ Culvert is Adequate OK

Check Inlet Control

HW depth = 48" $\frac{HW}{D} = 2.0$

using Chart 5 → Q_{HWI} = 22.0 cfs
assume projecting entrance

Since $Q_{HWI} > Q_{TOT}$ Inlet Control does not limit OK

Riprap Check

Discharges directly to Pond 00AA. Energy adsorbed by water pool.

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE Culvert Analysis
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED Paul
CHECKED ___
DATE ___

C-34-24

Receives flow from D-34A, D-34B

flow D-34A = 0.64 cfs
D-34B = 1.88 cfs

Q_{TOT} 2.52 cfs

Check Culvert Characteristics

- 24" CMP
- $n = 0.024$
- $S = 30\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 67.17$ cfs

Since $Q_{allow} > Q_{TOT}$ Culvert is adequate

Check Inlet Control

HW depth = 4.3'

$$\frac{HW}{D} = \frac{4.3}{2} = 2.15$$

using Chart 5 $\Rightarrow$ $Q_{HW_5} = 22.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet control does not limit

Riprap Check

Discharges directly to Pond 00AA - Energy absorbed by water pool.

C-35-18

Receives flow from C-36-12 and D-35

C-36-12 has 1.06 cfs
D-35 has 3.00 cfs

$Q_{TOT} = 4.06$ cfs

Check Pipe Characteristics

- 18" Cast Iron
- $n = 0.014$
- $S = 4\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 16.89$ cfs

Since $Q_{allow} > Q_{TOT}$ Culvert is acceptable

Check Inlet Control

HW depth = $\frac{HW}{D} = 1.0$

assume projecting entrance

using Chart 5 $\Rightarrow Q_{HW_5} = 5.5$ cfs

using Chart 7 $\Rightarrow Q_{HW_7} = 6.5$ cfs

select limiting value

$Q_{HW_5} = 5.5$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet control not limiting

Riprap Check $\frac{Qn}{1.49 S^{1/2}} = 0.191$, $y = 0.47'$, $V = 8.6$ fps

Outlet believed to be stable, if however active erosion is noted, place

$d_{50} = 0.5'$
$L_a = 10'$ } As per EPA, fig 1-15

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE C-36-12
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED RWT
CHECKED ___
DATE 5/9/89

C-36-12

Trib Flow D-36, C-38-6

D-36 ⇒ 0.70
C-38-6 ⇒ 0.36

1.06 cfs = Q_{TOT}

Check Pipe Flow

- 12" CMP
- $n = 0.024$
- $S = 30\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 10.57$ cfs

Since $Q_{allow} > Q_{TOT}$ Pipe Flow not limiting OK

Check Inlet Control

HW depth = 2.2 feet $\frac{HW}{D} = 2.2$

assume projecting entrance

from Chart 5 ⇒ $Q_{HW_5} = 4.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting

Riprap Check

$\frac{Qn}{1A9S^{1/2}} = 0.031$, $y = 0.22'$, $V = 8.3$ fps > 5.0

If active erosion is noted, Place

$d_{50} = 0.5'$
$L_0 = 5'$
} As Per EPA, fig 1-15

C-37-18

Receives all flow from D-37

$Q_{TOT} = 2.6$ cfs

Check Pipe Characteristics

- 18" Cast Iron
- $n = 0.014$
- $S = 2\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 16.89$ cfs

Since $Q_{allow} > Q_{TOT}$ culvert acceptable

Check Inlet Control

HW depth = $\frac{HW}{D} = 1.0$

Check Chart 5 - Assume projecting entrance

$Q_{HW_5} = 5.5$ cfs

Check Chart 7 Assume type A entrance

$Q_{HW_7} = 7.0$ cfs

Use limiting value $Q_{HW_5} = 5.5$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.173$, $y = 0.44'$, $V = 6.02'$

Outlet is considered stable, If active erosion is noted,

Either 1) Remove convenience culvert
2) Install $d_{50} = 0.25'$
$L_a = 5'$ As per EPA fig 1-15.

HANSEN ALLEN & LUCE INC

CLIENT Valley Camp
PROJECT Permit
FEATURE C-38-6
PROJECT NO. 007.11.100

SHEET ___ OF ___
COMPUTED Paul
CHECKED ___
DATE 5/8/89

C-38-6

Area tributary = 313 * 0.00702 = 2.2 acres

		Q		
CN = 70	A = 1.8	0.096	$\bar{Q}$ = 0.210	$\overline{CN}$ = 76.1
CN = 90	A = 0.4	0.723		

Precipitation = 1.55 inches (10 yr, 6 hr SCS)

hydraulic length = 570 feet

$$\text{Slope} = \frac{(1250)(25)(100)}{(2.2)43{,}560} = 32.6\%$$

Run-off Calculations $Q = 0.36$ cfs

Check Pipe Characteristics

- 6" CMP
- $n = 0.024$
- $S = 7\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 0.80$ cfs

since $Q_{allow} > Q_{TOT}$ culvert acceptable

Check Inlet Control

$HW = 18''$ $\frac{HW}{D} = 3.0$

based upon an extrapolation of Chart 5

$Q_{HW_5} \approx 1.3$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet control not limiting

Riprap Check $\frac{Qn}{1A^{5/3}S^{1/2}} = 0.022$, $y = 0.23'$, $V = 4.1$ fps < 5.0

NO LINING REQ'd

PROJECT : Valley Camp Coal - C-38-6 Runoff Calculation

AREA= 2.2 ACRES
AVERAGE BASIN SLOPE= 32.6 PERCENT
CURVE NUMBER= 76.1
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 570. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0444 HOURS QPCFS= 37.50 CFS QPIN=16.9046 INCHES
C3= 83.3230 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.23	.6241	.0000	.0000	.0	.00
2.25	.6411	.0000	.0000	7.6	.00
2.26	.6580	.0003	.0002	28.7	.00
2.28	.6750	.0007	.0004	37.5	.01
2.29	.6919	.0013	.0006	31.7	.03
2.31	.7089	.0020	.0008	21.1	.05
2.32	.7259	.0030	.0009	12.1	.07
2.34	.7428	.0040	.0011	6.2	.09
2.35	.7598	.0053	.0013	3.0	.12
2.37	.7768	.0067	.0014	1.3	.14
2.38	.7937	.0083	.0016	.6	.17
2.40	.8107	.0100	.0017	.2	.19
2.41	.8277	.0119	.0019	.0	.22
2.43	.8446	.0140	.0020	.0	.24
2.44	.8616	.0162	.0022	.0	.26
2.46	.8786	.0185	.0023	.0	.29
2.47	.8955	.0210	.0025	.0	.31
2.48	.9125	.0236	.0026	.0	.33
2.50	.9294	.0264	.0028	.0	.35
2.51	.9344	.0272	.0008	.0	.36
2.53	.9390	.0280	.0008	.0	.32
2.54	.9436	.0288	.0008	.0	.26
2.56	.9482	.0296	.0008	.0	.20
2.57	.9528	.0304	.0008	.0	.16
2.59	.9574	.0312	.0008	.0	.14
2.60	.9619	.0321	.0008	.0	.13

HYDROGRAPH PEAK= .36 cfs
TIME TO PEAK= 2.51 Hours
RUNOFF VOLUME= .04 Acre-Feet

C-39-8

Tributary Flow D-39A, D-39B

$Q_{TOT} = 0.06$ cfs

Check Pipe Flow

- 8" CMP
- $n = 0.024$
- $S = 8\%$

$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$

$Q_{allow} = 1.85$ cfs

Since $Q_{allow} > Q_{TOT}$ Pipe Flow not limiting OK

Check Inlet Control

- HW depth = 1.9 feet $\frac{HW}{D} = 2.8$

assume projecting entrance

from Chart 5 $Q_{HW_5} = 2.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting OK

Riprap Check

No lining Req'd - by Inspection

C-40-42 tributary area + culverts C-44-24, C-41-12

Trib. Area = 20 * 2.747 = 54.9 acres

CN = 70 (25% Oak/Aspen)

Precipitation = 1.55 inches (10 yr, 6 hr SCS)

Hydraulic length = 2,000 feet

$$\text{slope} = \frac{(8{,}500)(80)(100)}{(43{,}560)(54.9)} = 28.4\%$$

Run-Off Calculation: Q = 1.81 cfs $Q_{TOT} = 1.81 + 2.46 + 0.99$

$Q_{TOT} = 5.26$ cfs

Check Pipe Characteristics

- 42" CMP
- $n = 0.027$
- $S = 30\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 265.33$ cfs

since $Q_{allow} >> Q$ culvert adequate

* For a 100 yr, 6 hr event (Not required! CHECK ONLY! - Structure not permanent)

Precip = 2.25 in $\Rightarrow Q = 10.88$ cfs

$Q_{TOT} = 10.88 + 14.82 + 1.37 + 0.99 \Rightarrow Q_{TOT} = 28.06$ cfs

Note. even under the 100 yr event

$Q_{allow} > Q$ culvert still adequate

Check Inlet Control

HW depth = 15' $\frac{HW}{D} = \frac{15}{3.5} = 4.3$

assume projecting entrance

Using Chart 5 $\Rightarrow Q_{HW_5} = 140.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control does not limit

Riprap Check from EPA, fig 1-15, $d_{50} = 0.5'$

USE $d_{50} = 1.0'$ $L_a = 10'$ min

PROJECT : Valley Camp Coal - Culvert C-40-42 Capacity Calculations

AREA= 54.9 ACRES
AVERAGE BASIN SLOPE= 28.4 PERCENT
CURVE NUMBER= 70.0
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2000. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1539 HOURS QPCFS= 269.71 CFS QPIN= 4.8720 INCHES
C3= 24.0139 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.43	.8522	.0000	.0000	.0	.00
2.46	.8875	.0002	.0002	13.5	.00
2.49	.9229	.0010	.0008	83.8	.03
2.52	.9376	.0015	.0005	179.1	.11
2.56	.9472	.0019	.0004	247.6	.24
2.59	.9567	.0023	.0004	269.7	.37
2.62	.9662	.0027	.0004	252.6	.49
2.65	.9758	.0032	.0005	213.3	.58
2.68	.9853	.0037	.0005	166.8	.65
2.71	.9949	.0043	.0006	123.1	.71
2.74	1.0044	.0049	.0006	86.7	.77
2.77	1.0140	.0055	.0006	58.9	.82
2.80	1.0235	.0062	.0007	38.8	.88
2.83	1.0331	.0069	.0007	24.9	.94
2.86	1.0426	.0077	.0008	15.6	1.01
2.89	1.0521	.0085	.0008	9.6	1.07
2.92	1.0617	.0093	.0008	5.8	1.14
2.96	1.0712	.0102	.0009	3.5	1.20
2.99	1.0808	.0111	.0009	2.1	1.27
3.02	1.0893	.0119	.0008	1.2	1.33
3.05	1.0969	.0127	.0008	.7	1.39
3.08	1.1045	.0135	.0008	.4	1.43
3.11	1.1122	.0143	.0008	.2	1.45
3.14	1.1198	.0152	.0008	.1	1.46
3.17	1.1274	.0160	.0009	.0	1.47
2.43	.8522	.0000	.0000	.0	.00
2.46	.8875	.0002	.0002	13.5	.00
2.49	.9229	.0010	.0008	83.8	.03
2.52	.9376	.0015	.0005	179.1	.11
2.56	.9472	.0019	.0004	247.6	.24
2.59	.9567	.0023	.0004	269.7	.37
2.62	.9662	.0027	.0004	252.6	.49
2.65	.9758	.0032	.0005	213.3	.58
2.68	.9853	.0037	.0005	166.8	.65
2.71	.9949	.0043	.0006	123.1	.71

PROJECT : Valley Camp Coal - Culvert C-40-42 Capacity Calculations (Continued)

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.74	1.0044	.0049	.0006	86.7	.77
2.77	1.0140	.0055	.0006	58.9	.82
2.80	1.0235	.0062	.0007	38.8	.88
2.83	1.0331	.0069	.0007	24.9	.94
2.86	1.0426	.0077	.0008	15.6	1.01
2.89	1.0521	.0085	.0008	9.6	1.07
2.92	1.0617	.0093	.0008	5.8	1.14
2.96	1.0712	.0102	.0009	3.5	1.20
2.99	1.0808	.0111	.0009	2.1	1.27
3.02	1.0893	.0119	.0008	1.2	1.33
3.05	1.0969	.0127	.0008	.7	1.39
3.08	1.1045	.0135	.0008	.4	1.43
3.11	1.1122	.0143	.0008	.2	1.45
3.14	1.1198	.0152	.0008	.1	1.46
3.17	1.1274	.0160	.0009	.0	1.47
3.20	1.1351	.0169	.0009	.0	1.48
3.23	1.1427	.0178	.0009	.0	1.49
3.26	1.1503	.0188	.0009	.0	1.51
3.29	1.1580	.0197	.0010	.0	1.54
3.33	1.1656	.0207	.0010	.0	1.57
3.36	1.1732	.0217	.0010	.0	1.60
3.39	1.1809	.0227	.0010	.0	1.63
3.42	1.1885	.0238	.0010	.0	1.67
3.45	1.1961	.0248	.0011	.0	1.71
3.48	1.2038	.0259	.0011	.0	1.75
3.51	1.2107	.0269	.0010	.0	1.79
3.54	1.2159	.0277	.0008	.0	1.81
3.57	1.2212	.0285	.0008	.0	1.81
3.60	1.2264	.0293	.0008	.0	1.78
3.63	1.2317	.0301	.0008	.0	1.72
3.66	1.2369	.0309	.0008	.0	1.66
3.69	1.2422	.0317	.0008	.0	1.60
3.73	1.2474	.0326	.0008	.0	1.56

HYDROGRAPH PEAK= 1.81 cfs
TIME TO PEAK= 3.54 Hours
RUNOFF VOLUME= .44 Acre-Feet

PROJECT : Valley Camp Coal - Culvert C-40-42 (100 yr, 6 hour SCS)

AREA= 54.9 ACRES
AVERAGE BASIN SLOPE= 28.4 PERCENT
CURVE NUMBER= 70.0
DESIGN STORM= 2.25 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2000. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1539 HOURS QPCFS= 269.71 CFS QPIN= 4.8720 INCHES
C3= 24.0139 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.19	.8270	.0000	.0000	.0	.00
2.22	.8783	.0001	.0001	13.5	.00
2.25	.9295	.0012	.0011	83.8	.02
2.28	.9808	.0035	.0023	179.1	.14
2.31	1.0321	.0069	.0034	247.6	.46
2.34	1.0833	.0113	.0045	269.7	1.05
2.37	1.1346	.0169	.0055	252.6	1.94
2.40	1.1858	.0234	.0065	213.3	3.10
2.43	1.2371	.0309	.0075	166.8	4.49
2.46	1.2884	.0394	.0085	123.1	6.03
2.49	1.3396	.0488	.0094	86.7	7.67
2.52	1.3610	.0530	.0042	58.9	9.29
2.56	1.3749	.0558	.0028	38.8	10.46
2.59	1.3888	.0587	.0029	24.9	10.88
2.62	1.4026	.0616	.0029	15.6	10.58
2.65	1.4165	.0646	.0030	9.6	9.81
2.68	1.4303	.0676	.0030	5.8	8.88
2.71	1.4442	.0707	.0031	3.5	7.98
2.74	1.4580	.0739	.0032	2.1	7.24
2.77	1.4719	.0771	.0032	1.2	6.68
2.80	1.4857	.0804	.0033	.7	6.30
2.83	1.4996	.0838	.0033	.4	6.07
2.86	1.5134	.0872	.0034	.2	5.95
2.89	1.5273	.0906	.0035	.1	5.90
2.92	1.5412	.0941	.0035	.0	5.92

HYDROGRAPH PEAK= 10.88 cfs
TIME TO PEAK= 2.59 Hours
RUNOFF VOLUME= 1.55 Acre-Feet

HANSEN ALLEN & LUCE inc

CLIENT Valley Camp
PROJECT Permit
FEATURE Culverts
PROJECT NO. 007-11-100

SHEET ___ OF ___
COMPUTED Paul
CHECKED ___
DATE 5/9/89

C-41-12

Mine Discharge from concrete settling Ponds.

data based on records gathered by Vaughn Hansen Assoc from 9/77 to 10/83.

Max value measured. ⇒ 9/12/83 $\underline{0.99 \text{ cfs}} = Q_{TOT}$

Assume maximum value.

Check Pipe Characteristics

- 12" CMP
- $n = 0.024$
- $S = 17\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$\underline{Q_{allow} = 7.96 \text{ cfs}}$$

Since $Q_{allow} > Q$ Culvert is acceptable

Check Inlet Control

HW depth = 2.4' $\quad \frac{HW}{D} = 2.4$

assume projecting entrance

Use Chart 5 ⇒ $\underline{Q_{HW_5} = 4.5 \text{ cfs}}$

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.039$, $y = 0.24$, $V = 6.8$ fps > 5.0

Mine discharge is very constant, If active erosion is noted, place $d_{50\,mm} = 1.0'$ at outlet for a minimum of 10'

C-42-8

Tributary Area D-42

$Q_{TOT} = 0.51$ cfs

Check Pipe Flow

- $n = 0.024$
- 8" CMP
- $S = 5\%$

$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$

$Q_{allow} = 1.46$ cfs

Since $Q_{allow} > Q_{TOT}$ Pipe flow not limiting ok

Check Inlet Control

- Hwd depth = 2.67' $\frac{HW}{D} = \frac{2.67}{.67} = 4.0$

assume projecting entrance

from Chart 5 ⇒ $Q_{HW_5} = 2.5$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting ok

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.037$, $y = 0.27'$, $V = 3.85$ fps < 5.0

NO LINING REQ'd

C-43-8

Total Area = 184 * 0.00702 = 1.3 ac
Undist. Area = 57 * 0.00702 = 0.4 ac (25% Oak/Aspen)
Dist. Area = 130 * 0.00702 = 0.9 ac

CN	Area	Q
70	0.4	0.096
90	0.9	0.723

$\bar{Q} = 0.530$ $\bar{CN} = 86.1$

Precip = 1.55" (10 yr, 6 hr, SCS)

Hydraulic length = 325 feet

$$\text{slope} = \frac{(350)(25)(100)}{43{,}560\,(1.3)} = 24.3\%$$

Run-Off Calculation: $Q = 0.71$ cfs

Check Pipe Characteristics

8" CMP
$n = 0.024$
$S = 30\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 3.59$ cfs

since $Q_{allow} > Q$ culvert adequate

For a 100 year, 6 hour event - NOT REQUIRED, CHECK ONLY

$P = 2.25$ inches $\Rightarrow Q_{TOT} = 1.37$ cfs

since $Q_{allow} > Q_{TOT}$ culvert is acceptable

Check Inlet Control

HW depth = 2.17'

$$\frac{HW}{D} = \frac{2.17}{0.67} = 3.25$$

Use projecting entrance

Use Chart 5 $\Rightarrow Q_{HW_5} = 2.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting

C-43-8 - (Continued)

Riprap Check

$\frac{Qn}{1AS^{1/2}} = 0.021$, y 0.021', V = 8.1 fps > 5.0

Outlet believed to be stable, EPA figure 1-15 shows data off Chart. Should active erosion be noted, Place riprap at outlet.

$d_{50} = 0.25$
$L = 5'$

PROJECT : Valley Camp Coal - Culvert C-43-8 Capacity Calculations

AREA= 1.3 ACRES
AVERAGE BASIN SLOPE= 24.3 PERCENT
CURVE NUMBER= 86.1
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 325. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0238 HOURS QPCFS= 41.38 CFS QPIN=31.5630 INCHES
C3=155.5737 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
1.88	.3212	.0000	.0000	.0	.00
1.90	.3282	.0000	.0000	41.4	.00
1.93	.3352	.0000	.0000	13.2	.00
1.95	.3422	.0002	.0001	1.5	.00
1.98	.3493	.0004	.0002	.1	.00
2.00	.3563	.0007	.0003	.0	.01
2.36	.7651	.0951	.0103	.0	.57
2.38	.7924	.1058	.0107	.0	.59
2.40	.8197	.1169	.0111	.0	.62
2.43	.8470	.1284	.0115	.0	.64
2.45	.8743	.1404	.0119	.0	.66
2.48	.9016	.1527	.0123	.0	.69
2.50	.9289	.1654	.0127	.0	.71
2.52	.9371	.1693	.0039	.0	.35
2.55	.9444	.1728	.0035	.0	.22
2.57	.9518	.1763	.0035	.0	.20
2.59	.9592	.1799	.0036	.0	.20
2.62	.9666	.1835	.0036	.0	.20
2.64	.9740	.1871	.0036	.0	.20

HYDROGRAPH PEAK= .71 cfs
TIME TO PEAK= 2.50 Hours
RUNOFF VOLUME= .06 Acre-Feet

PROJECT : Valley Camp Coal - Culvert C-43-8 (100 yr, 6 hour SCS)

AREA= 1.3 ACRES
AVERAGE BASIN SLOPE= 24.3 PERCENT
CURVE NUMBER= 86.2
DESIGN STORM= 2.25 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 325. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .0237 HOURS QPCFS= 41.53 CFS QPIN=31.6773 INCHES
C3=156.1375 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
1.52	.3109	.0000	.0000	.0	.00
1.54	.3211	.0000	.0000	41.5	.00
1.56	.3312	.0000	.0000	13.3	.00
1.59	.3413	.0003	.0002	1.5	.00
1.61	.3515	.0006	.0003	.1	.02
1.64	.3616	.0010	.0004	.0	.02
2.35	1.0941	.2522	.0212	.0	1.19
2.37	1.1336	.2740	.0218	.0	1.22
2.39	1.1730	.2964	.0224	.0	1.25
2.42	1.2125	.3193	.0229	.0	1.28
2.44	1.2519	.3428	.0234	.0	1.31
2.46	1.2914	.3667	.0239	.0	1.34
2.49	1.3309	.3911	.0244	.0	1.37
2.51	1.3555	.4066	.0155	.0	1.00
2.54	1.3662	.4133	.0067	.0	.52
2.56	1.3768	.4201	.0068	.0	.40
2.58	1.3875	.4269	.0068	.0	.38
2.61	1.3982	.4338	.0068	.0	.39
2.63	1.4088	.4406	.0069	.0	.39

HYDROGRAPH PEAK= 1.37 cfs
TIME TO PEAK= 2.49 Hours
RUNOFF VOLUME= .11 Acre-Feet

C-44-24

Total Area = 27+(2.747) = 74.2 Acres (25% Oak/Aspen)

CN = 70 (25% Oak/Aspen)

Precipitation = 1.55 inches (10 year, 6 hr SCS)

Hydraulic length = 2,000 feet

$$\text{slope} = \frac{(12{,}200)(80)(100)}{43{,}560\ (74.2)} = 30.2\%$$

$T_c = 0.224$ hrs.

Run-Off Calculation: $Q = 2.46$ cfs

Check Pipe Characteristics

- 24" CMP
- $n = 0.024$
- $S = 8\%$

$$Q_{allow} = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$Q_{allow} = 34.66$ cfs

Since $Q_{allow} > Q$ culvert adequate

* For a 100 year, 6hr event - NOT REQ'd - CHECK ONLY

Precip = 2.25 inches $\Rightarrow Q_{TOT}$ 14.82 cfs

$Q_{TOT} = 14.82 + 1.37$ ← From C-43-8 $Q_{TOT} = 16.19$ cfs

Note: even under the 24 hr. conditions

$Q_{allow} > Q$ ∴ culvert acceptable

Check Inlet Control

HW depth = 6.3 feet $\frac{HW}{D} = \frac{6.3}{2.0} = 3.15$

Assume projecting entrance

Use Chart 5 $\Rightarrow Q_{HW_5} = 29.0$ cfs

Since $Q_{HW_5} > Q_{TOT}$ Inlet Control not limiting

C-44-2A - (Continued)

Riprap Check

$\frac{Qn}{1.49 S^{1/2}} = 0.140$, $y = 0.36'$, $V = 6.4$ fps > 5.0

from EPA fig 1-15, $d_{50} = 0.3'$ use $0.5'$
$L = 5'$

Recommend modification to stream channel only if active erosion is noted. Stream is believed to be stable under current conditions.

PROJECT : Valley Camp Coal - Culvert C-44-24 Capacity Calculation

AREA= 74.2 ACRES
AVERAGE BASIN SLOPE= 30.2 PERCENT
CURVE NUMBER= 70.0
DESIGN STORM= 1.55 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2000. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1493 HOURS QPCFS= 375.90 CFS QPIN= 5.0240 INCHES
C3= 24.7632 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.42	.8363	.0000	.0000	.0	.00
2.45	.8705	.0000	.0000	18.9	.00
2.48	.9048	.0005	.0005	116.8	.01
2.51	.9324	.0013	.0008	249.5	.08
2.54	.9417	.0016	.0003	345.1	.23
2.57	.9510	.0020	.0004	375.9	.42
2.60	.9602	.0024	.0004	352.1	.60
2.63	.9695	.0029	.0004	297.2	.74
2.66	.9787	.0034	.0005	232.5	.85
2.69	.9880	.0039	.0005	171.5	.93
2.72	.9972	.0044	.0006	120.9	1.00
2.75	1.0065	.0050	.0006	82.1	1.07
2.78	1.0157	.0057	.0006	54.1	1.14
2.81	1.0250	.0063	.0007	34.7	1.22
2.84	1.0343	.0070	.0007	21.8	1.30
2.87	1.0435	.0078	.0007	13.4	1.38
2.90	1.0528	.0085	.0008	8.1	1.47
2.93	1.0620	.0093	.0008	4.9	1.55
2.96	1.0713	.0102	.0008	2.9	1.64
2.99	1.0805	.0111	.0009	1.7	1.73
3.02	1.0888	.0119	.0008	1.0	1.81
3.05	1.0962	.0126	.0008	.6	1.89
3.08	1.1036	.0134	.0008	.3	1.94
3.10	1.1110	.0142	.0008	.2	1.97
3.13	1.1184	.0150	.0008	.0	1.98
3.37	1.1777	.0223	.0010	.0	2.20
3.40	1.1851	.0233	.0010	.0	2.24
3.43	1.1925	.0243	.0010	.0	2.29
3.46	1.1999	.0254	.0010	.0	2.35
3.49	1.2073	.0264	.0011	.0	2.40
3.52	1.2129	.0273	.0008	.0	2.44
3.55	1.2180	.0280	.0008	.0	2.46
3.58	1.2231	.0288	.0008	.0	2.44
3.61	1.2282	.0296	.0008	.0	2.37

PROJECT : Valley Camp Coal - Culvert C-44-24 Capacity Calculation
(Continued)

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
3.64	1.2333	.0303	.0008	.0	2.29
3.67	1.2384	.0311	.0008	.0	2.21
3.70	1.2435	.0319	.0008	.0	2.14
3.73	1.2485	.0328	.0008	.0	2.09

HYDROGRAPH PEAK= 2.46 cfs
TIME TO PEAK= 3.55 Hours
RUNOFF VOLUME= .59 Acre-Feet

PROJECT : Valley Camp Coal - Culvert C-44-24 (100 yr, 6 hour SCS)

AREA= 74.2 ACRES
AVERAGE BASIN SLOPE= 30.2 PERCENT
CURVE NUMBER= 70.0
DESIGN STORM= 2.25 INCHES
STORM DURATION= 6.0 HOURS
HYDRAULIC LENGTH= 2000. FEET
MINIMUM INFILTRATION RATE= .00 IN/HR

TP= .1493 HOURS QPCFS= 375.90 CFS QPIN= 5.0240 INCHES
C3= 24.7632 ITERATIONS= 8 SCS 6-hour

TIME HOURS	ACCUMULATED RAINFALL INCHES	RUNOFF INCHES	RAINFALL EXCESS INCHES	UNIT HYDROGRAPH CFS	OUTFLOW HYDROGRAPH CFS
2.18	.8163	.0000	.0000	.0	.00
2.21	.8660	.0000	.0000	18.9	.00
2.24	.9157	.0008	.0008	116.8	.02
2.27	.9654	.0027	.0019	249.5	.13
2.30	1.0152	.0056	.0029	345.1	.47
2.33	1.0649	.0096	.0040	375.9	1.16
2.36	1.1146	.0146	.0050	352.1	2.24
2.39	1.1643	.0205	.0060	297.2	3.69
2.42	1.2140	.0274	.0069	232.5	5.45
2.45	1.2637	.0352	.0078	171.5	7.43
2.48	1.3134	.0439	.0087	120.9	9.57
2.51	1.3535	.0515	.0076	82.1	11.76
2.54	1.3670	.0542	.0027	54.1	13.69
2.57	1.3804	.0569	.0027	34.7	14.78
2.60	1.3939	.0597	.0028	21.8	14.82
2.63	1.4073	.0626	.0029	13.4	14.02
2.66	1.4207	.0655	.0029	8.1	12.79
2.69	1.4342	.0685	.0030	4.9	11.49
2.72	1.4476	.0715	.0030	2.9	10.33
2.75	1.4610	.0746	.0031	1.7	9.42
2.78	1.4745	.0777	.0031	1.0	8.77
2.81	1.4879	.0809	.0032	.6	8.34
2.84	1.5013	.0842	.0033	.3	8.09
2.87	1.5148	.0875	.0033	.2	7.98
2.90	1.5282	.0908	.0034	.0	7.95

HYDROGRAPH PEAK= 14.82 cfs
TIME TO PEAK= 2.60 Hours
RUNOFF VOLUME= 2.10 Acre-Feet

I - Undisturbed Area Runoff Channels

A - Drainage Basin Characteristics

Channel	Basin Area (acres)	Hydraulic Length ft	Watershed Slope %
UDA-1	6.7	700	27
UDA-2	14.7	1900	37
UDA-3	8.1	1500	32
UDA-4	5.5	1250	31
UDA-5	7.2	1100	28

$$\text{Watershed Slope} = \frac{L_c * CI}{DA} * 100$$

L_c = length of Contour lines
CI = Contour Interval in feet.
DA = Drainage Area

Channel	L_c ft	CI ft	DA ft²	Slope %
UDA-1	1000	80	291,852	27
UDA-2	11,900	20	640,332	37
UDA-3	5,600	20	352,836	32
UDA-4	3750	20	239,580	31
UDA-5	4400	20	313,632	28

B - Curve Number

Use CN = 75 for sage-grass cover
CN = 70 for Aspen or Forest Cover

See Calculations 9/18/88 for justification.

Channel	Area Sage-Grass ac	Area Aspen or Forest ac
UDA-1	6.7	0
UDA-2	1.4	13.3
UDA-3	8.1	0
UDA-4	0	5.5
UDA-5	0	7.2

HANSEN ALLEN & LUCE INC

CLIENT: Valley County
PROJECT: Hydrology - Surface Facilities
FEATURE: Runoff; Peak Flow Projections
PROJECT NO.:

SHEET 2 OF 17
COMPUTED: DMA
CHECKED:
DATE: 1/18/89

Weighted Curve #s based on runoff volume

Channel	Area ac	CN	10-yr 24-hr Runoff Depth in	Wt. CN based on Wt. Q
UAD-1	6.7	75	0.62	75
	0	70	0.43	
			WTQ = 0.62	
UAD-2	1.4	75	0.62	
	13.3	70	0.43	70.5 Use 71
			WTQ = 0.45	
UAD-3	8.1	75	0.62	
	0.0	70	0.43	75
			WTQ = 0.62	
UAD-4	0	75	0.62	
	5.5	70	0.43	70
			WTQ = 0.43	
UAD-5	0	75	0.62	
	7.2	70	0.43	70
			WTQ = 0.43	

10-yr 24-hr Precip = 2.45 inches

10-yr 6-hr Precip = 1.55 inches.