APPENDIX C
SAMPLE PROBLEMS

C-1. Grit chambers. (Refer to para 10-4b)

a. Weir velocity control. As stated previously in paragraph 10-4b, a proportional weir, Parshall flume or a Venturi flume may be used for control of water velocity in grit chambers. Illustrations of these control devices are contained in figure C-1.

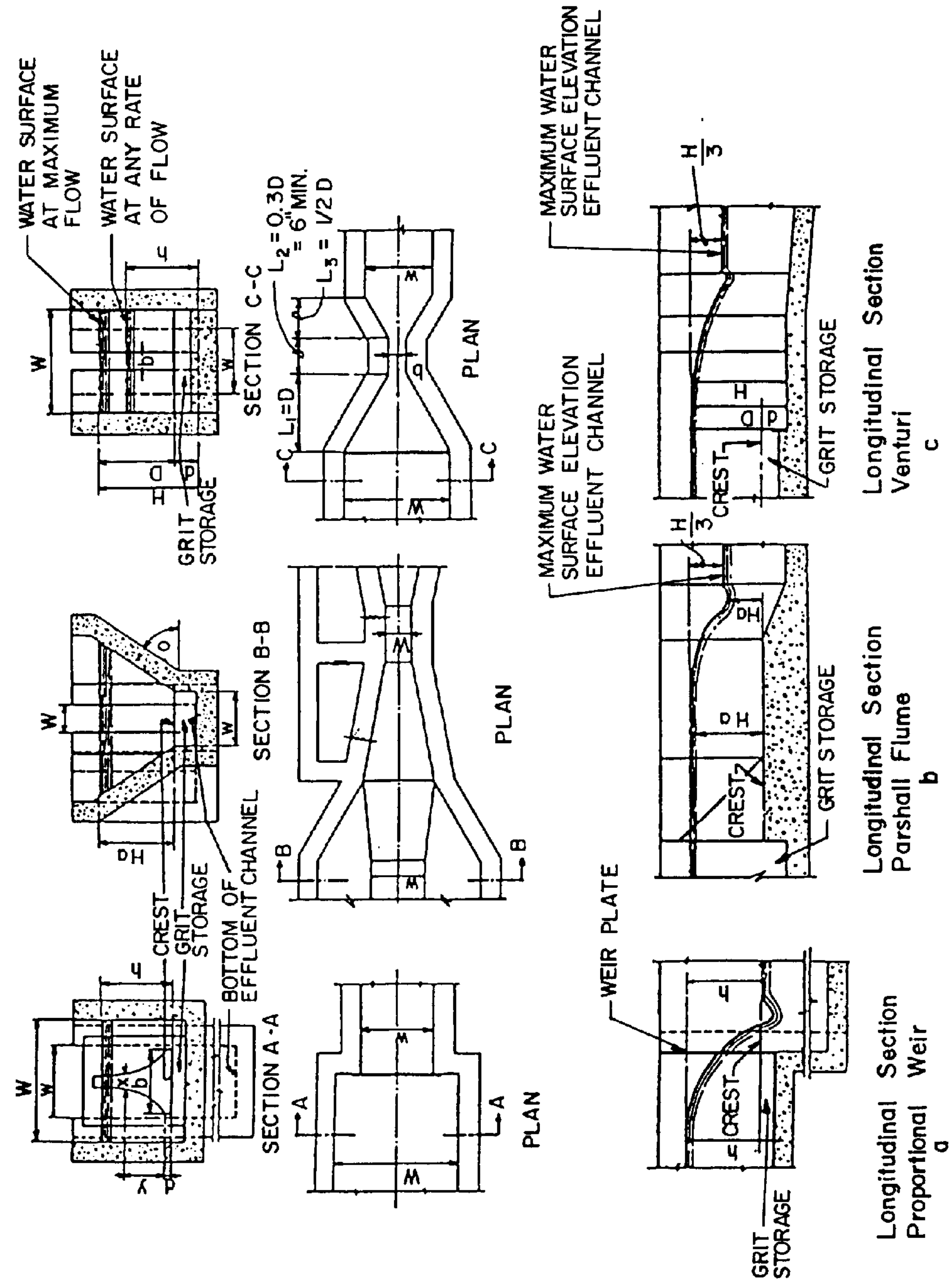


Figure C-1. Grit chamber velocity control devices.

(1) **Proportional weir design formula.**

$$W = \frac{Q}{vh} \qquad \text{(eq C–1)}$$

$$Q = cb\sqrt{2G}\left(h - \frac{d}{3}\right)\sqrt{d} \qquad \text{(eq C–2)}$$

$$x = 2b\left(1 - \frac{2}{\pi}\arctan\sqrt{\frac{y}{d}}\right) \qquad \text{(eq C–3)}$$

in which

h = static head above weir crest;

G = acceleration due to gravity (normally 32.2 fps/sec);

Q = total discharge past weir (cfs);

c = weir coefficient (0.61 for practical design);

v = velocity through grit chamber (fps);

π = 3.1416 when arctan $\sqrt{\frac{y}{d}}$ is expressed in radians

Other symbols are as indicated in figure C-1**a** and all dimensions are expressed in feet.

Table C–1. Values of x/b for various values of y/d as related to equation C–3.

y/d	x/b	y/d	x/b	y/d	x/b
0.0	1.000				
0.1	0.805	1.0	0.500	10	0.195
0.2	0.732	2.0	0.392	12	0.179
0.3	0.681	3.0	0.333	14	0.166
0.4	0.641	4.0	0.295	16	0.156
0.5	0.608	5.0	0.268	18	0.147
0.6	0.580	6.0	0.247	20	0.140
0.7	0.556	7.0	0.230	25	0.126
0.8	0.536	8.0	0.216	30	0.115
0.9	0.517	9.0	0.205		

(2) **Parshall flume design formula**. In the illustration (fig C-1**b**), the depth of the liquid in the grit chamber above the grit space is the same as Ha, the depth in the Parshall flume ahead of the throat. The velocity through a channel of the illustrated cross-section can be determined from the following formula:

$$v = \frac{Q}{N(w + H_a \cot \theta)H_a} \qquad \text{(eq C–4)}$$

in which

Q = discharge of Parshall flume (cfs);

N = number of grit settling channels in service;

H_a = depth of liquid in the grit chamber and in the Parshall flume ahead of the throat, and other symbols are as previously stated or as indicated in figure C-1**b**.

Table C-2 illustrates the variations in velocity through channels of different sizes used in conjunction with Parshall flumes of different sizes discharging various quantities under free-flow conditions. The quantities and values of H_a are taken from Table C-2. Table C-3 can serve as a guide in selecting the size of Parshall flume and the number and approximate size of channels to suit a specified set of flow conditions.

Table C–2. Table of discharge rates for Parshall flumes.

Head, lin (feet)	3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	Discharge, Q, for throat widths, W, of --						
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
0.10	0.028	0.05	0.09	----	----	----	----
.11	.033	.06	.10	----	----	----	----
.12	.037	.07	.12	----	----	----	----
.13	.042	.08	.14	----	----	----	----
.14	.047	.09	.15	----	----	----	----
.15	.053	.10	.17	----	----	----	----
.16	.058	.11	.19	----	----	----	----
.17	.064	.12	.20	----	----	----	----
.18	.070	.14	.22	----	----	----	----
.19	.076	.15	.24	----	----	----	----
.20	.082	.16	.26	0.35	0.51	0.66	0.97
.21	.089	.18	.28	.37	.55	.71	1.04
.22	.095	.19	.30	.40	.59	.77	1.12
.23	.102	.20	.32	.43	.63	.82	1.20
.24	.109	.22	.35	.46	.67	.88	1.28
.25	.117	.23	.37	.49	.71	.93	1.37
.26	.124	.25	.39	.51	.76	.99	1.46
.27	.131	.26	.41	.54	.80	1.05	1.55
.28	.138	.28	.44	.58	.85	1.11	1.64
.29	.146	.29	.46	.61	.90	1.18	1.73
.30	.154	.31	.49	.64	.94	1.24	1.82
.31	.162	.32	.51	.68	.99	1.30	1.92
.32	.170	.34	.54	.71	1.04	1.37	2.02
.33	.179	.36	.56	.74	1.09	1.44	2.12
.34	.187	.38	.59	.77	1.14	1.50	2.22
.35	.196	.39	.62	.80	1.19	1.57	2.32
.36	.205	.41	.64	.84	1.25	1.64	2.42
.37	.213	.43	.67	.88	1.30	1.72	2.53
.38	.222	.45	.70	.92	1.36	1.79	2.64
.39	.231	.47	.73	.95	1.41	1.86	2.75
.40	.241	.48	.76	.99	1.47	1.93	2.86
.41	.250	.50	.78	1.03	1.53	2.01	2.97
.42	.260	.52	.81	1.07	1.58	2.09	3.08
.43	.269	.54	.84	1.11	1.64	2.16	3.29
.44	.279	.56	.87	1.15	1.70	2.24	3.32
.45	.289	.58	.90	1.19	1.76	2.32	3.44
.46	.299	.61	.94	1.23	1.82	2.40	3.56
.47	.309	.63	.97	1.27	1.88	2.48	3.68
.48	.319	.65	1.00	1.31	1.94	2.57	3.80
.49	.329	.67	1.03	1.35	2.00	2.65	3.92

Table C-2. (Cont'd)

Head, lin (feet)	Discharge, Q, for throat widths, W, of -- 3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
.50	.339	.69	1.06	1.39	2.00	2.73	4.05
.51	.350	.71	1.10	1.44	2.13	2.82	4.18
.52	.361	.73	1.13	1.48	2.19	2.90	4.31
.53	.371	.76	1.16	1.52	2.25	2.99	4.44
.54	.382	.78	1.20	1.57	2.32	3.08	4.57
.55	.393	.80	1.23	1.62	2.39	3.17	4.70
.56	.404	.82	1.26	1.66	2.45	3.26	4.84
.57	.415	.85	1.30	1.70	2.52	3.35	4.98
.58	.427	.87	1.33	1.75	2.59	3.44	5.11
.59	.438	.89	1.37	1.80	2.66	3.53	5.25
.60	.450	.92	1.40	1.84	2.73	3.62	5.39
.61	.462	.94	1.44	1.88	2.80	3.72	5.53
.62	.474	.97	1.48	1.93	2.87	3.81	5.68
.63	.485	.99	1.51	1.98	2.95	3.91	5.82
.64	.497	1.02	1.55	2.03	3.02	4.01	5.97
.65	.509	1.04	1.59	2.08	3.09	4.11	6.12
.66	.522	1.07	1.63	2.13	3.17	4.20	6.26
.67	.534	1.10	1.66	2.18	3.24	4.30	6.41
.68	.546	1.12	1.70	2.23	3.31	4.40	6.50
.69	.558	1.15	1.74	2.28	3.39	4.50	6.71
.70	.571	1.17	1.78	2.33	3.46	4.60	6.86
.71	.584	1.20	1.82	2.38	3.51	4.70	7.02
.72	.597	1.23	1.86	2.43	3.62	4.81	7.17
.73	.610	1.26	1.90	2.48	3.69	4.91	7.33
.74	.623	1.28	1.91	2.53	3.77	5.02	7.49
.75	.636	1.31	1.98	2.58	3.85	5.12	7.65
.76	.649	1.34	2.02	2.63	3.93	5.23	7.81
.77	.662	1.36	2.06	2.68	4.01	5.34	7.97
.78	.675	1.39	2.10	2.74	4.09	5.44	8.13
.79	.689	1.42	2.14	2.80	4.17	5.55	8.30
.80	.702	1.45	2.18	2.85	4.26	5.66	8.46
.81	.716	1.48	2.22	2.90	4.34	5.77	8.63
.82	.730	1.50	2.27	2.96	4.42	5.88	8.79
.83	.744	1.53	2.31	3.02	4.50	6.00	8.96
.84	.757	1.56	2.35	3.07	4.59	6.11	9.13
.85	.771	1.59	2.39	3.12	4.67	6.22	9.30
.86	.786	1.62	2.44	3.18	4.76	6.33	9.48
.87	.800	1.65	2.48	3.24	4.84	6.44	9.65
.88	.814	1.68	2.52	3.29	4.93	6.56	9.82
.89	.828	1.71	2.57	3.35	5.01	6.68	10.0

Table C–2. (Cont'd)

Head, lin (feet)	Discharge, Q, for throat widths, W, of -- 3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
.90	.843	1.74	2.61	3.41	5.10	6.80	10.2
.91	.858	1.77	2.66	3.46	5.19	6.92	10.4
.92	.872	1.81	2.70	3.52	5.28	7.03	10.5
.93	.887	1.84	2.75	3.58	5.37	7.15	10.7
.94	.902	1.87	2.79	3.64	5.46	7.27	10.9
.95	.916	1.90	2.84	3.70	5.55	7.39	11.1
.96	.931	1.93	2.88	3.76	5.64	7.51	11.3
.97	.946	1.97	2.93	3.82	5.73	7.63	11.4
.98	.961	2.00	2.98	3.88	5.82	7.75	11.6
.99	.977	2.03	3.02	3.94	5.91	7.88	11.8
1.00	.992	2.06	3.07	4.00	6.00	8.00	12.0
1.01	1.01	2.09	3.12	4.06	6.09	8.12	12.2
1.02	1.02	2.12	3.17	4.12	6.19	8.25	12.4
1.03	1.04	2.16	3.21	4.18	6.28	8.38	12.6
1.04	1.05	2.19	3.26	4.25	6.37	8.50	12.8
1.05	1.07	2.22	3.31	4.31	6.47	8.63	13.0
1.06	1.09	2.26	3.30	4.37	6.56	8.76	13.2
1.07	1.10	2.29	3.40	4.43	6.66	8.88	13.3
1.08	1.12	2.32	3.45	4.50	6.75	9.01	13.5
1.09	1.13	2.36	3.50	4.56	6.85	9.14	13.7
1.10	----	2.40	3.55	4.62	6.95	9.27	13.9
1.11	----	2.43	3.60	4.68	7.04	9.40	14.1
1.12	----	2.46	3.65	4.75	7.14	9.54	14.3
1.13	----	2.50	3.70	4.82	7.24	9.67	14.5
1.14	----	2.53	3.75	4.88	7.34	9.80	14.7
1.15	----	2.57	3.80	4.94	7.44	9.94	14.9
1.16	----	2.60	3.85	5.01	7.54	10.1	15.1
1.17	----	2.64	3.90	5.08	7.64	10.2	15.3
1.18	----	2.68	3.95	5.15	7.74	10.3	15.6
1.19	----	2.71	4.01	5.21	7.84	10.5	15.8
1.20	----	2.75	4.06	5.28	7.94	10.6	16.0
1.21	----	2.78	4.11	5.34	8.05	10.8	16.2
1.22	----	2.82	4.16	5.41	8.15	10.9	16.4
1.23	----	2.86	4.22	5.48	8.25	11.0	16.6
1.24	----	2.89	4.27	5.55	8.36	11.2	16.8
1.25	----	2.93	4.32	5.62	8.46	11.3	17.0
1.26	----	2.97	4.37	5.69	8.56	11.5	17.2
1.27	----	3.01	4.43	5.76	8.67	11.6	17.4
1.28	----	3.04	4.48	5.82	8.77	11.7	17.7
1.29	----	3.08	4.53	5.89	8.88	11.9	17.9

Table C–2. (Cont'd)

Head, lin (feet)	Discharge, Q, for throat widths, W, of -- 3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
1.30	----	3.12	4.59	5.96	8.99	12.0	18.1
1.31	----	3.16	4.61	6.03	9.09	12.2	18.3
1.32	----	3.19	4.69	6.10	9.20	12.3	18.5
1.33	----	3.23	4.75	6.18	9.30	12.4	18.8
1.34	----	3.27	4.80	6.25	9.41	12.6	19.0
1.35	----	3.31	4.86	6.32	9.52	12.7	19.2
1.36	----	3.35	4.92	6.39	9.63	12.9	19.4
1.37	----	3.39	4.97	6.46	9.74	13.0	19.6
1.38	----	3.43	5.03	6.53	9.85	13.2	19.9
1.39	----	3.47	5.08	6.60	9.96	13.3	20.1
1.40	----	3.51	5.14	6.68	10.4	13.5	20.3
1.41	----	3.55	5.19	6.75	10.2	13.6	20.6
1.42	----	3.59	5.25	6.82	10.3	13.8	20.8
1.43	----	3.63	5.31	6.89	10.4	13.9	21.0
1.44	----	3.67	5.37	6.97	10.5	14.1	21.2
1.45	----	3.71	5.42	7.04	10.6	14.2	21.5
1.46	----	3.75	5.48	7.12	10.7	14.4	21.7
1.47	----	3.79	5.54	7.19	10.8	14.5	21.9
1.48	----	3.83	5.59	7.26	11.0	14.7	22.2
1.49	----	3.87	5.65	7.34	11.1	14.9	22.4
1.50	----	----	5.71	7.41	11.2	15.0	22.6
1.51	----	----	5.77	7.49	11.3	15.2	22.9
1.52	----	----	5.83	7.57	11.4	15.3	23.1
1.53	----	----	5.89	7.64	11.5	15.5	23.4
1.54	----	----	5.94	7.72	11.7	15.6	23.6
1.55	----	----	6.00	7.80	11.8	15.8	23.8
1.56	----	----	6.06	7.87	11.9	15.9	24.1
1.57	----	----	6.12	7.95	12.0	16.1	24.3
1.58	----	----	6.18	8.02	12.1	16.3	24.6
1.59	----	----	6.24	8.10	12.2	16.4	24.8
1.60	----	----	6.31	8.18	12.4	16.6	25.1
1.61	----	----	6.37	8.26	12.5	16.7	25.3
1.62	----	----	6.43	8.34	12.6	16.9	25.5
1.63	----	----	6.49	8.42	12.7	17.1	25.8
1.64	----	----	6.55	8.49	12.8	17.2	26.0
1.65	----	----	6.61	8.57	12.0	17.4	26.3
1.66	----	----	6.67	8.65	13.1	17.6	26.5
1.67	----	----	6.73	8.73	13.2	17.7	26.8
1.68	----	----	6.79	8.81	12.3	17.9	27.0
1.69	----	----	6.80	8.89	13.5	18.0	27.3

Table C-2. (Cont'd)

Head, lin (feet)	Discharge, Q, for throat widths, W, of -- 3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
1.70	----	----	6.92	8.97	13.6	18.2	27.6
1.71	----	----	6.98	9.05	13.7	18.4	27.8
1.72	----	----	7.04	9.13	13.8	18.5	28.1
1.73	----	----	7.11	9.21	13.9	18.7	28.3
1.74	----	----	7.17	9.29	14.1	18.9	28.6
1.75	----	----	7.23	9.38	14.2	19.0	28.8
1.76	----	----	7.29	9.46	14.3	19.2	29.1
1.77	----	----	7.36	9.54	14.4	19.4	29.3
1.78	----	----	7.43	9.62	14.6	19.6	29.6
1.79	----	----	7.48	9.70	14.7	19.7	29.9
1.80	----	----	7.54	9.79	14.8	19.9	30.1
1.81	----	----	7.61	9.87	15.0	20.1	30.4
1.82	----	----	7.68	9.95	15.1	20.2	30.7
1.83	----	----	7.74	10.0	15.2	20.4	30.9
1.84	----	----	7.81	10.1	15.3	20.6	31.2
1.85	----	----	7.87	10.2	15.5	20.8	31.5
1.86	----	----	7.91	10.3	15.6	20.9	31.7
1.87	----	----	8.00	10.4	15.7	21.1	32.0
1.88	----	----	8.06	10.5	15.8	21.3	32.3
1.89	----	----	8.13	10.5	16.0	21.3	32.5
1.90	----	----	8.20	10.6	16.1	21.6	32.8
1.91	----	----	8.26	10.7	16.2	21.8	33.1
1.92	----	----	8.33	10.8	16.4	22.0	33.3
1.93	----	----	8.40	10.9	16.5	22.2	33.6
1.94	----	----	8.46	11.0	16.6	22.4	33.9
1.95	----	----	8.53	11.1	16.7	22.5	34.1
1.96	----	----	8.59	11.1	16.9	22.7	34.4
1.97	----	----	8.66	11.2	17.0	22.9	34.7
1.98	----	----	8.73	11.3	17.2	23.1	35.0
1.99	----	----	8.80	11.4	17.3	23.2	35.3
2.00	----	----	----	11.5	17.4	23.4	35.5
2.01	----	----	----	11.6	17.6	23.6	35.8
2.02	----	----	----	11.7	17.7	23.8	36.1
2.03	----	----	----	11.8	17.8	24.0	36.4
2.04	----	----	----	11.8	18.0	24.2	36.7
2.05	----	----	----	11.9	18.1	24.3	36.9
2.06	----	----	----	12.0	18.2	24.5	37.2
2.07	----	----	----	12.1	18.4	24.7	37.5
2.08	----	----	----	12.2	18.5	24.9	37.8
2.09	----	----	----	12.3	18.7	25.1	38.1

Table C–2. (Cont'd)

Head, lin (feet)	Discharge, Q, for throat widths, W, of -- 3 inches	6 inches	9 inches	1 foot	1.5 feet	2 feet	3 feet
	cfs	cfs	cfs	cfs	cfs	cfs	cfs
2.10	----	----	----	12.4	18.8	25.3	38.4
2.11	----	----	----	12.5	18.9	25.5	38.6
2.12	----	----	----	12.6	19.0	25.6	38.9
2.13	----	----	----	12.6	19.2	25.8	39.2
2.14	----	----	----	12.7	19.3	26.0	39.5
2.15	----	----	----	12.8	19.5	26.2	39.8
2.16	----	----	----	12.9	19.6	26.4	40.1
2.17	----	----	----	13.0	19.7	26.6	40.4
2.18	----	----	----	13.1	19.9	26.8	40.7
2.19	----	----	----	13.2	20.0	27.0	41.0
2.20	----	----	----	13.3	20.2	27.2	41.3
2.21	----	----	----	13.4	20.3	27.3	41.5
2.22	----	----	----	13.5	20.5	27.5	41.8
2.23	----	----	----	13.6	20.6	27.7	42.1
2.24	----	----	----	13.7	20.7	27.9	42.4
2.25	----	----	----	13.7	20.9	28.1	42.7
2.26	----	----	----	13.8	21.0	28.3	43.0
2.27	----	----	----	13.9	21.2	28.5	43.3
2.28	----	----	----	14.0	21.3	28.7	43.6
2.29	----	----	----	14.1	21.4	28.9	43.9
2.30	----	----	----	14.2	21.6	29.1	44.2
2.31	----	----	----	14.3	21.7	29.3	44.5
2.32	----	----	----	14.4	21.9	29.5	44.8
2.33	----	----	----	14.5	22.0	29.7	45.1
2.34	----	----	----	14.6	22.2	29.9	45.4
2.35	----	----	----	14.7	22.4	30.1	45.7
2.36	----	----	----	14.8	22.5	30.3	46.0
2.37	----	----	----	14.9	22.6	30.5	46.4
2.38	----	----	----	15.0	22.8	30.7	46.7
2.39	----	----	----	15.1	22.9	30.9	47.0
2.40	----	----	----	15.2	23.0	31.1	47.3
2.41	----	----	----	15.3	23.2	31.3	47.6
2.42	----	----	----	15.4	23.3	31.5	47.9
2.43	----	----	----	15.5	23.5	31.7	48.2
2.44	----	----	----	15.6	23.7	31.9	48.5
2.45	----	----	----	15.6	23.8	32.1	48.8
2.46	----	----	----	15.7	23.9	32.3	49.1
2.47	----	----	----	15.9	24.1	32.5	49.5
2.48	----	----	----	15.9	24.2	32.7	49.8
2.49	----	----	----	16.0	24.4	32.9	50.1
2.50	----	----	----	16.1	24.6	33.1	50.4

Table C-3. Parshall flume velocities.

(based on equation C-4)

Parshall flume width "W", (ft)	0.50					0.75					1.00				
Number of grit chambers	1					2					2				
Cot Φ	0.67					0.67					0.67				
Grit chamber width "W", (ft)	------	1.1	1.2	1.3	1.4	------	0.7	0.8	0.9	1.0	------	1.1	1.2	1.3	1.4
H_a	Q	v	v	v	v	Q	v	v	v	v	Q	v	v	v	v
0.1	0.05	0.43	0.39	0.36	0.34	0.09	0.59	0.52	0.47	0.42					
.2	.16	.65	.60	.56	.52	.26	.78	.70	.63	.57	0.35	0.71	0.65	0.61	0.57
.3	.31	.80	.74	.69	.65	.49	.91	.82	.74	.68	.64	.82	.76	.71	.66
.4	.48	.88	.82	.77	.72	.76	.98	.89	.81	.75	.99	.90	.84	.79	.74
.5	.69	.96	.90	.84	.80	1.06	1.02	.94	.86	.80	1.39	.96	.90	.84	.80
.6	.92	1.02	.96	.90	.85	1.40	1.06	.97	.90	.83	1.84	1.02	.96	.90	.85
.7	1.17	1.06	1.00	.94	.89	1.78	1.09	1.00	.93	.87	2.33	1.06	1.00	.94	.89
.8	1.45	1.11	1.04	.99	.94	2.18	1.10	1.02	.95	.89	2.85	1.06	1.02	.96	.91
.9	1.74	1.13	1.07	1.02	.97	2.61	1.11	1.03	.97	.91	3.41	1.11	1.05	1.00	.94
1.0	2.06	1.16	1.10	1.05	1.00	3.07	1.12	1.04	.98	.92	4.00	1.13	1.07	1.01	.96
1.1	2.40	1.19	1.13	1.07	1.02	3.55	1.12	1.05	.99	.93	4.62	1.14	1.08	1.03	.98
1.2	2.75	1.20	1.14	1.09	1.04	4.06	1.12	1.05	.99	.94	5.28	1.16	1.09	1.04	.99
1.3	3.12	1.22	1.16	1.11	1.06	4.59	1.12	1.06	1.00	.95	5.96	1.16	1.10	1.05	1.01
1.4	3.51	1.23	1.17	1.12	1.07	5.14	1.12	1.06	1.00	.95	6.68	1.16	1.11	1.06	1.02
1.5	3.91	1.24	1.18	1.13	1.08	5.71	1.12	1.05	1.00	.95	7.41	1.17	1.12	1.07	1.02
1.6						6.31	1.11	1.05	1.00	.95	8.18	1.18	1.12	1.07	1.03
1.7						6.92	1.11	1.05	1.00	.95	8.97	1.18	1.12	1.08	1.03
1.8						7.54	1.10	1.04	1.00	.95	9.79	1.18	1.13	1.08	1.04
1.9						8.20	1.09	1.04	.99	.95	10.60	1.18	1.13	1.08	1.04
2.0						8.87	1.08	1.03	.99	.95	11.50	1.18	1.13	1.09	1.05
2.1											12.40	1.18	1.13	1.09	1.05
2.2											13.3	1.17	1.13	1.09	1.05
2.3											14.2	1.17	1.12	1.09	1.05
2.4											15.2	1.17	1.12	1.08	1.05
2.5											16.1	1.16	1.12	1.08	1.05

(3) **Venturi flume design formula.**

$$W = \frac{Q}{D(1 + e)v} \qquad \text{(eq C–5)}$$

$$H = \frac{D(1 - r)}{1 - r^{0.667}} \qquad \text{(eq C–6)}$$

$$b = \frac{Q(max)}{3.09H^{1.5}} \qquad \text{(eq C–7)}$$

$$d = H - D \qquad \text{(eq C–8)}$$

$$h = \left[\frac{Q}{3.09b}\right]^{0.667} \qquad \text{(eq C–9)}$$

$$v = \frac{Q}{(h - d)W} \qquad \text{(eq C–10)}$$

in which

Q = flow through grit chamber of flume discharge;

e = permissible divergence from design velocity (fps);

r = ratio of minimum rate of flow to maximum rate of flow; and other symbols are as previously stated or as indicated in figure C—1**c**.

b. Example. Assuming that the diameter of the inlet sewer is 2 feet, that the average rate of sewage flow is 1.67 cfs, that the maximum rate of flow and as high as practicable over the entire range of flows. Assume a depth in the effluent channel of 1.5 feet at maximum rate of flow. Then the required width (w) would be 5 ÷ (1.5 × 2) = 1 ft. 8 in. The designs for various types of control sections and grit chamber cross-sections are as follows:

(1) **Proportional weir**. As the depth (h) in the chamber above the weir crest cannot exceed 2 feet without submerging the crown of the inlet sewer, let h = 1.75 for first trial. Determine W by substitution in equation C-1. Then,

$$W = \frac{5 \text{ cfs}}{1 \text{ fps} \times 1.75 \text{ ft.}} \quad 2.86 \text{ ft.}$$

$$\text{The design value of } h = \frac{5}{(1)\,(2.86)} \quad 1.77 \text{ ft.}$$

To prevent appreciable divergence of the velocity from 1 fps when the flows are low, the depth (d) of the rectangular section of the weir opening should be minimum practicable. As indicated by equations C-2 and C-3, this depth is a function of b and x. The breadth (b) is limited by the width of the effluent channel (in this case, 1 ft. 8 in.) and x is limited by the size of solids to be passed (in this case, about 3 inches, assuming that a bar screen will be ahead of the grit chamber). Try d 0.15 and solve for b in equation 2. Then,

$$Q = 5 = 0.61(0.15)^{0.5}b(64.4)^{0.5}(1.77 - \frac{0.15}{3});$$

therefore, b = 1.53 (satisfactory). For various values of y, values of y/d are determined and the corresponding values of x/b are taken from the table; the values of x are then determined as follows:

y (ft)	y/d	x/b	x (ft)
0.00	0	1.000	1.530
0.15	1	0.500	0.765
0.30	2	0.392	0.600
0.45	3	0.333	0.510
0.60	4	0.295	0.452
0.90	6	0.247	0.378
1.50	10	0.195	0.298
1.65	11	0.186	0.285

The above tabulation indicates that the trial design is satisfactory.

(2) **Bar screen**. (Refer to para 10-2**b**.)

Assumptions:

Maximum daily flow = 4 mgd;

Maximum storm flow = 7 mgd;

Maximum allowable velocity through bar rack for maximum daily flow = 2 fps.

Then, using the design procedure in the preceding paragraph:

Maximum daily flow = 4 × 1.547 = 6.188 cfs;

Maximum storm flow = 7 × 1.547 = 10.829 cfs.

Since Q = Av, 6.188 = A × 2, and the net area A through the bars is 3.094 sq. ft. For a maximum allowable velocity through the bar rack of 3 fps during maximum storm flow, the net area through the bars must be 10.829/3 = 3.61 sq. ft. The gross area will be based on the larger of the two net areas (in this case, 3.61 sq. ft.). A rack consisting of 2-inch × 5/16-inch bars, spaced to provide clear openings of 1 inch, has an efficiency of 0.768 (table 10-1), yielding:

$$\text{Gross area} = \frac{3.61}{0.768} = 4.70 \text{ sq. ft.}$$

(3) **Wastewater depth**. The channel width in this case might be established at 3 feet; in which case, the water depth would be 4.70/3.0 = 1.57 feet. This is a theoretical water depth which may be affected by subsequent plant units. For instance, a grit chamber may follow the screen chamber and be of such design that a head may build up, the effect of which would be observed in the screen chamber. This is particularly true if the sewage flow by pipeline from the screen chamber to the grit chamber and the flow in the grit chamber are subject to controlled velocity. A screen chamber may be followed immediately by a wet well; in which case the sewage depth, especially at the low flows, would be less than that computed. In fact, it is sometimes necessary to increase the sewage depth by installing stop planks across the channel behind the screen. The planks serve as a weir, and the head built over this impromptu weir further serves to increase the sewage depth in the channel. The head loss through a bar rack is computed from the formula:

$$h = \frac{V^2 - v^2}{2g} \times \frac{1}{0.7} = \frac{V^2 - v^2}{45} \qquad \text{(eq C–11)}$$

where

h = head loss in feet;
V = velocity above rack;
v = velocity above rack;
g = acceleration due to gravity (32.2 ft/sec/sec);

or

h = $0.0222 (V^2 - v^2)$.

Again making use of Q = Av,

$$V = 10.829/4.70 = 2.3 \text{ fps};$$

therefore,

$$\begin{aligned} h &= 0.222\ (3^2 - 2.3^2) \\ &= 0.222 \times 3.7 \\ &= 0.082 \text{ ft, or approximately 1 inch.} \end{aligned}$$

If the screen is half plugged with screening, leaves and other debris:

From Q = Av, the area is directly proportional to the velocity. In other words, if the area is cut in half, the velocity must double. The head loss, therefore, is:

$$\begin{aligned} h &= 0.222\ (6^2 - 2.3^2) \\ &= 0.222 \times 30.7 \\ &= 0.682 \text{ ft, or approximately } 8\tfrac{1}{4} \text{ inches.} \end{aligned}$$

The increase in head loss is over one-half foot as the screen becomes half plugged. The need for accurate control of the cleaning cycle and protection against surge loads is thus demonstrated.

(4) **Parshall flume.** From inspection of table C-13, it is readily obvious that by using two grit channels with bottom width of 0.75 ft and with sides sloping at an angle whose cotangent is 0.67, a Parshall flume with a throat width of 0.75 ft would control the velocity within the specified limits. The table of discharge for Parshall flumes (table C-2) indicates that for flows of 0.67, 1.67 and 5.0 cfs, the values of H_a are 0.34, 0.67 and 1.38, respectively. By appropriate substitution in equation C-4, the corresponding velocities are found to be 0.91, 1.04 and 1.08 fps.

(5) **Venturi flume**. In this case, design for D = 2.0. Then W, H, b and d are determined by substituting (in eqs C-S through C-8, respectively) as follows:

$$W = \frac{5.0}{2(1 + 0.10)v} = 2.27$$

By setting v = 1.0 fps minimum velocity,

$$r = 0.67 \div 5.0 = 0.134$$

$$H = \frac{2(1 - 0.134)}{1 - (0.134)^{0.667}} = 2.35$$

$$b = \frac{5}{3.09\ (2.35)^{1.5}} = 0.45$$

$$d = 2.35 - 2.0 = 0.35$$

For various values of Q, the values of h and v (determined from eqs C-9 and C-10) are as follows:

Q	h	v
5.00	2.35	1.10
4.00	2.03	1.05
3.0	1.67	1.00
2.00	1.27	0.95
1.67	1.13	0.94
1.00	0.80	0.98
0.67	0.62	1.10

This tabulation indicates that for values of Q varying between 0.67 cfs and 5.0 cfs, the velocity in the grit chamber would not vary more than 10 percent from 1.0 fps and that the design would be satisfactory in this respect.

c. Discussion The text states that for proper operation of a proportional weir, complete free-flow conditions must exist below the weir crest. Therefore, in this case, the depth of the effluent channel below the weir crest would have to be 1.5 ft, the maximum liquid depth in the effluent channel. With a Parshall flume (according to the text), the permissible submergence is 65 percent of H_a or, in this case, 0.65 × 1.38 = 0.9 ft. The effluent channel floor would have to be 1.5 - 0.9 = 0.6 ft below the floor of the approach to the Parshall flume. For a Venturi flume (according to the text), the head loss to be provided for should not be less than H/3 or, in this case, 2.35/3 = 0.78 ft. The crest could, therefore, be submerged 2.0-0.78 = 1.22 ft and the effluent channel floor could be 1.5-1.22 = 0.28 ft below the crest. However, to satisfy the design with respect to d, the depth of the effluent channel would have to be 0.35 ft below the crest. The foregoing analysis indicates that the three devices would require head losses of 1.5, 0.60 and 0.35 ft, respectively. Consideration of these losses and local conditions would determine which of the three designs should be used.

C-2. Mechanical flocculation. (Refer to para 10-6b.)

a. Design requirements and criteria. Design a mechanical flocculator as a part of chemical precipitation to handle a flow rate of 4 mgd. The following conditions apply:

Temperature = 20°C (68°f);
Paddle-tip speed = 1.2 fps;
Coefficient of drag of paddles = 1.8.

b. Calculations and results.

(1) Using a detention time of 20 minutes, determine tank volume.
Volume = flow rate × detention time

$$= \left[4 \times 10^6 \frac{\text{gal}}{\text{day}} \times 0.1337 \frac{\text{cu ft}}{\text{gal}}\right] \times \left[20 \text{ min} \times \frac{1 \text{ day}}{1{,}440 \text{ min}}\right]$$

$= 7{,}427.7$ cu ft; use 7,430 cu ft.

(2) Using a depth of 10 ft, determine tank dimensions. Width is usually set by standard sludge removal equipment sizes. A width of 12 ft will be used here.

$$\text{Length} = \frac{\text{Surface area}}{\text{Width}} = \frac{742.7 \text{ sq ft}}{12 \text{ ft}} = 61.9\text{; use 62 ft.}$$

A tank size of 12 ft × 62 ft would be appropriate here and would produce a satisfactory length-to-width ratio of about 5.4:1.

(3) A major design factor in flocculator design is mean velocity gradient (G), measured in ft/sec ft. A typical value of 30 ft/sec ft will be used in this case.

(4) The theoretical power requirement is calculated by using the following formula (derived from para 10-6c):

$$P = MG^2V \qquad \text{(eq C-12)}$$

where

M = absolute fluid viscosity, lb force sec/sq ft (For water, at 20°C, $M = 2.1 \times 10^{-5}$);
V = tank volume;
P = power, ft lb/sec;

in this case,

$$P = (2.1 \times 10^{-5} \text{ lb force sec/sq ft})(30 \text{ ft/sec ft})^2(7{,}430 \text{ cu ft}) = 140.4 \text{ ft lb/sec.}$$

Convert this to horsepower:

$$Hp = P \times \frac{1\ hp}{550\ ft\ lb/sec} = \frac{140.4}{550} = 0.26;\ use\ 0.3hp.$$

(5) Determine the paddle area from the following formula:

$$A = \frac{2P}{C_D v^3}$$

where

C_D = dimensionless coefficient of drag (= 1.8);

= mass fluid density, lb/cu ft/g (1.94 $\frac{lb-sec^2}{ft^4}$ 2at 20°C);

v = relative relocity of paddles in fluid, fps (assume to be 0.75 times paddle-tip speed) = 0.75 × 1.2 fps;

= 0.9 fps, with paddle-tip speed of 1.2 fps;

in this case,

$$A = \frac{2(160.3\ ft/lb\ sec)}{1.8(1.94 lb.-sec^2/ft^4)(0.9ft/sec)^3}$$

$$= 125.9\ sq\ ft.$$

C-3. Sedimentation. (Refer to para 11-2.)

a. Design requirements and criteria. Design a sedimentation unit to provide settling for a sewage flow rate of 4 mgd, with suspended solids concentration of 300 mg/L. The following conditions apply:

Surface loading rate = 600 gpd/sq ft;
Suspended solids removal = 60%;
Sludge solids content = 4%;
Sludge specific density = 1.02.

b. Calculations and results.

(1) Calculate total tank surface area:

$$Surface\,Area = \frac{Flow\,Rate}{Surface\,Loading.Rate} = \frac{4{,}000{,}000\ gpd}{600\,gpd/sq\,ft} = 6{,}666.7;\ use\ 6{,}670\,sq\,ft.$$

(3) Using a depth of 8 ft, calculate total volume:

V = 8 × 6,670 = 53,360 cu ft.

(3) This volume can be divided among three rectangular tanks (in parallel), 20 ft wide and 120 ft long, with a satisfactory length-to-width ratio of 6:1. Two circular tanks (in parallel), 35 ft in diameter, would also be suitable. This will provide flexibility of operation during routine or emergency maintenance.

(4) Calculate weir length requirement, assuming 3 rectangular tanks and allowable weir loading rate of 15,000 gpd/linear ft.

$$Design\ flow/tank = \frac{Total\ \ Flow}{3} = \frac{4{,}000{,}000\ gpd}{3} = 1{,}333{,}333\ gpd;$$

$$Weir\ lenth/tank = \frac{1{,}333{,}333\ gpd}{15{,}000\ gpd/lin\ ft} = 89\ lin\ ft.$$

(5) Comnplete weight of solids removed, assuming 60% removal:

Weight removed = 4 mgd × 300 mg/L × 0.60 = 6,000 lb/day;
therefore, 1,500 lbs are removed per 1 mpd flow.

(6) Calculate sludge volume, assuming a specific gravity of 1.02 and a moisture content of 96% (4% solids):

$$\text{Sludge volume} = \frac{6{,}000 \text{ lb/day}}{1.20 (62.4 \text{ lb/cu ft}) (0.04)} = 2{,}360 \text{ cu ft/day} (@4 \text{ mgd}) = 17{,}700 \text{gpd.}$$

(7) Sludge handling in this example consists of removing sludge manually from settling tank sludge hopper, using a telescoping drawoff pipe which discharges the sludge into a sump from which it is removed by a sludge pump (or pumps). Assume that the sludge will be wasted every 8 hours and pumps for ½-hour to the digester.

$$\text{Sludge sump capacity} = \frac{\text{daily sludge volume}}{\text{Number of wasting periods per day}} = \frac{2{,}360 \text{ cu ft}}{3} = 787 \text{ cu ft } (5{,}900 \text{ gal}).$$

Increase capacity 10 percent to compensate for scum removal volumes:

$$\text{Sludge pumping capacity} = \frac{\text{Sludge and scum volume/wasting period}}{\text{30 minutes pumping/wasting period}} = \frac{6{,}500}{30 \text{ min}} = 217; \text{ use } 220 \text{ gpm.}$$

C-4. Chemical precipitation. (Refer to para 11-5.)

a. Design requirements and criteria. Calculate the sludge production, using chemical addition in primary sedimentation. Assume that addition of 60 lbs of ferrous sulfate and 700 lbs/mil gal of lime yields 70 percent suspended solids removal under the following conditions:

Flow rate = 4 mgd;
Suspended solids concentration = 300 mg/L.

The reactions that will occur are as follows:

$$FeSO_4.7H_2^0 + Ca(HO_3) Fe(HCO_3)_2 + CaSO_4 + 7H_2O$$

If lime in the form of $Ca(OH)_2$ is added, the following reaction occurs:

$$Fe(HCO_3)_2 + 2Ca(OH)_2 + Fe(OH)_2 + 2CaCo_3 + 2H_2O$$

The ferrous hydroxide is next oxidized to ferric hydroxide by the dissolved oxygen in the sewage:

$$Fe(OH)_2 + O_2 + 2H_2O\ 4Fe(OH)_3.$$

The reaction of quicklime with water alkalinity and carbon dioxide:

$CaO + H_2O\ CA(OH)_2$ hydrated lime;
$Ca(OH)_2 + CO_2\ CaCO_3 + H_2O$;
$Ca(OH)_2 + Ca(HCO_3)_2\ 2CaCo_3 + 2H_2O$.

b. Calculations and results. All interim calculations are computed on the basis of a flow volume of 1 mil gal.

(1) Determine the weight of suspended solids removed:

$$\text{Solids weight} = (0.70)(300 \text{ mg/L}) \frac{8.34 \text{ lb/mil gal}}{\text{mg/L}} = 1{,}750 \text{ lb/mil gal;}$$

(2) Determine weight of ferric hydroxide formed from ferrous sulfate:

$$Fe(OH)_3 \text{ weight} = 60 \text{ lb } (FeSO_4 7H_2O) \frac{106.9^{\text{Mol Wt } (Fe(OH)^3)}}{278_{\text{Mol Wt } (FeSO_4 7H_2O)}} = 23 \text{ lb/mil gal.}$$

(3) Determine weight of $CaCO_2$ formed in reacting with SO_4 hardness:

$$CaCO_3 \text{ weight} = \left[60 \text{ lb } (FeSO_4 7H_2O) \frac{112}{278} \right]_{\text{Mol Wt } (FeSO_4 7H_2O)}^{2 \times \text{Mol Wt (CaO)}} \times \left[\frac{100}{56} \right]_{\text{Mol Wt (CaO)}}^{\text{Mol Wt } (CaCO_2)} = 43 \text{ lb/mil gal.}$$

$CaCO_3$ formed in reacting with CO and Ca $(HCO_3)_2$:

$$CaCO_3 = \left[700 \text{ lb (CaO)} - 43 \text{ lb (CaCO3) } \frac{56}{100} \right]_{\text{MW } (CaCO_3)}^{\text{MW } (CaCO_3)} \times \left[\frac{300}{112} \right]_{2 \times \text{MW (CaO)}}^{3 \times \text{MW } (CaCO_3)} = 1{,}810 \text{ lb/mil gal.}$$

Solubility of $CaCO_3$(25 mg/LL):

$$CaCO_3 \text{ dissolved} = 25 \text{ mg/L} \times 8.34 \frac{\text{lb/mil gal}}{\text{mg/L}} = 208 \text{ lb/mil gal;}$$

Total $CaCO_3$ weight 43 + 1,810 - 208 = 1,645 lb/mil gal.

Sum total solids weight:

Total solids weight = 1,750 (SS) + 23 ($Fe(OH)_3$) + 1,645 ($CaCO_3$) = 3,418 lb/mu gal.

At a flow rate of 4 mgd, the total solids weight becomes (3,418 lb/mu gal) × (4 mgd) = 13,672 lb/day.

Calculate sludge volume, assuming an overall specific gravity of 1.06 and a moisture content of 93 percent (7 percent solids):

$$\text{Sludge volume} = \frac{3{,}418 \text{ lb/mil gal}}{1.06 \ (62.4 \text{ lb/cu ft})(0.07)} = 738 \text{ cu ft/mil gal} = 2{,}952 \text{ cu ft/day.}$$

C-5. Single stage stone-media trickling filters. (Refer to para 12-2.)

a. Design requirements and criteria. Design a trickling filter to treat 2 mgd of primary settled effluent under the following conditions:

Raw wastewater BOD_5 = 250 mg/b;
Primary clarifier BOD_5 removal efficiency 30 percent;
Required effluent BOD_5 = 30 mg/b;
Design temperature = 20°C;
Design without and with recirculation.

b. Calculations and results.

(1) **Design without recirculation.**

Primary treated effluent BOD = 250 (1 - 0.3) = 175 mg/L;

$$\text{Required trickling filter efficiency} = \frac{175 - 30}{175} = 0.83$$

Since the design temperature is 20°C, no temperature correction is required as per paragraph 12-2**e**.

$$\text{BOD loading to filter} = (2.0\text{mgd}) \times (175\text{mg/L}) \times \frac{8.34 \text{ lb/mil gal}}{\text{mg}}/\text{L} = 2{,}930 \text{ lb/day.}$$

Assume a practical filter depth of 6 ft for stone-media filters. Apply NRC formula (para 12-12**f**[1]).

$$E = \frac{1}{1 + 0.0085(W/VF)^{0.5}} \qquad \text{(eq C-14)}$$

resulting in

$$0.83 = \frac{1}{1 + 0.0085\ (2.920/V)^{0.5}}$$

Solving for the volume:
V = 5.0 acre ft = 218,760 cu ft.

With a filter depth of 6 ft, filter area (A = 218,760/6) 36,500 sq ft; therefore, to allow for plant flexibility and partial treatment should a filter require maintenance, a minimum of 2 filters should be provided.

$$\text{Filter area (each)} = \frac{36{,}460}{2} = 18{,}200\ \text{sq ft}$$

$$A = \frac{D^2}{4} \text{ or } D = \left[\frac{A \times 4}{\ }\right]^{1/2}$$

$$\text{Therefore } D = \left[\frac{18{,}200 \times 4}{\ }\right]^{1/2} = 152 \text{ ft (each filter); use 155 ft.}$$

(2) **Design with 1:1 recirculation.**

$$\text{Recirculation factor } F = \frac{1 + R}{(1 + 0.1R)^2} = 1.65 \qquad \text{(eq C-15)}$$

$$E = \frac{1}{1 + 0.0085\ (W/VF)^{0.5}}$$

$$0.83 = \frac{1}{1 + 0.0085\left[\frac{2{,}920}{1.65V}\right]^{0.5}}, \text{ or}$$

$$V = 3.04 \text{ acre feet} = \frac{132{,}610}{6} = 22{,}100 \text{ sq ft; again use 2 filters.}$$

Filter diameter D = 119 ft; use 120 ft (each filter).

(3) **Design with 1:2 recirculation**.

$$\text{Recirculation factor } F = \frac{1 + 2}{1 + 0.1 \times 2)^2}$$

$$0.83 = \frac{1}{1 + 0.0085\left[\frac{2{,}920}{2.08V}\right]^{0.5}}$$

V = 2.4 acre ft.

$$\text{Filter area} = \frac{2.4 \times 43{,}560}{6} = 17{,}500 \text{ sq ft; use 2 filters.}$$

Filter diameter = 105 ft (each filter).

c. Pumps. Recirculation pumps will be sized to provide constant rate recirculation. Vertical-shaft, single suction units with motors mounted on top of the pumps, installed in a dry well or on an upper floor, will be used. Each pump will be provided with its individual pipe connection to the wet well.

C-6. Two stage stone-media trickling filters. (Refer to para 12-2.)

a. Design requirements and criteria. Design a two-stage trickling filter to treat 3.0 mgd of primary settled effluent, assuming the following conditions.

Raw wastewater BOD_5 = 250 mg/L;
Primary clarified BOD removal efficiency = 30 percent;
Required effluent BOD_5 = 30 mg/L;
Design temperature = 20°C.

b. Calculations and results.

Primary clairfier effluent BOD = 250 (1 - 0.3 = 175 mg/L;

Overall trickling filter efficiency $= \dfrac{175 - 30}{175} = 0.828$;

Filter depth = 6 ft;

Recirculation = 1:2.

Assuming that the first stage filter efficiency = 75 percent:

Overall efficiency $= 0.828 = E_1 + E_2 (1 - E_1) = 0.75 + E_2 (1 — 0.75)$ $E_2 = 0.31$;

Recirculation factor $F = \dfrac{1 + R}{(1 + 0.1R)^2} = \dfrac{1 + 2}{1 + 0.1 \times 2)^2} = 2.08$;

Organic loading to first stage filter,
W = 3.0 mgd x 8.34 x 250(1 — 0.30) = 4,379; use 4,380 lb/day.

Now, using the NRC formula:

$$E_1 = \frac{1}{1 + 0.085\left[\dfrac{W}{VF}\right]^{0.5}}$$

$$0.75 = \frac{1}{1 + 0.0085}\left[\frac{4,380}{2.08V}\right]^{0.5}$$

V = 1.37 acre ft;

Filter area $= \dfrac{1.35 \times 43,560}{6}$ = 9,946 sq ft; use 2 filters, 5,000 sq ft each.

Filter diameter = 80 ft, use 80 ft (each filter).

Design second stage filter:
Organic loading, W = W (1 - 0.75);
= 4,380 (1 - 0.75);
= 1,095 lb/day.

$$E_2 = \frac{1}{1 + 0.0085\left[\frac{W}{VF}\right]^{0.5}}$$

$$0.31 = \frac{1}{1 + 0.0085\left[\frac{1,085}{2.08V}\right]^{0.5}}$$

V = 0.122 acre ft;

Filter area A = $\frac{0.122 \times 43,560}{6}$ = 886 sq ft; use 1 filter.

Filter diameter D = 33.6 ft; use 35 ft filter.

The design of a two—stage system requires careful economic considerations to insure that minimal filter volume is required and that proper recirculation rates are selected to optimize filter volume and pumping requirements, resulting in the lowest—cost, most flexible system.

C-7. Plastic media trickling filters. (Refer to para 12-2**f**[2].)

a. Design requirements and criteria. Design a plastic media trickling filter to treat 0.2 mgd of primary settled effluent, assuming the following conditions:

Raw wastewater BOD = 250 mg/L;
Primary clarifier BOD removal efficiency = 30 percent;
Required final effluent BOD = 25 mg/L;
Winter design temperature = 10°C.

b. Calculations and results. The formula presented in paragraph 12-2**f**(2) states the following: L_e and L_o in units of mg/L; K_{20} as day^{-1}; D as ft; and Q as gpm/sq ft.

(eq C-16)

$$\frac{L_e}{L_o} = \exp\left[\frac{-(O^{t-20})K_{20}\ D}{Q^n}\right]$$

The raw waste BOD = 250 mg/b; 30 percent removal is obtained in the primary clarifiers; therefore:

L_o = 250 × (1 — 0.3) = 175 mg/L;

Le = 25 mg/L, based on local, state or Federal requirements.

Assume a filter depth of 12 ft:

$$\frac{25}{175} = \exp\left[\frac{-[1.0353(^{10-20})](0.088)(12)}{Q^{0.67}}\right]$$

therefore,

$$0.143 = \exp\left[\frac{-0.75}{Q^{0.67}}\right]$$

Taking the natural logarithm of both sides of the equation results as follows:

$$\ln\ 0.143 = \frac{0.75}{Q^{0.67}}\ \ln\ e$$

$$-1.94 = \frac{0.75}{Q^{0.67}} \times 1.0$$

$$Q^{0.67} = 0.75/1.94;$$
$$Q^{0.67} = 0.387;$$
$$Q^{0.67} = (0.387)^{1.49};$$
$$= 0.243 \text{ gpm/sq ft.}$$

Required surface area can be computed by:

$$\text{Surface Area} = \frac{\text{Flow}}{Q}\ Q$$

$$\text{Surface area} = \frac{\dfrac{0.2 \times 10^{6} \text{ gpd}}{1{,}440 \text{ min/day}}}{0.243 \text{ gpm/sq ft}} = 5{,}715;\ \text{use } 5{,}720 \text{ sq ft.}$$

$$\text{Since } A = \frac{D^{2}}{4}$$

$$\frac{D^{2}}{4} = 5{,}720;$$

$$D = \left[\frac{5{,}720 \times 4}{}\right]^{0.5} = 85.4;\ \text{use } 86 \text{ ft.}$$

Therefore, the filter dimension sould be:

86 ft diameter × 12 ft deep.

Plastic media manufacturers should be consulted to determine the exact proportions of filter media that provide for a minimum of 5,720 sq ft area and 12 ft depth.

C-8. Activated sludge, closed-loop reactor. (Refer to para 13-3.)

a. The following design criteria are for an oxidation ditch with horizontal-shaft rotor aerators, using a single- channel, oval configuration and miltiple units in parallel, operated in an extended aeration mode with nitrification.

Influent requirements:

Q, avg daily flow	= 1.0 mgd;
Q, peak flow	= 2.0 mgd;
BOD_5	= 250 mg/b, typical of Army installations;
TSS	= 250 mg/b, typical of Army installations;
VSS	= 200 mg/b;
TKN	= 25 mg/b (all NH3 N);
P	= 8mg/b;
pH	= 6.5 to 8.5;
Minimum temp	= 10°C;
Maximum temp	= 25°C.

Effluent requirements:

BOD_5 ≤ 10 mg/b;
TKN ≤ 5 mg/b;
NH_3^{-N} ≤ mg/b;
TSS ≤ 20 mg/b.

Waste characteristics:

Y = 0.8 lb solids produced/lb BOD_5 removed;
= total sludge produced;
k_d = 0.05 day^{-1}.

Clarifier characteristics:

Overflow rate ≤ 450 gpd/sq ft @ Q_{avg};
Solid loading ≤ 30 lb/sq ft-day.

Flow divided equally into two units in parallel, or Q = 0.5 mgd average daily flow each; the calculation shown below applies to either unit:

Q_{avg} = 0.5 mgd = 347.2 gpm;

Organic load = 0.5 mgd × 250 mg/b × 8.34 = 1042.5 lb BOD/day.

b. Calculation of oxidation ditch volume.

Use 20 lb BOD/1,000 cu ft—day;

$$V = \frac{1042.5 \text{ lb/day}}{20\text{lb}/1{,}000 \text{ cu ft}-\text{day}} = 52{,}125 \text{ cu ft};$$

$$\text{Hydraulic detention time} = \frac{52{,}125 \times 7.48 \times 24}{0.5 \times 10^6} = 18.75 \text{ hr.}$$

c. Calculation of rotor requirements.

Rotor mixing requirement = 16,000 gal/ft of rotor; recommended by bakeside Equipment Corporation for maintaining a channel velocity of 1.0 fps.

$$\text{Length of rotor} = \frac{52{,}125 \quad 7.48}{16{,}000} = 24.4; \text{ use } 25 \text{ ft.}$$

Oxygenation requirement = 2.35 lb O_2/lb BOD-recommended for domestic sewage. Assume the following operating conditions:

RPM = 72;
Immersion = 8 in.

Oxygenation = 3.75 lb O_2/hr-ft (from fig C-2), using a 42-in diameter MAGNA rotor.

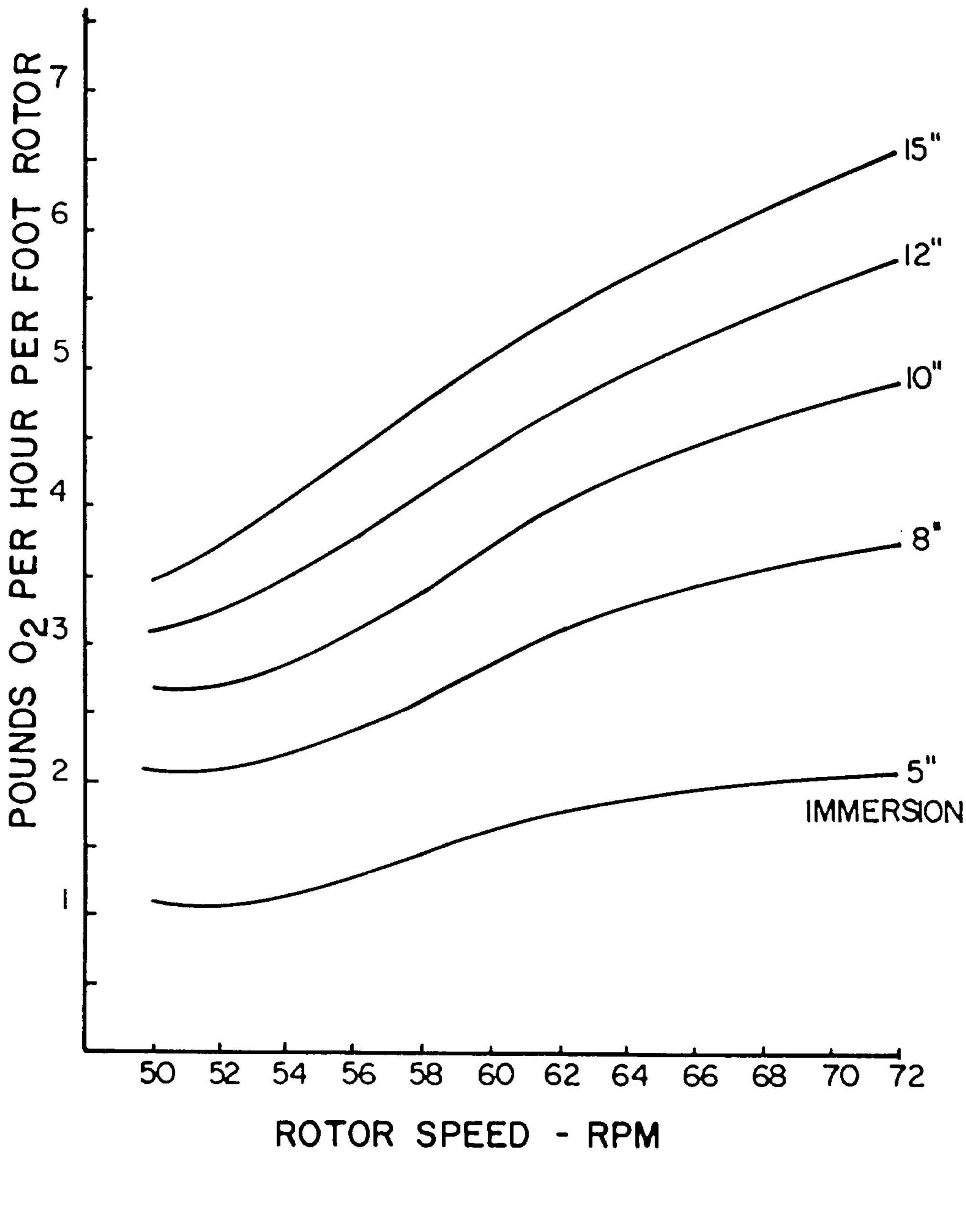


Figure C–2. Rotor aerator oxygenation capacity curve.

$$\text{Length of rotor} = \frac{1042.5 \times 2.35}{24 \times 3.75} = 27.2;\ \text{use 28 ft.}$$

Use two rotors per each unit oxidation ditch or 2- × 14-ft length rotor.

Theoretical oxygen transfer requirement:

$$\text{lb } O_2/\text{hr}-\text{ft} = \frac{1042.5 \times 2.35}{24 \times 28} = 3.65$$

Actual immersion = 8 in.

Power requirement = 0.84 kW/ft of rotor (from fig C-3), using 42-in diameter MAGNA rotor.

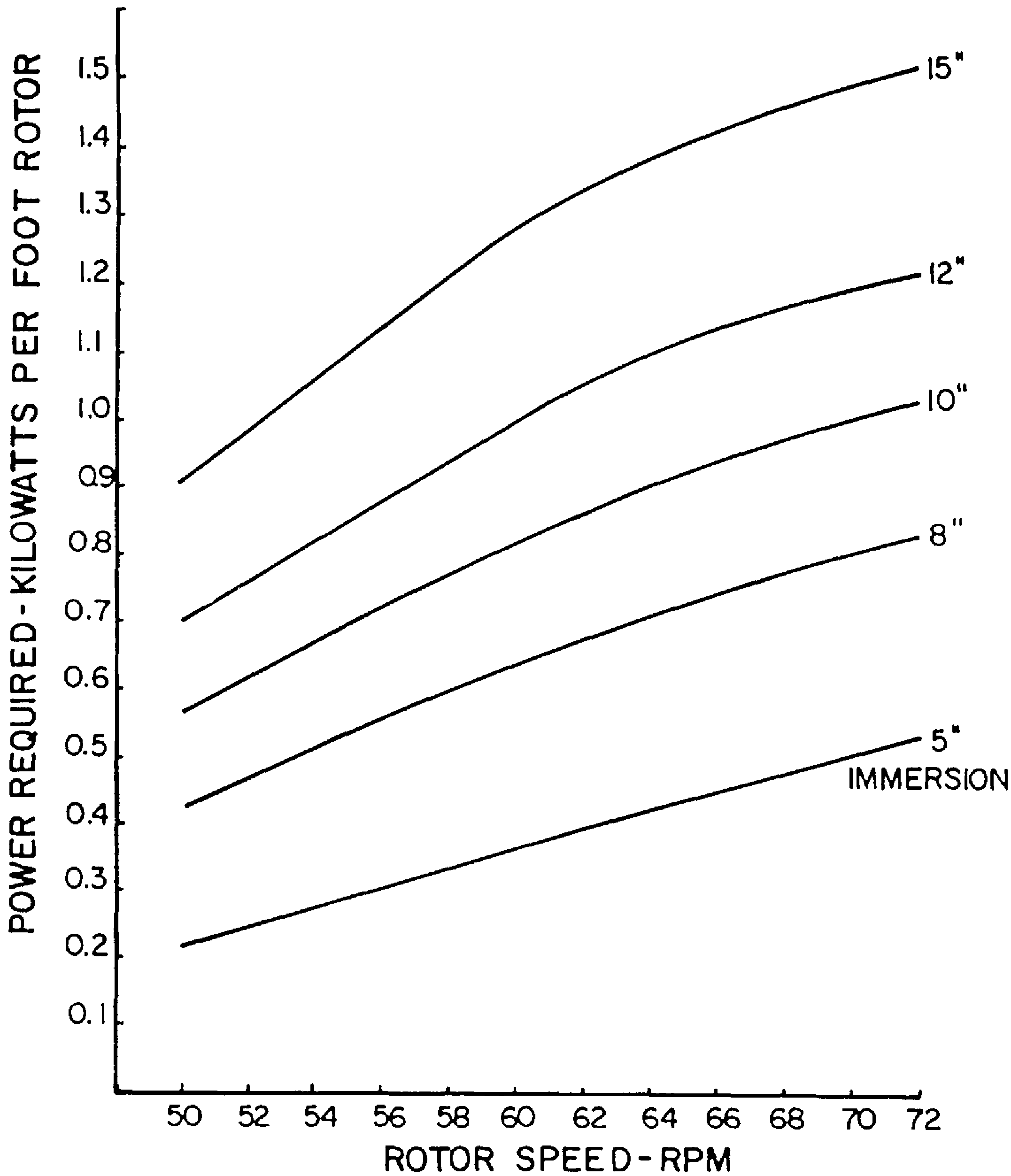


Figure C–3. Rotor aerator power requirements curve.

Aerator brake horsepower requirement = 1.34 × 0.84 × 14 = 15.75 BHP for each rotor.

d. Calculation of motor horsepower. Size all motors to allow at least 1½ inch above the 8-in actual mmersion to allow for peak flows.

Motor horsepower required at (8 + 1.5) = 9.5 in;

$$= \frac{0.99 \text{ BHP} \times 14 \times 1.34}{0.95} = 19.6 \text{ hp;}$$

Use standard horsepower, or 20 hp each.

e. Channel sizing (fig C-4).

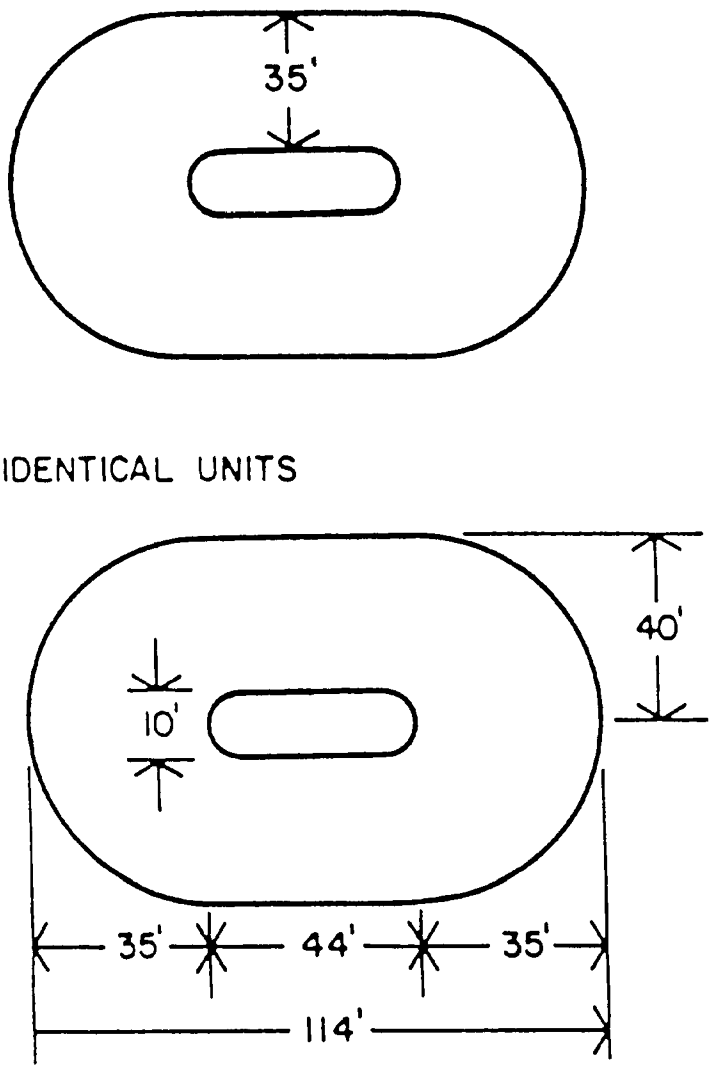


Figure C–4. Closed–loop reactor channel sizing.

Ditch liquid volume = 52,125 cu ft.

Choose:

Depth: 10 ft;
Side wall slope: 45 degrees;
Median strip width: 10 ft;
Ditch flat bottom width = rotor length + 1.0 ft = 14 ± 1 = 15 ft;
Ditch width at water surface = 2 x 10 + 15 = 35 ft;

Ditch cross-section area = $(\frac{15 + 35}{2}) \times 10 = 250 \text{ sq ft};$

Curvature volume = (2)(3.142)(22.5)(250) = 35,325 cu ft (by Theorem of Pappus);
Ditch straight wall volume = 52,125 - 35,325 = 16,800 cu ft;

Total length of the ditch at the waterline = $\frac{16,800}{2 \times 250} = 33.6, \text{ or } 34 \text{ ft};$

Ditch width = 2(35) + 10 = 80 ft;
Total ditch length = 34 + 80 + 114 ft;
Overall ditch dimensions = 114 ft x 80 ft x 10 ft deep.

f. Final clarifier.

One clarifier required for each ditch unit (Spiraflow clarifier).

Overflow rate $\leq$ 450 gpd/sq ft;

Detention time = 3 hr;

Area required = $\frac{.05 \times 10^6}{1 \times 450} 1,111 \text{sq ft};$

Diameter $= \left[\frac{1,111}{0.785}\right]^{0.5} = 37.6, \text{ use } 38 \text{ ft};$

Actual area = $(38)^2$ x 0.785 = 1133.5 sq ft;

Actual overflow rate = $\frac{0.5 \times 10^6}{1133.5} = 441 \text{ gpd/sq ft};$

Volume = $\frac{0.5 \times 10^6 \times 3}{24 \times 7.48 \times 1} = 8356.5 \text{ cu ft};$

Straight wall dept = $(\frac{8356.5}{1133.5}) = 7.4 \text{ ft, use } 8 \text{ ft};$

Actual detention time = $8 \times 1133.5 \times 24 \times 7.48/0.5 \times 10^6 = 3.25$ hr.

g. Check for nitrificaiton in oxidation ditch. Find MLVSS concentration required at standard conditions (20°C, pH8.5) to completely nitrify ammounia shown in figure C-5.

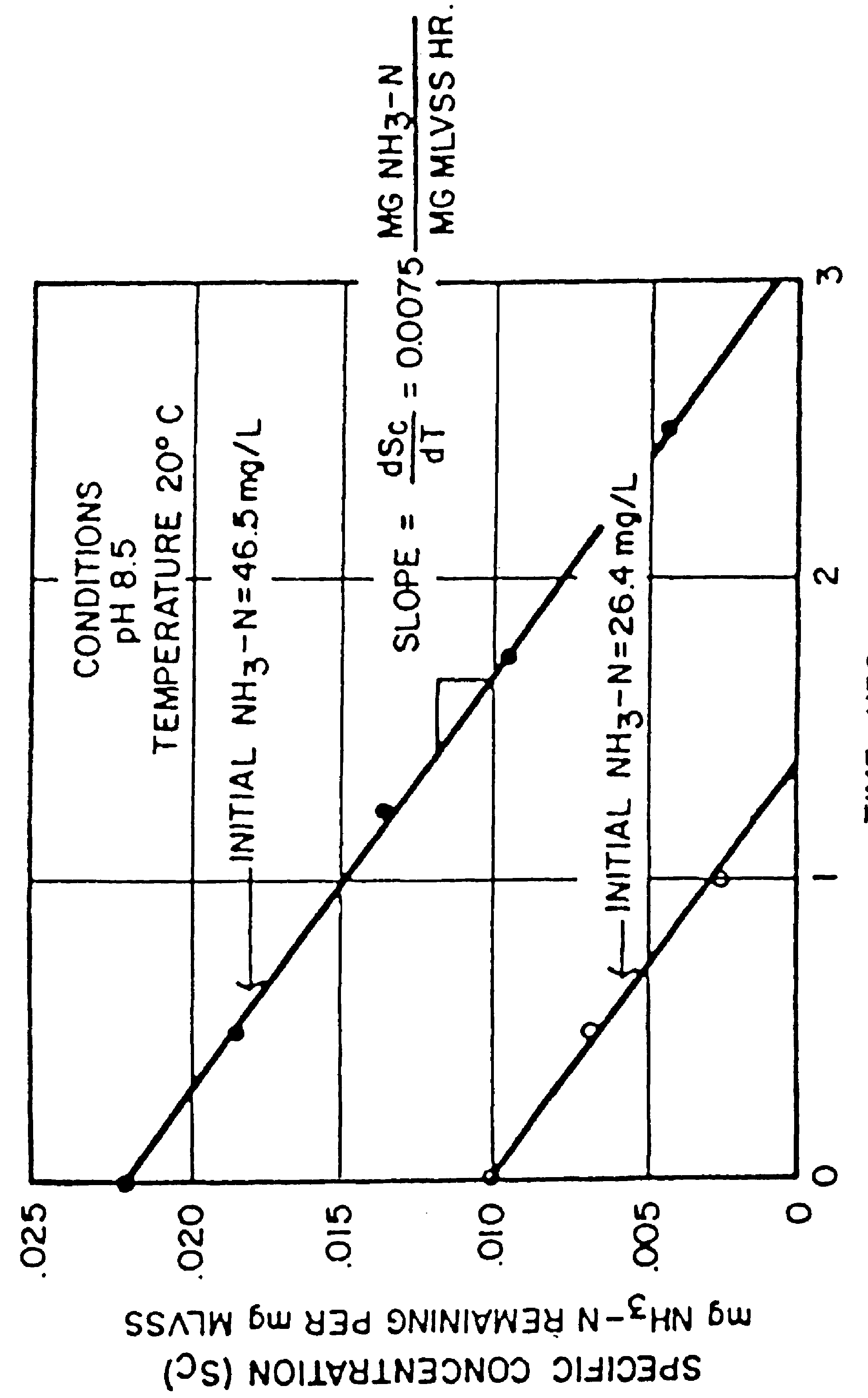


Figure C-5. Ammonia nitrification.

$$\text{MLVSS} = \frac{1}{0.0075} \times \frac{25 \text{ mg/L } NH_3^{-N}}{18.75 \text{ hr.}} = 177.8 \text{ mg/L},$$

where 0.0075 mg NH_3^{-N}/mg MLVSS-hr is the rate of nitrification given in figure C-5.

Change MLVSS concentration at standard conditions to design conditions (10°C, pH 6.5 to 8.5[assume pH is 7.0]), using figures C-6 and C-7 for temperature and pH corrections.

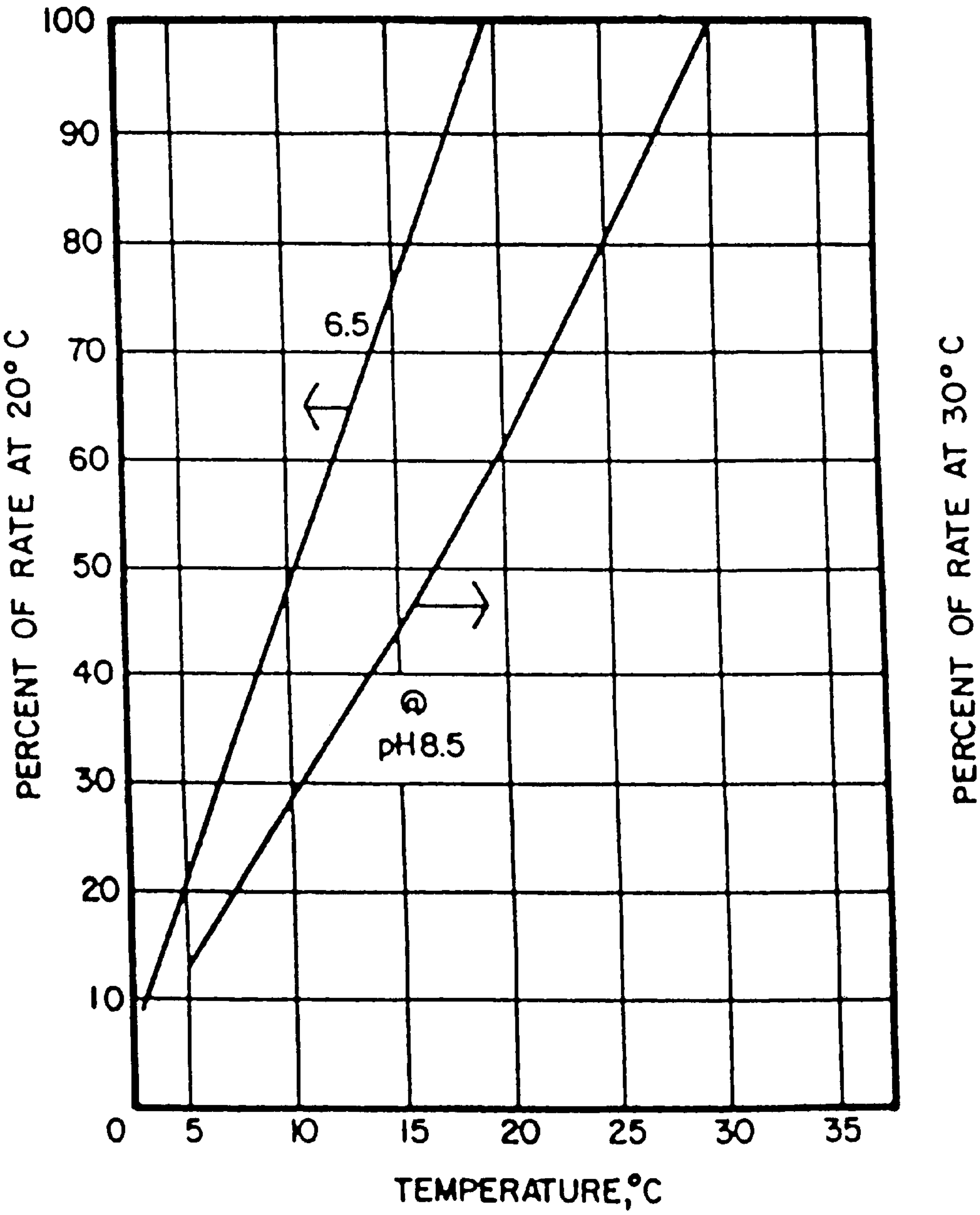


Figure C–6. Nitrification temperature corrections.

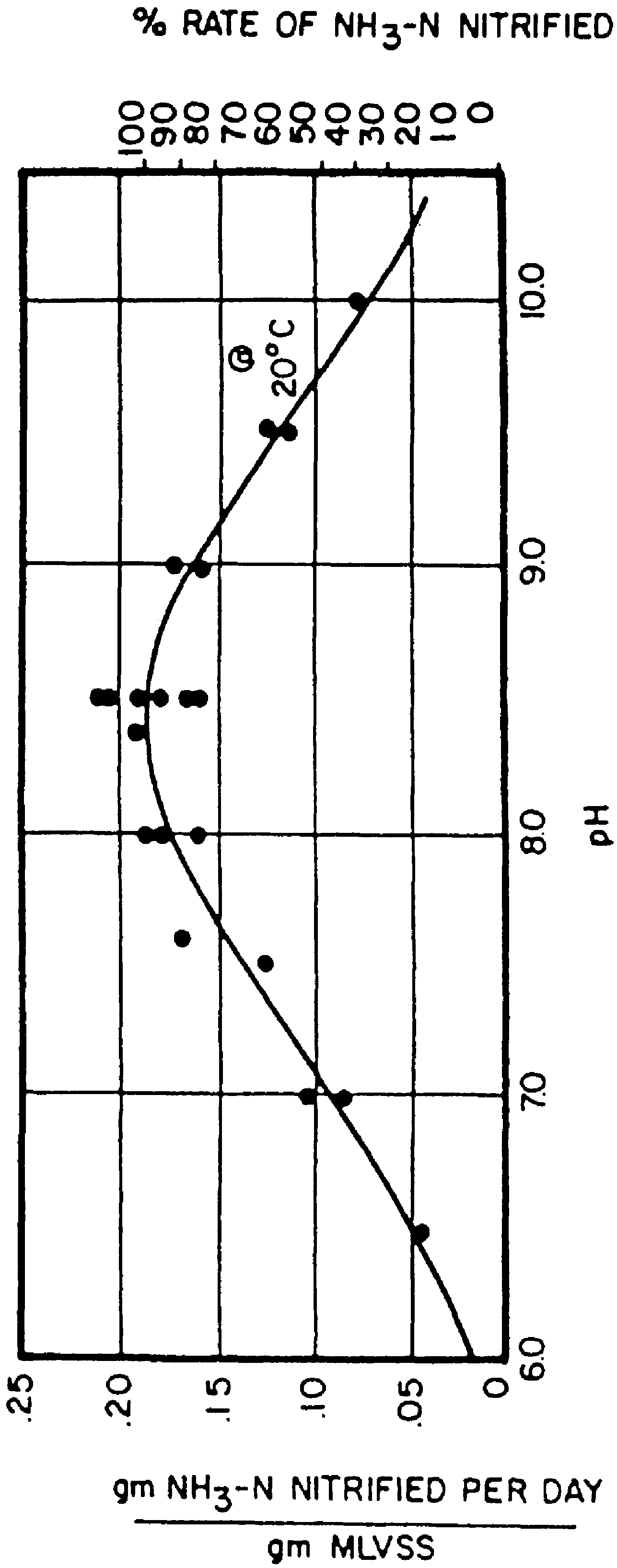


Figure C-7. Nitrification pH corrections.

Temperature correction = 0.46 (from 20°C to 10°C);
pH correction = 0.50 (pH 8.5 to 7.0);

$$\text{MLVSS concentration} = \frac{177.8 \text{ mg/L}}{0.46 \times 0.50} = 774 \text{ mg/L.}$$

This concentration is low; the sludge age would be too short to give a stable performance. To obtain a stabilized sludge:
Use 3200 MLVSS,

$$\text{MLVSS} = \frac{3200}{0.8} = 4000 \text{ mg/L.}$$

Check sludge age (with negligible sludge wasting):

$$c = \frac{\text{MLVSS}}{\text{Y(BOD)}} = \frac{3200}{0.8(250)} = 16 \text{ days.}$$

This sludge age is considered adequate.

h. Check size of final clarifier with solids loading.
Spiraflow clarifier allows 30 lb solids/sq ft-day.

$$\text{Actual solids loading} = \frac{(441 \text{ gpd/sg ft})(8.34)(4000)}{500{,}000} = 29.4 \text{ lb/sq ft}-\text{day} < 30.$$

i. Adjustable oxidation ditch weir. A weir on each oxidation ditch is provided and sized so that the head differential over the weir between the maximum and minimum flow rates is less than 1.25 in.

Maximum flow rate = Q_{peak} + maximum recirculation rate;
Minimum flow rate = Q_{min} + recirculation rate.
For average daily flows between 200,000 gal/day to 1 mgd, the length (L) of the overflow weir is:

$$L = \frac{3.5 \times Q_{avg} \text{ gpm}}{102} = \frac{3.5 \times 347.2}{102} = 11.9, \text{ use 12 ft.}$$

j. Calculate the return sludge flow rate, Q_R:

The equipment manufacturer recommends the following:
Minimum pumping capacity = $0.25 \times Q_{avg}$ = 0.25×347.2 gpm = 87 gpm.
Maximum pumping capacity = $1.0 \times Q_{avg}$ = 1.0×347.2 gpm = 347 gpm.
2 pumps, each with 44 to 174 gpm capacity.

k. Calculate amount of sludge for disposal. The amount of sludge wasted is the same as in the vertical-shaft aerator example since the influent and effluent characteristics and the MLVSS concentrations are identical.
Px = 683.2 lb/day of VSS from 2 oxidation ditches.
Total sludge 933.4 lb/day dry solids from 2 oxidation. ditches.
2 pumps, each with a capacity of 120 gpm for a 52-min/day wasting schedule as in the vertical-shaft aerator example.

l. Sizing of sludge drying beds.

$$\text{BOD population equivalent} = \frac{1 \times 250 \ 8.34}{0.17} = 12{,}265 \text{ persons;}$$

Drying bed area = 12,265 × 1 sq ft/cap = 12,265 sq ft.
Use 10 drying beds, each at 35 ft × 35 ft.
Total area = 12,250 sq ft.

Table C-4 summarizes the design criteria for the horizontal-shaft aerators and multiple-ditch units.

Table C-4. Summary of closed–loop reactor design.

Quantity	Unit	Size
2	Oval–shaped oxidation ditch with 10–ft wide median strip; 45 degree slope. Lining using gunite or shotcrete method of construction.	114 ft × 80 ft, 10 ft deep
4	Horizontal–shaft rotor aerator; 2 ea in oxidation ditch.	14 ft long ea; 16 BHP ea (motor 20 hp)
2	Secondary clarifier.	38 ft dia × 8 ft SWD (Spiraflo clarifier)
2	Return sludge pump.	0.125 to 0.5 mgd ea (44 to 174 gpm)
2	Sludge wasting pump.	120 gpm ea
2	Adjustable oxidation ditch weir.	12 ft ea
10	Sludge drying bed.	35 ft × 35 ft ea

C-9. Microstrainer. (Refer to para 15—4.)

a. Design requirements. Determine the solids loaidng on a microstrainer, treating a secondary effluent containing 15 mg/L suspended solids, at a flow rate of 0.8 mgd.

b. Calculations and results.

(1) Determine screen surface area, using a hydraulic loading of 600 gal/sq ft/hr.

$$\text{Surface area} = \frac{800{,}000 \text{ gal/day} \times \text{day/24 hr}}{600 \text{ gal/sq ft/hr}} = 55.6 \text{ sq ft, use 60 sq ft;}$$

Assuming **b** of the drum surface area is submerged: a drum, 5 ft dia × 3 ft wide, would provide 32 sq ft of submerged screen area.

(2) The solids loading is determined as follows:
Solids loading in lb/mil gal = SS concentration in mg/L×8.34;

$$15 \text{ mg/L} \times 8.34 \frac{\text{lb/mil gal}}{\text{mg/L}} = 125 \text{ lb/mil gal;}$$

125 lb/mil gal × 0.8 mgd = 100 lb/day;

$$\text{Solids loading} = \frac{\text{c100 lb/day}}{32.0 \text{ sq ft}} = 3.1 \text{ lb/sq ft/day.}$$

This loading is high and might require solids removal pretreatment if screen clogging occurs.

The required horsepower is taken from the following tabulation (Source: EPA **Design Manual for Suspended Solids Removal):**

Microstrainer sizes, motors and capacities.

Drive Sizes (ft)		Motors (BPH)		
Diam	Width	Drive	Wash Pump	Approx Range of Capacity (mgd)
5.0	1.0	0.50	1.0	0.1–0.5
5.0	3.0	0.75	3.0	0.3–1.5
7.5	5.0	2.00	5.0	0.8–4.0
10.0	10.0	5.00	7.5	3.0–10.0

C-10. Multi-media filtration. (Refer to para 15—5.)

a. Design requirements. Determine the size of a multi-media gravity filter used in treating a secondary effluent, assuming a hydraulic loading of 5 gpm/sq ft and a flow rate of 0.7 mgd.

b. Calculations and results.

(1) Surface area:

$$\text{Surface area} = \frac{700{,}000 \text{ gal/day} \times \text{day/1,440 min}}{5 \text{ gal/min/sq ft}} = 97.2 \text{ sq ft, use 100 sq ft;}$$

Provide two 50 sq ft filter units.

(2) Depth is determined by the filter media layer depth. A common media depth would be 18 in coal and 6 in sand.

(3) Backwash pumping:

Per filter unit = filter surface areas × backwash rate;
= 50 sq ft × 15 gpm/sq ft;
= 750 gpm.

Provide two 750 gpm pumps (includes 1 standby) for each filter unit.

(4) Backwash water storage tank volume:

Vol = wash rate×waste time period;
= 750 gpm×8 min;
= 6,000 gal/filter unit (mimimum).

Backwash water may be obtained from the chlorine contact basin if its design volume is, as a minimum, slightly greater than the required washwater volume.

(5) Backwash wastewater is to be returned to the primary settling tanks and the return rate cannot exceed 15 percent of the design flow.

Maximum pumping rate = 0.15×700,000 gpd;
= 105,000 gpd;
= 73 gpm.

Provide two 70-gpm waste pumps (includes 1 standby) and provide a wastewater storage sump equivalent to backwash water volume or 6,000 gallons.

Activated carbon adsorption. (Refer to para 15-6.)

a. Design requirements.

Q = 1.5 mgd = 1,042 gpm;
Hydraulic loading = 5 gpm/sq ft (determined from pilot unit or estimated);
Contact time = 30 minutes (determined from laboratory carbon isotherms);
Secondary effluent BOD = 25 mg/L;
Effluent BOD required = 10 mg/L;

lb BOD removed/lb carbon applied $= 0.2 \dfrac{\text{lb BOD rem'd}}{\text{lb carbon}}$ (determined laboratory carbon isotherms);

Carbon type = Calgon Filtrasorb 300 (determined from laboratory carbon isostherms).

b. Design calculations.

(1) Surface area:

$$\text{Area} = \frac{1{,}042 \text{ gpm}}{(5 \text{ gpm/sq ft})} = 208 \text{ sq ft};$$

$$\frac{D^2}{4} = \frac{208}{2} = 104 \text{ sq ft per unit, using 2 units;}$$

D^2 = 132 sq ft;
D = 11.5 ft.

Use inside diameter D = 12 ft;

$$\text{Actual area} = 2\frac{(12)^2}{4} = 226 \text{ sq ft.}$$

$$\text{Actual hydraulic loading} = \frac{1}{226 \text{ sq ft}} = 4.6 \text{ gpm/sq ft.}$$

(2) Bed depth required at 30 min contact time:

$$\text{Volume required} = \frac{(1{,}042\ \text{gpm})(30\ \text{min})}{(7.48\ \text{gpm})(2\ \text{units})} = 2{,}090\ \text{cu ft/unit};$$

$$\text{Bed depth} = \frac{2{,}090\ \text{cu ft/unit}}{113\ \text{sq ft/unit}} = 18.5\ \text{ft per unit, use d} = 20\ \text{ft};$$

$$\text{Actual contact time} = \frac{(20\ \text{ft})(226\ \text{sq ft})(7.48\ \text{gal/cu ft})}{1{,}042\ \text{gpm}} = 32.4\ \text{minutes.}$$

Supply 40 percent expansion room at top for backwashing and 3.0 ft freeboard at bottom:

Actual depth = .4(20’) + 3.0 ft + 20 ft = 31 ft.

(3) Regeneration of carbon:

BOD removal required = 25 mg/L 10 mg/L = 15 mg/L;

$$\text{Carbon exhaustion rate} = 0.2\ \frac{\text{lb BOD removed}}{\text{lb carbon}};$$

BOD removed/day = 15 mg/L (8.34 lb/gal)(l.5 mgd) = 188 lb/day.

$$\text{Carbon required} = \frac{188\ \text{lb BOD removed/day}}{0.2\ \frac{\text{lb BOD removed}}{\text{lb carbon}}} = 940\ \text{lb carbon/day.}$$

$$\text{Lbs carbon required per column per day} = \frac{940}{2\ \text{units}} = 470\ \frac{\text{lb/column}}{\text{day}}$$

Carbon in column at 26 lb/cu ft = (20 ft)(ll3 sq ft)(26 lb/cu ft) = 58,760 lb/column.

$$\text{Regeneration required at interval of} = \frac{58\ 760\ \text{lb/column}}{470\ \frac{\text{lb/column}}{\text{day}}} = 125\ \text{days.}$$

C-12. Phosphorus removal. (Refer to para 15-7.)

a. Design requirements and criteria. Determine what post-secondary mineral treatment for phosphorus removal will meet an effluent requirement of 1 mg/L as P. Assume the following conditions apply:

Wastewater flow = 0.5 mgd;

Influent phosphorus concentration = 12 mg/L;

Treatment will be post secondary;

Influent alkalinity = 300 mg/L as $CaCO_3$.

b. Calculations and results.

(1) Determine lb/day of phosphorus incoming:

$$\text{lb/day} = (12\ \text{mg/L} \times 8.34\frac{\text{lb/mil gal}}{\text{mg/L}} \times (0.5\ \text{mgd})$$

= 50.0 lb/day phosphorus as P.

(2) Determine total dosage rate for mineral addition, using an Al:P weight ratio of 2:1 and an Fe:P ratio of 3:1.

$$\text{Alum dose} = (50 \text{ lb/day Phosphorus} \times \left[\frac{2}{1}\ \frac{\text{Al}}{\text{Phosphorus}}\right] \times \left[\frac{594}{54}\ \frac{\text{Alum}}{\text{Al}}\right]$$

= 1,100 lb/day Alum;
= 265 mg/L as $AL_2(SO_4)_3.14H_20$.

$$FeCl_3 \text{ dose} = (50 \text{ lb/day Phosphorus} \times \left[\frac{3}{1}\ \frac{\text{Fe}}{\text{Phosphorus}}\right] \times \left[\frac{162.35}{56}\ \frac{FeCl_3}{\text{Fe}}\right]$$

= 435 lb/day $FeCl_3$;
= 104 mg/L $FeCl_3$ = 36 mg/L as Fe.

c. Other methods available. The above examples are not to limit the designer's selection of phosphorus removal methods. They all should be investigated to determine the most cost effective. The designer should refer to paragraph 15-7 for guidance.

C-13. Nitrification-denitrification. (Refer to para 15-9.)

a. Design requirements and criteria. Design a nitrification-denitrification system to treat 20 mg/L of influent nitrogen. Use a separate stage configuration with suspended growth denitrification. Assume the following conditions apply:

Wastewater flow rate = 1.2 mgd;
Temperature: 15°C;
Operating pH = 7.8.
Nitrification:
InfluentNH_3^{-N} concentration = 20 mg/L;
MLVSS = 2,000 mg/L;
Recycle = 100% of average flow.
There is no waste sludge.
Denitrification:
MLVSS = 2,000 mg/L;
Recycle = 75%;
Detention time = 2.5 hr.

b. Calculations and results.

(1) Determine loading:

$$NH_3^{-N} \text{ loading} = 20 \text{ mg/L} \times 8.34\ \frac{\text{lb/mil gal}}{\text{mg/L}} \times 1.2 \text{ mgd} = 200 \text{ lb/day } NH_3^{-N}.$$

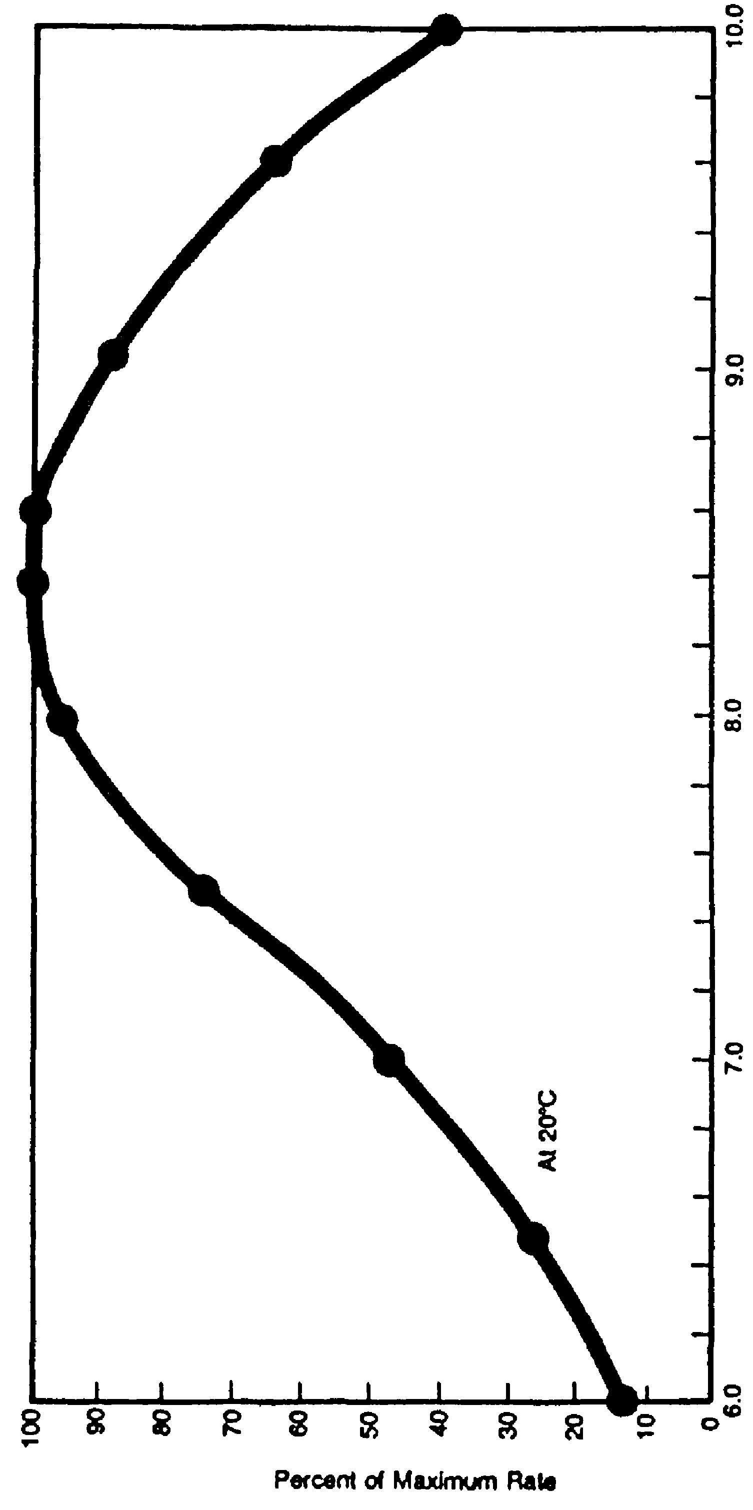
(2) Determine tank volume. Use figure 15-4 to estimate the volumetric NH_3^{-N} loading at 15°C with an MLVSS concentration of 2,000 mg/L:

$$\text{lb/day}\ \frac{NH_3^{-N}}{1{,}000 \text{ cu ft}} = 116.5;$$

$$\text{Tank volume} = 200 \text{ lb/day } NH_3^{-N} \times \frac{1{,}000 \text{ cu ft}}{16.5 \text{ lb/day } NH_3^{-N}} = 12{,}120 \text{ cu ft.}$$

From figure C—8, get the performance efficiency and adjust taqnk volume accordingly: at pH = 7.8, the efficiency is 88 percent.

$$\text{Tank volume} = \frac{12{,}120 \text{ cu ft}}{0.88} = 13{,}773 \text{ cu ft.}$$



Source: Metcalf & Eddy, "Design of Nitrification and Denitrification Facilities" for EPA April 1971

Figure C-8. Nitrification efficiency versus pH.

(3) Determine detention time:

$$\text{Detention time} = \frac{13{,}773 \text{ cu ft} \times 7.48 \text{ gal/hr}}{1{,}200{,}000 \text{ gal/day} \times 1 \text{ day/24 hr}} = 2.06 \text{ hr, use } 2.1 \text{ hr;}$$

(4) Set diffused air supply at 1 scf/gal wastewater treated.

(5) Determine clarifier volume, using an overflow rate of 800 gpd/sq ft and a tank depth of 12 ft:

$$\text{Surface area} = \frac{1{,}200{,}000 \text{ gpd}}{800 \text{ gpd/sq ft}} = 1{,}500 \text{ sq ft;}$$

Volume = (12 ft)×(1,500 sq ft) = 18,000 cu ft;

- 1 tank, 20 ft wide by 80 ft long, would be suitable.

(6) Assuming 90 percent NH_3^{-N} reduction in the nitrification stage, determine NO_3^{-N} and loading applied to the denitrification stage.

(0.90)(200 lb/day NH_3^{-N}) = 180 lb/day NO_3^{-N}

(7) Determine methanol dosage rate, assuming 3 lb methanol per pound nitrate nitrogen removed.

$$\text{Methanol dose} = (180 \text{ lb/day } NH_3^{-N}).\ \frac{3 \text{ lb methanol}}{\text{lb } NO_3^{-N}} = 540 \text{ lb methanol/day}$$

(8) Determine tank volume, using detention time:

Volume = (1,200,000 gal/day × 1 day/24 hr)(2.1 hr)(1 cu ft/7.48 gal) = 14,037 cu ft, use 14,040.

C-14. Anaerobic sludge digestion. (Refer to para 16-6b.)

a. Design requirements and criteria. Determine the digester volume, and gas yield and heat requirement for the anaerobic digestion of combined activated sludge and primary sludge. Assume the following conditions apply:

Sludge amount = 1,200 lb/day;
Sludge solids after thickening = 3%;
Detention time = 15 days;
Volatile matter reduction = 60%;
Temperature = 80°F (in digester);
Gas yield rate = 15 cu ft/lb VSS destructed;
Volatile sludge content = 75%.

b. Calculation and results.

(1) Determine digester volume:

$$\text{Sludge volume} = \frac{1{,}200 \text{ lb/day}}{8.34 \times (0.03)} = 4{,}796, \text{ use } 4{,}800 \text{ gal/day;}$$

$$= 4{,}800 \times \frac{1}{7.48} = 642 \text{ cu ft/day;}$$

Digester volume = 642 cu ft/day × 15 days = 9,630 cu ft.

(2) Determine gas yield:

$$\text{Gas yield} = (15\ \frac{\text{cu ft}}{\text{lb VSS destroyed}}\ 1{,}200 \text{ lb/day} \times 0.75 \times 0.60) = 8{,}100 \text{ cu ft/day} = 5.6 \text{ cfm.}$$

(3) Using a circular tank 20 feet deep and 26 feet in diameter, determine the heat requirement. Given:

Heat Transfer Coefficient	*Temperature*	
Walls = 0.14	(air)	40°F
Floor = 0.12	(ground)	50°F
Roof = 0.16	(air)	40°F

Area

Walls = (26)(20) = 1,634 sq ft;
Floor = $(13)^2$ = 531 sq ft;
Roof = $(13)^2$ = 531 sq ft.

Heat Loss

Walls = (0.14)(1,634)(80 - 40) = 9,150 Btu/hr;
Floor = (0.12)(530)(80 - 50) = 1,908 Btu/hr;
Roof = (0.16)(530)(80 - 40) = 3,392 Btu/hr;

Total Heat Loss = 14,450 Btu/hr = 346,800 Btu/day.

$$\text{Sludge requirement} = \frac{(1{,}200\ \text{lb/day})}{.03}(80 - 45)°\text{F} = 1{,}400{,}000\ \text{Btu/day.}$$

Total heat requirement = 346,800 + 1,400,000 Btu/day = 1,746,800 Btu/day.

(4) Determine heat supplied by utilizing sludge gas, assuming a heat value of 600 Btu/cu ft;
Heat from sludge gas = (8,100 cu ft/day)(600 Btu/cu Ft) = 4,860,000 Btu/day.

C-15. Aerobic sludge digestion. (Refer to para 16-6a.)

a. Design requirements and criteria. Determine digester volume and air requirements for the aerobic digestion of activated sludge after thickening. Assume the following conditions apply:

Sludge quantity = 1,200 lb/day;
Sludge concentration = 3 percent;
Detention time = 20 days;
Air supply requirement = 30 cfm/l,000 cu ft digester volume.

b. Calculations and results.

(1) Determine digester volume:

$$\text{Sludge volume} = \frac{1{,}200\ \text{lb/day}}{8.34 \times (0.03)} = 4{,}796\ \text{gal day} = 4{,}796 \times \frac{1\ \text{cu ft}}{7.48\ \text{gal}} = 641\ \text{cu ft/day.}$$

Digester volume = 641 cu ft/day × 20 day = 12,820 cu ft.

(2) Determine air requirement:
Air required = (12,820 cu ft)(30 cfm/1,000 cu ft) = 384.6 cfm, use 385 cfm.
(This also satisfies mixing requirements.)

C-16. Sludge pumping. (Refer to para 16-2.)

a. Design requirements and criteria. Determine the horsepower and pressure requirements for pumping sludge from a settling tank to a thickener. Assume the following conditions apply:

Sludge is pumped at 6 fps;
150 ft of 8” pipe is used;
Thickener is 10 ft above settling tank (elev diff = 10’);
Sludge specific gravity = 1.02;
Sludge moisture content = 95 percent;
Pressure at inlet side of pumps = 3 psi;
Coefficient of friction = 0.01.

b. Calculations and results.

(1) Calculate head loss, using Manning's Equation, to give friction loss; F (for water):

$$F = \frac{4\ f\ L}{D}\ \frac{V^2}{2\ g\ c} = \frac{2\ f\ V^2}{gc}\ \frac{L}{D} \qquad \text{(eq C-17)}$$

f = coefficient of friction;
L = length of pipe, ft;
D = diameter of pipe, ft;
V = mean velocity, fps;
gc = gravity constant.

$$F = \frac{4(0.01)(150\ \text{ft})}{(8\ \text{in} \times 1/12\ \text{ft/in})}\ \frac{(5\ \text{fps})^2}{2(32.17\ \text{ft lb/lb force sec}^2)}$$

= 5.04 ft of fluid.

Assuming friction losses for sludge 3 times that for water, the head loss from friction is:

h_t = (5.04 ft)(3) = 15.1 ft, use 15 ft.

(2) Calculate total pumping head:

H = 10 ft + 15 ft = 25 ft of sludge.

Add 3 ft to this account for losses due to valves, elbows, etc.

Total H = 29 ft of sludge.

(3) Assuming a pump efficiency of 60 percent, calculate horsepower requirement:

$$hp = \frac{QP_W(\text{sp gr})H}{550\ \text{eff}} = (P_W = \text{density of water}) \qquad \text{(eq C-18)}$$

Q = (6 fps) ($(0.333)^2$) = 2.1 cfs;

$$hp = \frac{(2.1\ \text{cu ft/sec})(62.4\ \text{lb/cu ft})(1.02)(28\ \text{ft})}{550\ (0.60)}$$

= 11.3 up, use 15 hp.

(4) Determine discharge pump pressure:

$$P = \frac{(\text{Total head})(\text{Specific gravity})(\text{weight } H_2O)}{144\ \text{sq in/sq ft}}$$

$$= \frac{(28\ \text{ft})(1.02)(6.24\ \text{lb/cu ft})}{144\ \text{sq in/sq ft}}$$

= 12.4 psi.

C-17. Gravity sludge thickener. (Refer to para 16-3.)

a. Design requirements and criteria. Size a gravity thickener tank to handle an activated sludge. Assume the following conditions apply:

Amount of sludge to thickener = 1,200 lb/day;
Sludge solids content = 1 percent (i.e., 10,000 mg/L);
Solids loading for thickener = 8 lbs/sq ft/day;
Tank depth = 12 ft.

b. Calculations and results.

(1) Determine surface area:

$$SA = \frac{1{,}200\ \text{lb/day}}{8\ \text{lb/sq ft/day}} = 150\ \text{sq ft.}$$

(2) Determine surface loading rate:

$$\text{Sludge flow} = \frac{1{,}200 \text{ lb/day}}{(8.34 \text{ lb/gal})(0.01 \text{ solid/liquid})(150 \text{ sq ft})} = 96 \text{ gpd/sq ft.}$$

(3) Check detention time:

$$DT = \frac{(150 \times 12) \text{ cu ft} \times 7.48 \times 24}{14{,}388 \text{ gal/day}} = 22.5 \text{ hrs.}$$

C-18. Vacuum filtration. (Refer to para 16-5c.)

a. Design requirements and criteria. Design a vacuum filter to dewater 10,000 gpd digested sludge containing 5 percent solids. Assume the following conditions:

Specific resistance, r = 3 × 10^7 sec^2/g;
Required vacuum = 25 in Hg;
Weight of filtered solids per unit volume of filtrate = 4 lb/cu ft;
Cycle time, 0 (from manufacturer)6 min;
Form time (from manufacturer)3 min.

b. Calculations and results.

$$\text{Pressure, } P = 25'' = \frac{25 \times 14.7}{29.9} = 12.3 \text{ psi.}$$

$$R = \frac{r}{10^7} = \frac{3 \times 10^7}{10^7} = 3.0;$$

$$\mathbf{x} = \frac{\text{Form time}}{\text{Cycle time}} \frac{3}{6} = 0.5;$$

O = 6min;
= 0.896 centipoise.

Calculate filter yield:

$$L = 35.6 \times \frac{0.5(12.3)(0.064)^{1/2}}{0.896(3.0)(6)} \qquad \text{(eq C-20)}$$

= 5.56 lb/sq ft/hr.

These values are typical values for digested sludge. For design purposes, it is preferred to use values based on pilot or laboratory studies.

Compute the total solids in the sludge (S):
S = 0.024×8.34×50,000
= 10,000 lb/day
= 417 lb/hr.

Required filter area:

$$A = \frac{417}{5.56} = 75 \text{ sq ft.}$$

Therefore:
Filter diameter = 4 ft;
Filter length = 6 ft.

If specific resistance is unknown for a specific application, use loading rates as follows:
3 lbs/sq ft/hr activated sludge;
10 lbs/sq ft/hr primary sludge;
5 lbs/sq ft/hr combined primary and activated sludge.

C-19. Chlorinator. (Refer to para 17-3.)

a. Design requirements and criteria. Determine the capacity of a chlorinator for an activated sludge wastewater treatment plant with an average flow of 2 mgd. The peaking factor for the treatment plant is 2.5. The average required chlorine dosage is 8 mg/L and the maximum required chlorine dosage is 20 mg/L. EPA regulations require 30 min contact time at peak hour conditions.

b. Calculations and results.

(1) Determine capacity of the chlorinator at peak flow:

lb Cl_2/day = 20 mg/L × 8.34 lb/gal × 2 mgd × 2.5 = 834.

Use four 250 lb/day units.

(2) Estimate the daily consumption of chlorine:

Average dose = 8 mg/I (see table in chap 16) lb Cl_2/day 8 × 8.34 × 2.0 = 133.4.

Use 140.

It should be noted that the total unit capacity is about 6 times the average needed chlorine. This is to cover the peak hydraulic flow of wastewater and to cover a wider range of dosage of chlorine that might be needed under unfavorable conditions. Space requirements for the 4 units and for storage of chlorine tanks is estimated to be about 400 sq ft.

(3) Chlorine contact tank. Required volume for 30 min contact time:

Vol = Q × T;

where

Q = peak flow rate, gpm;

T = detention time, min.

$$\text{Vol} = \frac{2.0 \times 106 \text{ gal/day}}{1{,}440 \text{ min/day}} (2.5)(30 \text{ min}) = 104{,}170 \text{ gal} = 13{,}930, \text{ use } 14{,}000 \text{ cu ft.}$$

Assume 6 *ft* SWD + 2 ft freeboard.

$$\text{Let } \frac{L}{W} = 10 \text{ for plug flow tank; therefore,}$$

L × W × D = Vol.

Since L = 10W; therefore,

$$10\ W^2 = \frac{\text{Vol}}{D}$$

$$W = .\ \frac{\text{Vol}}{10 \times D}½ = \frac{14{,}000}{10 \times 6}½ = 15.3 \text{ ft}$$

L = 10W = 153 ft in length.

Therefore, the basic chlorine contact tank dimensions are:

153 ft × 15.3ft × 8 ft (6 ft SWD).

Baffling is used to construct a more regular tank shape and to prevent short circuiting. (Refer to EPA **Process Design Manual for Upgrading Existing Wastewater Treatment Plants** for layout of tank baffles.) Using a flow channel width of 15.3 ft and 4 side-by-side plug flow compartments, the overall tank width is:

$$\frac{153 \text{ ft}}{4} = 38.3 \text{ ft.}$$

The length is:

4 × 15.3 ft = 61.2.

The tank dimensions are therefore:

61.2 ft × 38.3 ft × 8 ft (6 ft SWD).

C-20. Mound systems. (Refer to para 6-5.)

a. Design requirements and criteria. Munitions storage guard house. Maximum capacity: 9 persons. Area is underlain by coarse sand and irregular rocks. Bedrock is over 50' below site. Soil excavations reveal no hard-pan, but water table is only 2' below ground.

Slope = 5 percent;

Percolation rate = 55 min/in at 24 in;

Manifold length = 125 ft;

Septic tank anticipated volume = 2,000 gal.

(1) **Choose a location**: Using soil maps, boreholes and soil pits, choose a location free of trees with no clay lenses or large boulders in the soil profile. Choose mound site before building the housing.

(2) **Calculate waste load**: Consider fill occupancy. Whoever is on guard lives there, so anticipate 9 berthing areas.

150 gal/day/room × 9 rooms = 1,350 gal/day.

(3) **Select fill material**: When quality fill is not readily available, the mound system is excessively expensive. Therefore, utilize the soil maps to locate areas of medium sand. If the sand is not >25 percent coarse sand and <5 percent fine sand, then it should bbe "up—graded" by screening out fines and materials greater than 2mm in size, using a trommel. Final stockpiled material should have an infiltration rate of 1.0 to 1.5 gal/ft^2/day.

(4) **Size the absorption area**: Use filtration rate of 1.20 gal/ft2/day (table 6—3). Absorption area required = 1,350 gal/day 1.20 gal/ft2/day = 1,124 ft2. Use a conservative value of 1,200 ft2. Since the area is permeable and of constant slope, a bed system could be used (trenches might be used if the topography required following a contour or was very narrow). Two long narrow beds, A = 10" wide; each bed will be 60" long (B).

(5) **Mound height:** At upslope side, minimum depth = 1 ft (see fig 6-7). At downslope side, depth = E = D + slope (A). Side depth = E = 1 + 0.05 (25) = 2.25. Bed depth (F) = 1 ft minimum with at least 8 in of aggregate (not limestone) beneath the distribution piping. Bed total thickness = 1 ft. Cap should be a minimum of 6 in of subsoil and 6 in of reclaimed topsoil. To provide drainage, use 2.0" at centerline (see fig C-9).

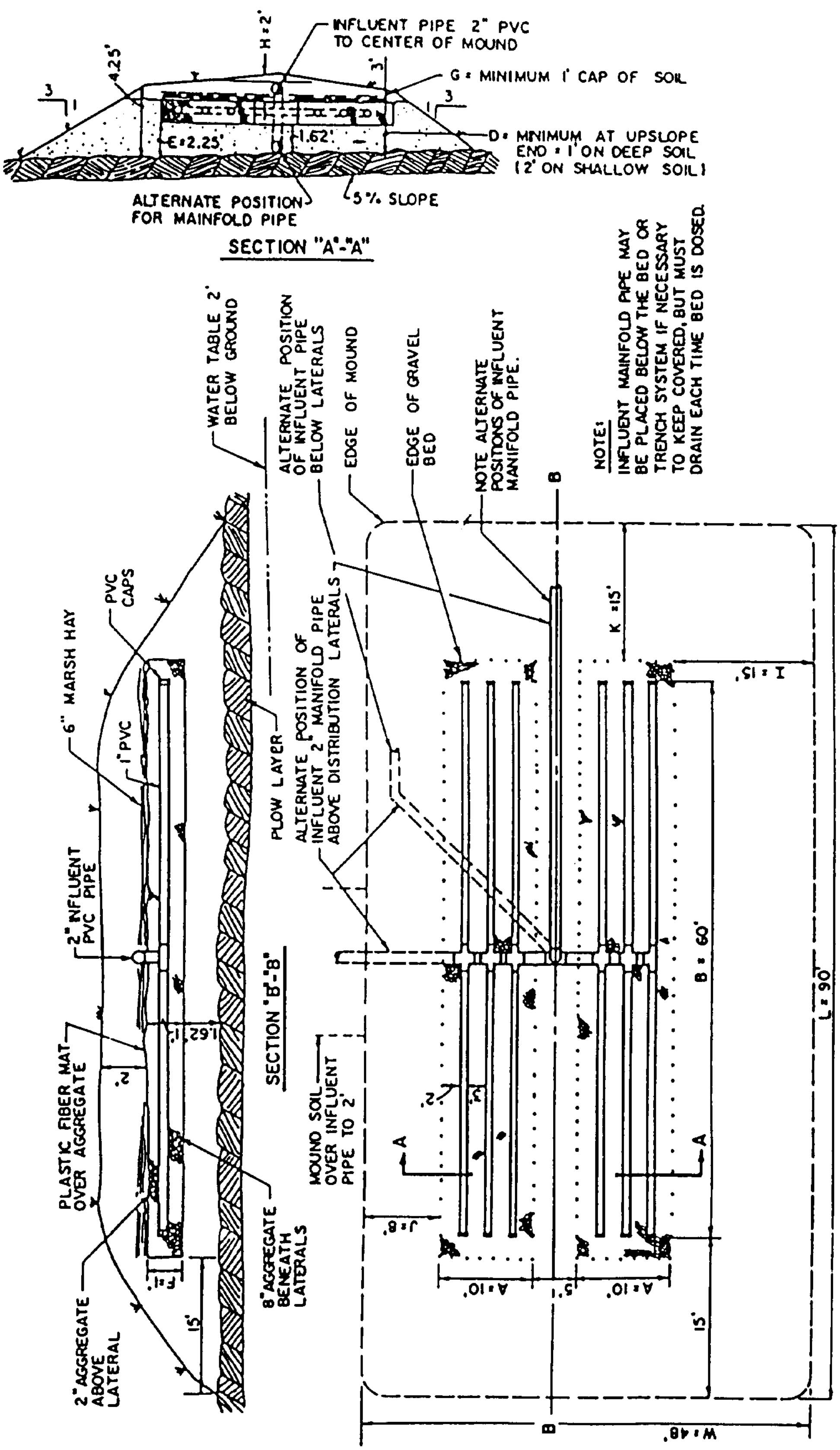


Figure C–9. Mound design on sloped ground.

(6) **Mound length and width:**

End slopes (K) = mound depth at center × 3:1 slope = $\frac{D + E}{2} + F + H \times 3$

$= \frac{1 + 2.25}{2} + 1.00 + 2.0 \times 3$

= 13.8', use 15'

Upslope width (J) = mound depth at upsiope edge × 3:1 slope × slope correction (table 6-2)
= (D + F + G) × 3 × .875
= (1.00 + 1.00 + 1.00) × 3 × .875
= 7.88, use 8.

Downslope width (I) = mound depth at lower edge × 3:1 × slope correction
= (E + F + G) × 3 × 1.18
= (2.25 + 1.00 + 1.00) × 3 × 1.18
= 15.04 ft, use 15 ft.

Mound length (L) = B = 2K = 60 + 2(15) = 90 ft.

Mound width (W) = I + A + J = 15 + 2(10) + 5 + 8 = 48 ft.

(7) **Calculate basal area:** On a sloping site, the basal area is that area under the slope of the bed; while on level sites, the whole area of the mound may be used. Since this is a sloped area:
Basal area (BA) = Bx (A + I) = 60 × [)2 × 10) + 5 + 15] = 2,400 ft2.
Basal area required (BAR) = daily flow infiltration capacity of soil.
Typical design loading rates are:

Percolation Rates	
3-29 min/in	1.2 gal/ft2/day
30-60 min/in	0.75 gal/ft2/day
60-120 min/in	0.24 gal/ft2/day

Therefore, BAR = 900 gal/day × 0.74 gal/ft2/day = 666.

Since BA< BAR, the slopes could be shortened. However, if slope is cut to 2:1, erosion may set in. Therefore, size is acceptable at 2,400 ft^2 since this figure is conservative by a factor of 4.

(8) **Distribution system:** For this size system, manifold pipe should be 2"; laterals, 1" PVC. The manifold pipe should run from the pumping chamber (or dosing siphon) to the center of the mound to an "elbow" and then through a "reducer" (from 2" to 1"), thence through a "tee" to the 6 lateral distribution pipes below. The manifold pipe may be run above or below the beds but, in either case, must be designed so that wastewater will drain from the laterals and the manifold between doses.

Table C-5 is used to determine perforation spacing and pipe diameter. In this case, laterals of 1-in PVC are 30 ft long so that a 30-in spacing of 7/32-in holes is chosen.

Table C–5. Allowable lateral lengths (feet).

Perforation Spacing (in)	Perforation Diameter (in)	Pipe Diameter (1 in)	(1¼ in)	(1½ in)
30	3/16	34	52	70
	7/32	30	45	57
	1/4	25	38	50
36	3/16	36	60	75
	7/32	33	51	63
	1/4	27	42	54

(9) **Calculate approximate void volume:** Laterals and manifold.

Lateral volume (cu in) $= A \times \text{length}$
$= (r)^2$ (No. of laterals × length (ft) × 12)
= 3,393 cu in.

Manifold volume $= (1)^2(125)(12)$
= 4,710 cu in.

Total void volume = 8,103 cu in = 35 gallons.

(10) **Dosing volume/pump selection:** To be certain that all laterals are quickly and completely filled upon each dosing, dosing volume is estimated at 10 times void volume, or 350 gallons. Controls, pump size and septic tanks should be chosen based upon conventional engineering practices and local codes. The design should include a high pressure cutoff to prevent over—pressurization of the system, a pump chamber with appropriate control, and a pump pedestal to protect the pump from settled solids. Use figure C-10 to check flow rate against mound absorption area. Dosing volume should be about 1/5th of the septic tank capacity. In this case, the septic tank is about 2,000 gallons. Since each dose will be 350 gallons, the tank will always remain 80 percent full, allowing settlement of solids and pumping (or siphoning) of relatively clear effluent to the mound. The pump must be sized to deliver 2 ft of head at the distal end of laterals after due consideration of:

(a) elevation difference between pump and lateral invert;

(b) friction losses in pump, manifolds and lateral.

In this case, a pump might supply 2.5 ft of head at the supply end to overcome friction losses. A pump capable of delivering 2.5 ft of head at 120 gals/mm would pump for about 3 min to deliver the 350-gal dose (fig C-10). After installation, pressure, timing and dose rate will be adjusted prior to final burial of the piping. Each dose must fill all laterals to about 2 ft head to assure even distribution. However, pressure should not be too great as pipes may burst.

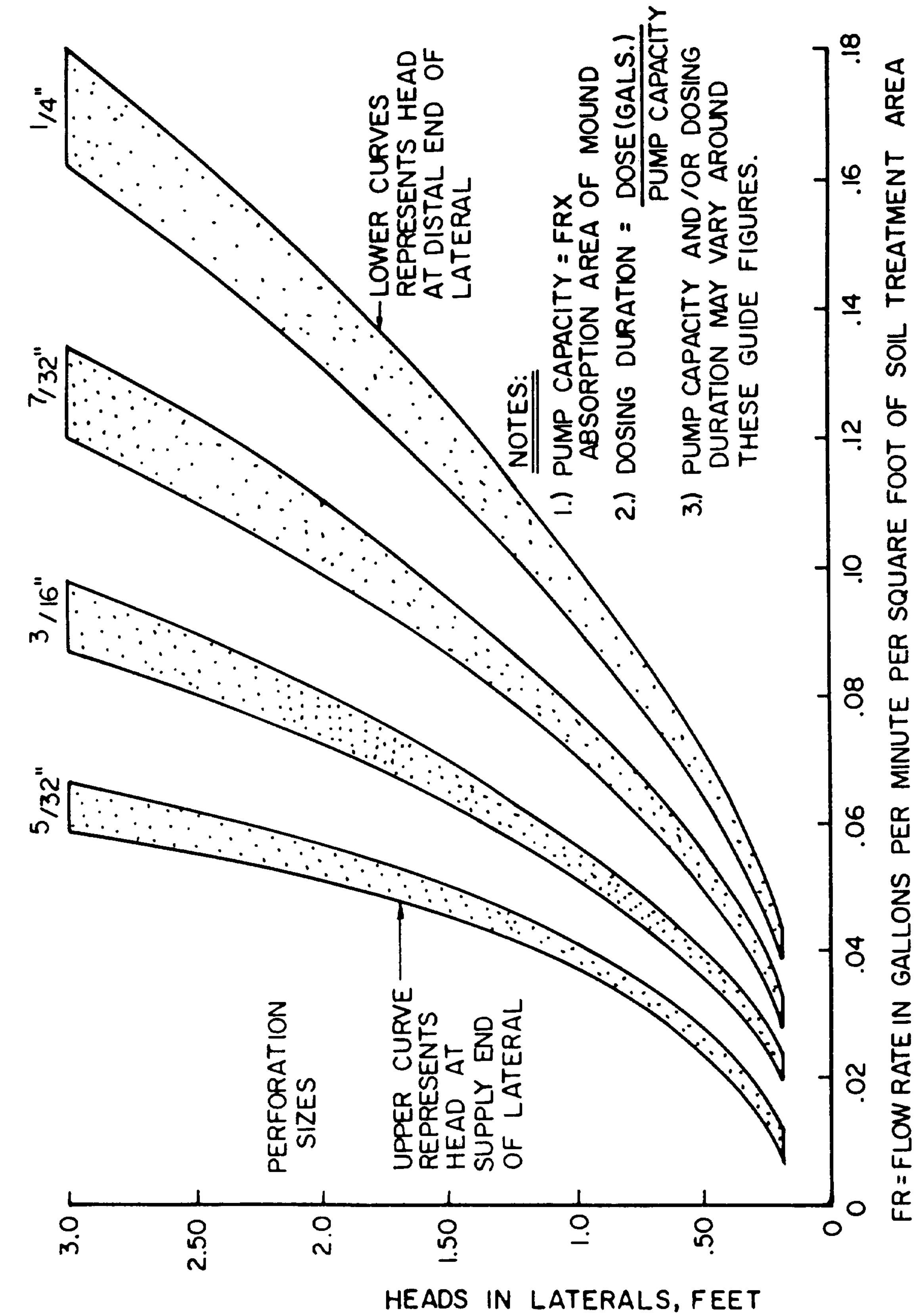


Figure C-10. Pump selection curves.

(11) When the system is fully laid out with pipes cemented in place, it should be tested and dosing volume adjusted as necessary to assure that all holes are operating, that all pipes are full at each dosing, and that there is no excess pressure. Pressure head at end of laterals should be about 2 ft when in full operation at time of pump cutoff. Pipes are then covered with more gravel, then protected by woven plastic fabric covered by straw. Finally, subsoil and then topsoil are carefully spread over the top, and the shaped mound is seeded to grass. Do not neglect to ditch the edges of the mound to channel runoff around the mound.