EASTERN MUNICIPAL
WATER DISTRICT
SINCE 1950

SANITARY SEWER SYSTEM
PLANNING & DESIGN

PRINCIPLE

GUIDELINES

CRITERIA

Updated February 9, 1993

Revised 09/1/2006

SANITARY SEWER SYSTEM BASIS OF DESIGN

Eastern Municipal Water District
Engineering Manual

				3
				1.0
SEWERAGE	GENERAL		(per I.D. Memo #10536 by WEP) DATED 2/9/93	

(BY BILL PLUMMER)

DESIGN FLOWS

In 1989, a survey of the District's sewer system was performed to determine flow generation rates from various land uses. This information is contained in the Wastewater Facilities Master Plan prepared by Black & Veatch dated 1990. The results of the survey showed variation in sewage generation not only by type of use but also by location (e.g. Sun City housing had a lower sewage generation per unit than Moreno Valley). However, for design purposes it is important that criteria be developed and used on a consistent basis. To achieve this goal a meeting was held to agree on the criteria for sewer design. The result of the meeting is a compromise of actual measurements vs design criteria. Table 1 attached shows the relationship of land use to the wastewater flow agreed to. The information in Table 1 shall be used by the District for future sewer design. This information has been adjusted to correspond to future conditions that are expected to uniformly occur as development takes place in all areas of the District.

The Wastewater Facility Master Plan also developed peak flow rates and obtained data which correlate peak flow rates with average flow rates. By a plot of this data, a curve has been established which is used in determining the peaking factor to be used in the design of the sewer. The peaking curve that is to be used in District design is shown on Table 2.

The procedure to be followed in determining design flows is to first determine the tributary drainage area for the sewer pipeline, determine the various average flows within the drainage area, add these average flows, and then convert these average flows to a peak flow for the design of the sewer (i.e., $Q_{Design} = Q_{AVE.}$ x Peaking Factor).

PIPE SIZE SELECTION

Sewers 12-inches in diameter and smaller are designed to flow at a maximum depth of one-half the diameter of the pipe. Sewers 15-inches in diameter and larger are designed to flow at a maximum of three-quarter depth of the pipe diameter.

It is important to maintain an air gap in the top of sewer pipes to convey sewer gases downstream along with the sewage flow. Maintaining the maximum depth of flow to pipe diameter ratio (D/d) conditions described above helps to ensure that sufficient space occurs to meet these conditions.

An 8-inch diameter pipeline has been established as the minimum sewer pipe size. This conclusion was established for two main reasons:

1. Maintenance problems can occur on smaller size pipes.

2. Sufficient space is necessary to convey sewage and debris down stream in the sewer pipe to avoid possible backflow up sewer laterals.

The only exceptions to the 8-inch minimum pipe size criteria are in the Communities of Romoland, Homeland and Green Acres, where 6-inch diameter sewer pipelines were installed due to grant conditions applied to the financing of sewers in these communities.

MANNING "n" VALUES

Pipe size is determined by using mannings equation which is shown below:
$Q = (1.486/n)\ AR^{2/3}\ S^{1/2}$
Q= flows (cfs) n= mannings coefficient
A= cross sectional area of pipe (feet^2)
R= hydrologic radius of the wetted cross-section of the pipe (feet)

S= slope of energy gradient

Refer to Handbook of Hydraulics by Brater and King or the Clay Pipe Engineering Manual for use of the equation.

PIPE SLOPES

The minimum slopes for sewer pipelines are based on obtaining a minimum velocity of 2 fps at design peak flow depth. This provides a means to resuspend solids deposited in the sewer during peak flows. Refer to Table 3 for minimum pipe slopes.

On small-size sewers, there is generally no particular concern with maximum slopes or velocities, except where water and ends of sewer may be insufficient in volume to move solids. On large-size sewers, it is necessary to design sewers which would have a peak velocity not exceeding 12 fps to avoid damage to plastic liners on RCP joints.

SEWER SIPHON

Sewer siphons are designed to convey sewage under obstructions. All efforts should be made to design a sewer system to avoid sewer siphons because severe odor and maintenance problems can result. The major concern is grease and other floating material cannot easily flow into the siphon and become trapped in the upstream manhole.

The hydraulic design of the sewer siphon is shown on Figure 1. The upstream sewer shall not flow full at peak flow rates. If feasible, install air jumper between the upstream and downstream manholes. This will be used to convey sewer gases across the siphon and can serve as an overflow if the siphon becomes plugged.

HYDRAULIC JUMPS

Hydraulic jumps can occur when a steep slope suddenly turns flat. Low jumps, that is, the change in depth is small, the water will not rise obviously and abruptly but will pass from the low to the high stage through a series of undulations gradually diminishing in size. When the jump is high, that is, when the change in depth is great, the jump is called a direct jump. The direct jump involves a relatively large amount of energy loss through dissipation in the turbulent body of water. High turbulance results in the release of sulphide gases. Since sulphide gases result in concrete corrosion it is necessary that all manholes have a PVC liner installed within 1000-feet downstream from the hydraulic jump location. Hydraulic jumps should be avoided in sewer design.

WEP:taj/slh

Table 1

EMWD - System Design and Loading Criteria

Average Daily Flow:

Residential	EDU's / Acre [1]		Population / EDU	GPD / Capita	GPD / Acre [2]
	Typical	Range			
Low Density (LDR)	2.5	0 to 2.9	4	105	1,050
Medium Density (MDR)	4.5	3 to 11	3.5	100	1,575
High Density (HDR)	12	12 to 16	2.5	80	2,400
Very High Density (VHDR)	17	17+	2.2	80	2,992
Mobile Homes (MH)	6	varies	2	80	960
Age Restricted Comm.	varies	varies	2	80	960

Non-Residential		
Commercial	1700	GPD / Acre
Industrial	1700	GPD / Acre
Institutional	1000	GPD / Acre
Hospital	250	GPD / Bed
Schools	20	GPD / Student

Manning's Coefficient "n": n = 0.013 (varies with depth for design)
use n = 0.015 (for sizing pipes)

Peaking Factor: See attached sheet (Table 2 - Peak Flow Rates)

Velocity: 2 ft/sec MINIMUM, 3 ft/sec recommended, & 10 ft/sec maximum

Notes:

(1) For calculation of actual flow, use actual Equivalent Dwelling Units (EDU) per Gross Acre

(2) Applies to Typical EDU's / Acre only

TABLE 2 - AVERAGE FLOW RATES

Population	Peaking Factor	105 (1)	100 (1)	80 (1)
		Avg. Daily Flow (MGD)		
100	2.50	0.011	0.010	0.008
250	2.50	0.026	0.025	0.020
500	2.50	0.053	0.050	0.040
750	2.50	0.079	0.075	0.060
1,000	2.49	0.105	0.100	0.080
1,050	2.48	0.110	0.105	0.084
1,100	2.47	0.116	0.110	0.088
1,150	2.46	0.121	0.115	0.092
1,200	2.45	0.126	0.120	0.096
1,250	2.44	0.131	0.125	0.100
1,300	2.43	0.137	0.130	0.104
1,350	2.42	0.142	0.135	0.108
1,375	2.41	0.144	0.138	0.110
1,400	2.40	0.147	0.140	0.112
1,450	2.39	0.152	0.145	0.116
1,500	2.38	0.158	0.150	0.120
1,550	2.37	0.163	0.155	0.124
1,600	2.36	0.168	0.160	0.128
1,625	2.35	0.171	0.163	0.130
1,650	2.34	0.173	0.165	0.132
1,675	2.33	0.176	0.168	0.134
1,700	2.32	0.179	0.170	0.136
1,750	2.31	0.184	0.175	0.140
1,800	2.30	0.189	0.180	0.144
1,850	2.29	0.194	0.185	0.148
1,900	2.28	0.200	0.190	0.152
1,950	2.27	0.205	0.195	0.156
2,000	2.26	0.210	0.200	0.160
2,075	2.25	0.218	0.208	0.166
2,150	2.24	0.226	0.215	0.172
2,225	2.23	0.234	0.223	0.178
2,300	2.22	0.242	0.230	0.184
2,375	2.21	0.249	0.238	0.190
2,425	2.20	0.255	0.243	0.194
2,500	2.19	0.263	0.250	0.200
2,675	2.18	0.281	0.268	0.214
2,775	2.17	0.291	0.278	0.222
2,850	2.16	0.299	0.285	0.228
2,925	2.15	0.307	0.293	0.234
3,000	2.14	0.315	0.300	0.240
3,100	2.13	0.326	0.310	0.248
3,200	2.12	0.336	0.320	0.256
3,350	2.11	0.352	0.335	0.268
3,500	2.10	0.368	0.350	0.280
3,600	2.09	0.378	0.360	0.288
3,700	2.08	0.389	0.370	0.296
3,800	2.07	0.399	0.380	0.304
3,900	2.06	0.410	0.390	0.312
4,000	2.05	0.420	0.400	0.320
4,200	2.04	0.441	0.420	0.336
4,400	2.03	0.462	0.440	0.352
4,500	2.02	0.473	0.450	0.360

Population	Peaking Factor	105 (1)	100 (1)	80 (1)
		Avg. Daily Flow (MGD)		
4,800	2.01	0.504	0.480	0.384
5,000	2.00	0.525	0.500	0.400
5,200	1.99	0.546	0.520	0.416
5,400	1.98	0.567	0.540	0.432
5,600	1.97	0.588	0.560	0.448
5,800	1.96	0.609	0.580	0.464
6,000	1.95	0.630	0.600	0.480
6,200	1.94	0.651	0.620	0.496
6,400	1.93	0.672	0.640	0.512
6,650	1.92	0.698	0.665	0.532
6,900	1.91	0.725	0.690	0.552
7,200	1.90	0.756	0.720	0.576
7,500	1.89	0.788	0.750	0.600
7,800	1.88	0.819	0.780	0.624
8,100	1.87	0.851	0.810	0.648
8,400	1.86	0.882	0.840	0.672
8,700	1.85	0.914	0.870	0.696
9,100	1.84	0.956	0.910	0.728
9,600	1.83	1.008	0.960	0.768
10,200	1.82	1.071	1.020	0.816
10,500	1.81	1.103	1.050	0.840
11,500	1.80	1.208	1.150	0.920
12,300	1.79	1.292	1.230	0.984
13,000	1.78	1.365	1.300	1.040
13,800	1.77	1.449	1.380	1.104
14,500	1.76	1.523	1.450	1.160
15,300	1.75	1.607	1.530	1.224
16,000	1.74	1.680	1.600	1.280
16,700	1.73	1.754	1.670	1.336
17,400	1.72	1.827	1.740	1.392
18,100	1.71	1.901	1.810	1.448
18,900	1.70	1.985	1.890	1.512
19,800	1.69	2.079	1.980	1.584
21,700	1.68	2.279	2.170	1.736
22,700	1.67	2.384	2.270	1.816
23,800	1.66	2.499	2.380	1.904
25,000	1.65	2.625	2.500	2.000
26,400	1.64	2.772	2.640	2.112
28,000	1.63	2.940	2.800	2.240
30,000	1.62	3.150	3.000	2.400
32,200	1.61	3.381	3.220	2.576
34,600	1.60	3.633	3.460	2.768
37,200	1.59	3.906	3.720	2.976
40,000	1.58	4.200	4.000	3.200
43,000	1.57	4.515	4.300	3.440
46,000	1.56	4.830	4.600	3.680
50,000	1.55	5.250	5.000	4.000
55,000	1.54	5.775	5.500	4.400
60,000	1.53	6.300	6.000	4.800
70,000	1.52	7.350	7.000	5.600
85,000	1.51	8.925	8.500	6.800
100,000	1.50	10.500	10.000	8.000

(1) GPD/Capita, Per Table 1

TABLE 3

PIPE MINIMUM PIPE SLOPES IN SEWER MAINS

Pipe Diameter	Preferred Minimum	Ordinary Minimum	Preferred Maximum slope (not mandatory)
8-inch	.0065	.0040	.12
10-inch	.0050	.0032	.085
12-inch	.0040	.0024	.066
15-inch	.0032	.0016	.050
18-inch	.0024	.0014	.037
21-inch	.0020	.0012	.030
24-inch	.0017	.0010	.025
27-inch	.0015	.0008	.022
30-inch	.0013	.0007	.018

a) House Connection Laterals

Pipe diameter	4-inch	6-inch	8-inch
Minimum slope	0.020	0.020	0.020

(0.010 Extreme Minimum with prior approval only)

Table 4 - Sewer Pipe Capacity

[3] D/d = 0.5

K' Coeff. = 0.232

Pipe Size (d) (in)	Min. Slope s [1]	Flow Area a (ft²)	[1]Manning Coeff. n = 0.013 Q (cfs)	V (fps)	Max # of EDUs [2]	[1]Manning Coeff. n = 0.015 Q (cfs)	V (fps)	Max # of EDUs [2]
8	0.0040	0.1745	0.383	2.193	283	0.332	1.901	245
10	0.0032	0.2727	0.621	2.277	486	0.538	1.973	412
12	0.0024	0.3927	0.874	2.226	724	0.758	1.929	616

[3] D/d = 0.75

K' Coeff. = 0.422

Pipe Size (d) (in)	Min. Slope s [1]	Flow Area a (ft²)	[1]Manning Coeff. n = 0.013 Q (cfs)	V (fps)	Max # of EDUs [2]	[1]Manning Coeff. n = 0.015 Q (cfs)	V (fps)	Max # of EDUs [2]
15	0.0016	0.9873	2.354	2.384	2,228	2.040	2.067	1,892
18	0.0014	1.4218	3.581	2.519	3,592	3.104	2.183	3,047
21	0.0012	1.9352	5.001	2.584	5,157	4.334	2.240	4,420
24	0.0010	2.5276	6.518	2.579	6,954	5.649	2.235	5,924
27	0.0008	3.1990	7.981	2.495	8,717	6.917	2.162	7,423
30	0.0007	3.9494	9.888	2.504	11,060	8.569	2.170	9,415
36	0.0007	5.6871	16.078	2.827	18,782	13.935	2.450	16,075
42	0.0007	7.7408	24.253	3.133	29,258	21.019	2.715	25,029
48	0.0007	10.1104	34.627	3.425	42,325	30.010	2.968	36,441
54	0.0007	12.7960	47.404	3.705	58,330	41.084	3.211	50,553

Notes:

[1] Per Table 1 & 3

[2] Based on Table 1 - Medium Density Residential (3.5 cap per DU / 100 gpcd) and Table 2

[3] Use D/d of 0.5 for 8", 10" and 12" sewerlines only; Use D/d = 0.75 for 15" and larger sewerlines.

Table 7-13. Values of K for Circular Channels in the Formula

$$Q = \frac{K}{n} D^{8/3} s^{1/2}$$

D = depth of water d = diameter of channel

$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		15.02	10.56	8.57	7.38	6.55	5.95	5.47	5.08	4.76
.1	4.49	4.25	4.04	3.86	3.69	3.54	3.41	3.28	3.17	3.06
.2	2.96	2.87	2.79	2.71	2.63	2.56	2.49	2.42	2.36	2.30
.3	2.25	2.20	2.14	2.09	2.05	2.00	1.96	1.92	1.87	1.84
.4	1.80	1.76	1.72	1.69	1.66	1.62	1.59	1.56	1.53	1.50
.5	1.470	1.442	1.415	1.388	1.362	1.336	1.311	1.286	1.262	1.238
.6	1.215	1.192	1.170	1.148	1.126	1.105	1.084	1.064	1.043	1.023
.7	1.004	.984	.965	.947	.928	.910	.891	.874	.856	.838
.8	.821	.804	.787	.770	.753	.736	.720	.703	.687	.670
.9	.654	.637	.621	.604	.588	.571	.553	.535	.516	.496
1.0	.463									

Table 7-14. Values of K' for Circular Channels in the Formula

$$Q = \frac{K'}{n} d^{8/3} s^{1/2}$$

D = depth of water d = diameter of channel

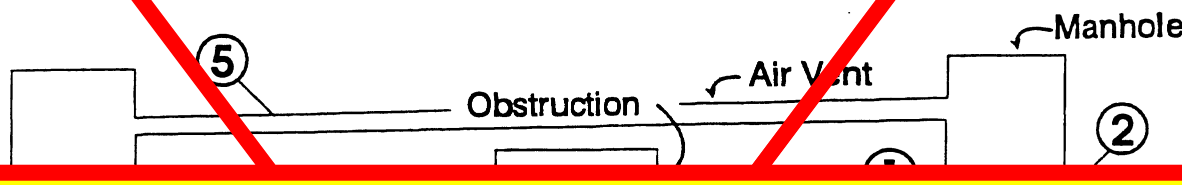
$\frac{D}{d}$	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
.0		.00007	.00031	.00074	.00138	.00222	.00328	.00455	.00604	.00775
.1	.00967	.0118	.0142	.0167	.0195	.0225	.0257	.0291	.0327	.0366
.2	.0406	.0448	.0492	.0537	.0585	.0634	.0686	.0738	.0793	.0849
.3	.0907	.0966	.1027	.1089	.1153	.1218	.1284	.1352	.1420	.1490
.4	.1561	.1633	.1705	.1779	.1854	.1929	.2005	.2082	.2160	.2238
.5	.232	.239	.247	.255	.263	.271	.279	.287	.295	.303
.6	.311	.319	.327	.335	.343	.350	.358	.366	.373	.380
.7	.388	.395	.402	.409	.416	.422	.429	.435	.441	.447
.8	.453	.458	.463	.468	.473	.477	.481	.485	.488	.491
.9	.494	.496	.497	.498	.498	.498	.496	.494	.489	.483
1.0	.463									

$$K' = \frac{Qn}{d^{8/3} S^{1/2}}$$

Q = CFS

d = FT

Siphon Design Criteria



Dec 11, 2008 update: SHEET 9 of 9 is VOID - Siphon is no longer allowed -

5 Air vent sloped to drain.

FIGURE 1