A PLANNING AND DESIGN GUIDEBOOK FOR COMBINED SEWER OVERFLOW CONTROL AND TREATMENT

by

Dulcie A. Weisman
Richard Field
Storm and Combined Sewer Section
Wastewater Research Division
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08837

September, 1981

INTRODUCTION

This report is a survey of control and treatment of combined sewer overflows (CSO), encompassing the Storm and Combined Sewer Section's research efforts over the last fifteen years.

The survey was prepared to assist Federal, state, and municipal agencies, and private consultants, in 201 Facilities Planning and Design, Steps 1 and 2, respectively.

The discussions of control/treatment technologies which consist, mostly, of downstream treatment, have been divided into seven chapters:

(1) Source Control
(2) Collection System Control
(3) Storage
(4) Physical with/without Chemical Treatment
(5) Biological Treatment
(6) Advanced Treatment
(7) Disinfection

Storage is the best documented CSO abatement measure currently practiced, and it must be considered at all times in system planning, because it allows for maximum use of existing dry-weather facilities. Physical with/without chemical treatment will generally be the minimum required to meet discharge or receiving water quality goals. If a higher degree of organics removal is needed, biological treatment should be examined. If maintaining a viable microorganism population is not feasible, but removal of dissolved and colloidal organics is desired, advanced treatment may be attractive.

General discussions of CSO control/treatment can be found in the following documents, which also served as principal references for this report:

EPA-670/2-74-040
Urban Stormwater Management and Technology: An Assessment

EPA-600/8-77-014
Urban Stormwater Management and Technology: Update and User's Guide

EPA-600/2-76-286
Cost Estimating Manual—Combined Sewer Overflow Storage and Treatment

EPA-600/8-80-035
Urban Stormwater Management and Technology: Case Histories

Field, R., and E. J. Struzeski, Jr. Management and Control of Combined Sewer Overflows. J. Water Pollution Control Federation, Vol. 44, No. 7, July, 1972.

Field, R., and J.A. Lager. Urban Runoff Pollution Control State-of-the-Art. J. Environmental Engineering Division, ASCE, Vol. 101, No. EE1, February, 1975.

A comprehensive list of references appears at the end of each chapter.

SOURCE CONTROL

Street Sweeping

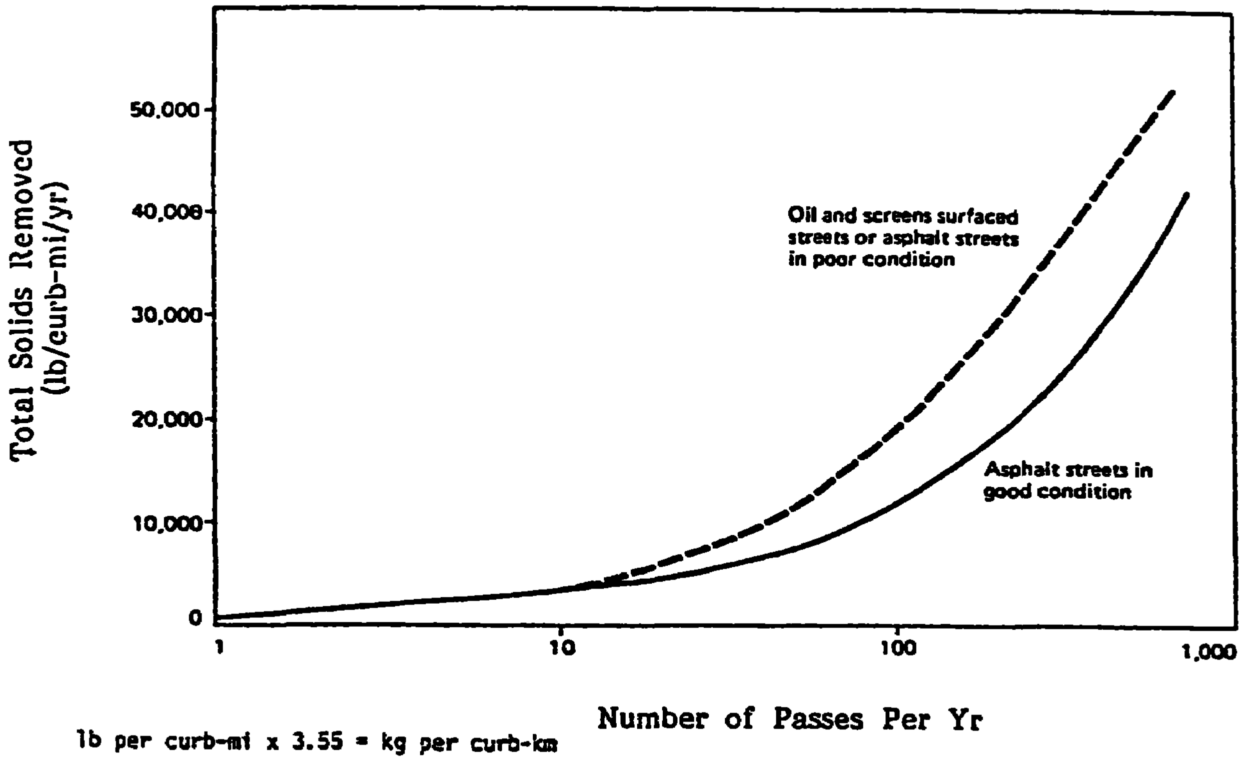
Street sweeping, to remove accumulated dust, dirt, and litter, has been shown to be an effective, but limited method of attacking the source of stormwater-related pollution problems. Street cleaning effectiveness is a function of (1) pavement type and condition, (2) cleaning frequency, (3) number of passes, (4) equipment speed, (5) sweeper efficiency and, (6) equipment type. Pavement type and condition affect performance more than differences in equipment, and in general, smooth asphalt streets are easier to keep clean than those in poor condition, or streets with oil and screens surfaces (pavement type consisting of loosely bound aggregate in a very thick, oily matrix).

The most important measure of street cleaning effectiveness is "pounds per curb-mile removed" for a specific program condition. This removal value, in conjunction with the unit curb-mile costs, allows the cost for removing a pound of pollutant for a specific street cleaning program to be calculated.

In the San Jose, CA, street sweeping project (EPA-600/2-79-161), experimental design and sampling procedures were developed that can be used in different cities to obtain specific information about street dirt characteristics and its effects on air and water quality. At the test site in San Jose, it was determined that frequent street cleaning on smooth asphalt streets (once or twice per day) can remove up to 50 percent of the total solids and heavy metal mass yields of urban runoff, whereas, typical street cleaning programs (once or twice per month) remove less than 5 percent of the total solids and heavy metals in the runoff.

It was also determined that removal per unit effort decreased with increasing numbers of passes per year. This is shown in Figure 1, which relates the annual total solids removed to the street cleaning frequency, for different street surface conditions in San Jose.

Street sweeping results are highly variable. Therefore, a street sweeping program for one city cannot be applied to other cities, unless the program is shown to be applicable through experimental testing. This may be seen when comparing street sweeping test results from San Jose, and an ongoing project in Bellevue, WA. In Bellevue, it was demonstrated that additional cleaning, after a certain level of effort, is not productive, and that the additional street cleaning effort would be better applied to other areas. For the study area in Bellevue, it is estimated that street cleaning operations of about two or three passes per week would remove up to about 68 kg of solids per curb-km (150 lb / curb-mi), or up to 25 percent of the initial street surface load. Increased utilization of street cleaning equipment would result in very little additional



lb per curb-mi x 3.55 = kg per curb-km

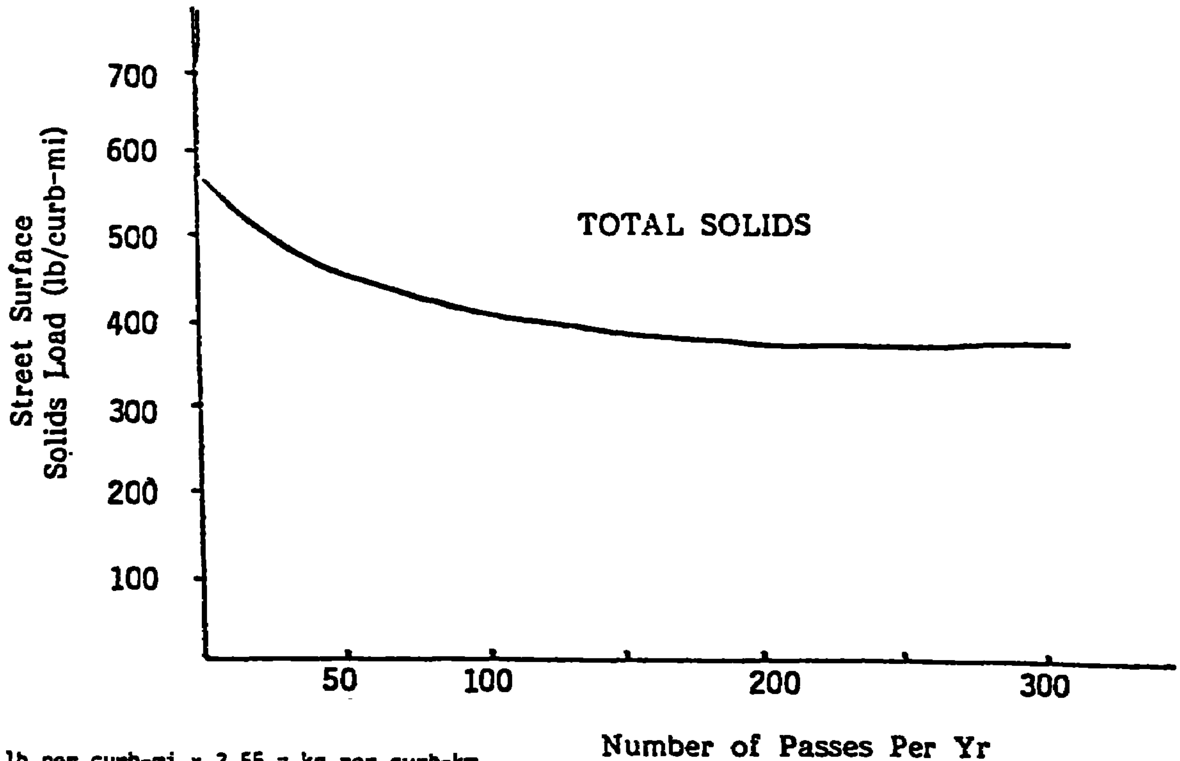
Figure 1. Annual Amount Removed as a Function of the Number of Passes per yr at San Jose Test Site

(EPA-600/2-79-161)

benefit. This is illustrated, for total solids and COD removals, in Figures 2, and 3, respectively. Increased street cleaning operations beyond two or three times per week are likely to increase the street surface loadings, due to erosion of the street surface. Increasing the cleaning frequency from once per week to two or more times per week, will have only a very small additional benefit. Cleaning very infrequently (once every two months) may not be beneficial at all, except in cities where it may be possible to schedule street cleaning so that it is coordinated with rainfall events.

Street cleaning not only affects water quality, but has multiple benefits, including improving air quality, aesthetic conditions, and public health. Since street cleaning alone will probably not ensure that water quality objectives are met, a street cleaning program would have to be incorporated into a larger program of "best management practices," and/or downstream treatment.

Costs of street cleaning have been reported to range from ($3.40 to $13.14/curb-km) ($5.47 to $21.13/curb-mi) swept (ENR = 3452). The wide variation in these costs was attributed to differences in labor rates, and equipment costs.



lb per curb-mi x 3.55 = kg per curb-km

Figure 2. Street Cleaner Productivity
(Bellevue, WA)

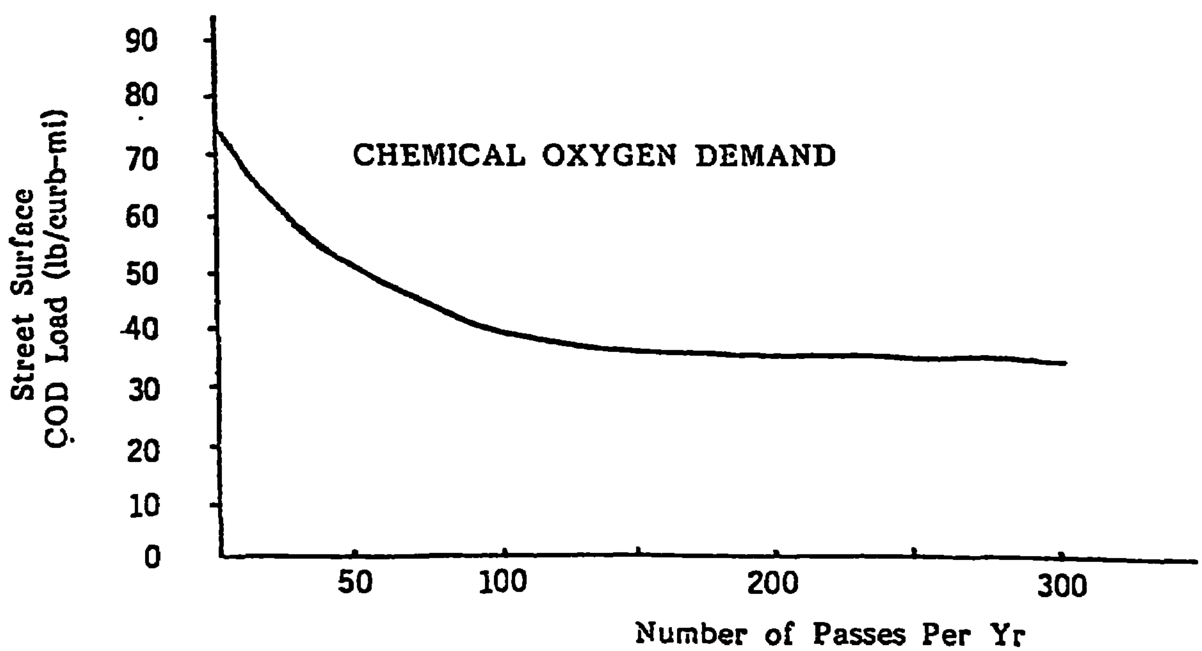


Figure 3. Street Cleaner Productivity
(Bellevue, WA)

References

Bellevue (WA) Street Sweeping Demonstration Project-First Annual Report, Cooperative Agreement # CR-805929, U.S. Environmental Protection Agency, January, 1981.

EPA-600/8-80-035 - Urban Stormwater Management and Technology: Case Histories: by W.G. Lynard et al., Metcalf & Eddy, Inc., Palo Alto, CA, August, 1980.
NTIS PB 81 107153

EPA-600/2-79-161 - Demonstration of Nonpoint Pollution Abatement Through Improved Street Cleaning Practices: by R. E. Pitt, Woodward-Clyde Consultants, San Francisco, CA, August, 1979.
NTIS PB 80-108988

EPA-600/2-75-004 - Contributions of Urban Roadway Usage to Water Pollution: by D.G. Shaheen, Biospherics Inc., Rockville, MD, March, 1975.
NTIS PB 245 854

EPA-R2-73-283 - Toxic Materials Analysis of Street Surface Contaminants: by R.E. Pitt, and G. Amy, URS Research Co., San Mateo, CA, August, 1973.
NTIS PB 224 677

EPA-R2-72-081 - Water Pollution Aspects of Street Surface Contaminants: by J.D. Sartor and G.B. Boyd, URS Research Co., San Mateo, CA, November, 1972.
NTIS PB 214 408

COLLECTION SYSTEM CONTROL

Catchbasins/Catchbasin Cleaning

A catchbasin is defined as a chamber or well, usually built at the curbline of a street, for the admission of surface water to a sewer or subdrain, having at its base a sediment sump designed to retain grit and detritus below the point of overflow. It should be noted that a catchbasin is designed to trap sediment, while an inlet is not. Historically, the role of catchbasins has been to minimize sewer clogging, by trapping coarse debris (from unpaved streets) and to reduce odor emanations from low-velocity sewers, by providing a water seal.

In a project conducted in the West Roxbury section of Boston, three catchbasins were cleaned, and subsequently, four runoff events were monitored at each catchbasin. Average pollutant removals per storm are shown in Table 1.

Table 1. Pollutants Retained in Catchbasins

Constituent	% Retained
SS	60-97
Volatile SS	48-97
COD	10-56
BOD_5	54-88

Catchbasins must be cleaned often enough to prevent sediment and debris from accumulating to such a depth that the outlet to the sewer might become blocked. The sump must be kept clean to provide storage capacity for sediment, and to prevent resuspension of sediment. Since the volume of stormwater detained in a catchbasin will reduce the amount of overflow by that amount (it eventually leaks out or evaporates), it is also important to clean catchbasins to provide liquid storage capacity.

To maintain the effectiveness of catchbasins for pollutant removal will require a cleaning frequency of at least twice per year, depending upon conditions. The increased cost of cleaning must be considered in assessing the practicality of catchbasins for pollution control.

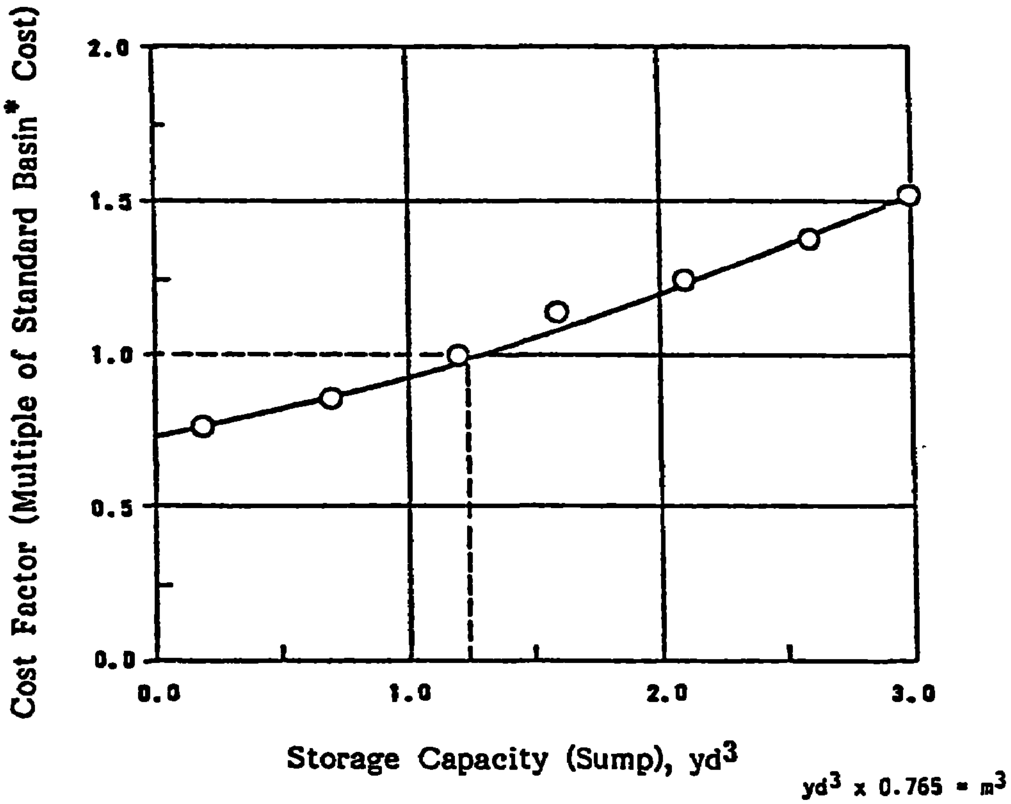
Typical cost data for catchbasins are presented in Table 2. The reported costs will vary, depending on the size of the catchbasin used by a particular city. Catchbasin cost multiplication factors, as a function of sump storage capacity, are shown in Figure 4.

Table 2. Catchbasin Costs

	Range	Avg
Total Installed Cost, $	700- 1,700	1,400

ENR = 3452

(EPA-600/2-77-051)



*The standard basin is basically a barrel 183 cm (6 ft) deep and 122 cm (4 ft) in diameter with an open top covered by a grating and an outlet pipe mounted at the side approximately 107 cm (3.5 ft) above the bottom.

Figure 4. Catchbasin Cost Factors vs Storage Capacity

(EPA-600/2-77-051)

Estimated national average costs for three catchbasin cleaning methods are presented in Table 3.

Table 3. Catchbasin Cleaning Costs

Manual			Eductor			Vacuum		
$/catchbasin	$/m^3	$/yd^3	$/catchbasin	$/m^3	$/yd^3	$/catchbasin	$/m^3	$/yd^3
13.20	32.50	25.00	10.20	9.25	7.00	13.80	19.40	14.20

ENR= 3452

(EPA-600/2-77-051)

References:

EPA-600/2-77-051 - Catchbasin Technology Overview and Assessment: by J. Lager et al., Metcalf & Eddy, Inc., Palo Alto, CA, in association with Hydro-Research-Science, Santa Clara, CA, May, 1977. NTIS PB 270 092

Evaluation of Catchbasin Monitoring- Draft Final Report, by G.L. Aronson et al., Environmental Design & Planning, Inc., Allston, MA, Grant No. R-804578.

Sewer Flushing

The deposition of sewage solids in combined sewer systems during dry weather has long been recognized as a major contributor to "first-flush" phenomena occurring during wet weather runoff periods. The magnitude of these loadings during runoff periods has been estimated to range up to 30 percent of the total annual dry weather sewage loadings.

Sewer flushing during dry weather is designed to remove the material, periodically, as it accumulates, and hydraulically convey it to the treatment facilities, thus, preventing resuspension and overflow of a portion of the solids during storm events, and lessening the need for CSO treatment. Flushing is particularly beneficial for sewers with grades too flat to be self-cleansing, and also helps ensure that sewers can carry their design flow capacities. Sewer flushing requires cooperation between the authorities with jurisdiction over collection system maintenance and wastewater treatment.

For developing sewer flushing programs, it is necessary to be able to estimate deposition build-up. Predictive equations have been developed, based on field studies in Boston, to relate the total daily mass of pollutant deposition in a collection system to collection system characteristics, such as per capita waste production rate, service area, total pipe length, average pipe slope, and average pipe diameter. A simple model is given by the equation:

$$TS = 0.0076(L')^{1.063}(\bar{S})^{-0.4375}(q)^{-0.51} \qquad (R^2=0.845)$$

where TS = deposited solids loading, lb/d
$\bar{S}$ = mean pipe slope, ft/ft
L' = total length of sewer system, ft
q = per capita waste rate* (plus allowance for infiltration), gpcd

(EPA-600/2-79-133)

*U.S. Public Health Service has indicated a national average of 150 gpcd (Wastewater Treatment Plant Design- WPCF and ASCE, 1977).

The total pipe length of the system, L', is generally assumed to be known. In cases where this information is not known, and where crude estimates will suffice, the total pipe length can be estimated from the total basin area, A (acres), using the expressions:

For low population density (10-20 people/acre)

$$L' = 168.95\,(A)^{0.928} \qquad (R^2 = 0.821)$$

For moderate-high population density (30-60 people/acre)

$$L' = 239.41\,(A)^{0.928} \qquad (R^2 = 0.821)$$

If data on pipe slope is not available, the mean pipe slope can be estimated using the following equation:

$$\bar{S} = 0.348(\bar{S}_g) \qquad (R^2 = 0.96)$$

where $\bar{S}_g$ = mean ground slope, ft/ft

It has been found that cleansing efficiency of periodic flush waves is dependent upon flush volume, flush discharge rate, sewer slope, sewer length, sewer flow rate, sewer diameter, and is also dependent upon population density. Maximum flushing rates at the downstream point are limited to the regulator/interceptor capacities prior to overflow.

Internal automatic flushing devices have been developed for sewer systems. An inflatable bag is used to stop flow in upstream reaches until a volume capable of generating a flushing wave is accumulated. When the appropriate volume is reached, the bag is deflated, with the assistance of a vacuum pump, releasing impounded sewage, and resulting in the cleaning of the sewer segment. Field experience has indicated that sewer flushing by manual means (water tank truck) is a simple, reliable method for CSO solids removal in smaller diameter laterals and trunk sewers.

Pollutant removals as a function of length of pipe flushed, (Dorchester, MA - EPA-600/2-79-133) are presented in Table 4. The relationship between cleaning efficiency and pipe length is important, since an aim of flushing is to wash the resuspended sediment to strategic locations, such as a point where sewage is flowing, to another point where flushing will be initiated, or to the sewage treatment plant.

Table 4. Pollutant Removals by Sewer Flushing as a Function of Length of Segment Flushed (254-381 mm (10-15 in.) pipe)

	% Removals, Organics and Nutrients	% Removals, Dry-weather grit/inorganic material
Manhole to Manhole Segments	75-95	75
Serial Segments up to 213 m (700 ft)	65-75	55-65
Segment lengths greater than 305 m (1000 ft)	35-45	18-25

Flushing is also an effective means for suspending and transporting heavy metals associated with light colloidal solids particles. Approximately 20-40 percent of heavy metals contained within sewage sediment, including cadmium, chromium, copper, lead, nickel and zinc, have been found to be transported at least 305 m (1000 ft) by flush waves.

Estimated costs of sewer flushing methods are shown in Table 5.

Table 5. Estimated Costs of Sewer Flushing Methods
Based on Daily Flushing Program

Number of Segments: 46 (254-457 mm (10-18 in) pipe)

Automatic Flushing Module Operation (one module/segment)

Capital Cost*	$15,000/segment
Annual O&M Cost	$138,000

Manual Flushing Mode

Capital Cost†	$95,000
Annual O&M Cost	$164,100

*includes site preparation, and fabrication and installation of air-operated module

†includes 3 outfitted water tankers

ENR = 3452 (EPA-600/2-79-133)

References:

EPA-600/2-80-118 - Review of Alternatives for Evaluation of Sewer Flushing-Dorchester Area-Boston: by H.L. Kaufman and F. Lai, Clinton Bogert Associates, Ft. Lee, NJ, August, 1980. NTIS No. Pending

EPA-600/2-79-133 - Dry-Weather Deposition and Flushing for Combined Sewer Overflow Pollution Control: by W. Pisano, Northeastern University, Boston, MA, August, 1979. NTIS PB 80-118524

EPA-600/2-77-120 - Procedures for Estimating Dry-Weather Pollutant Deposition in Sewerage Systems: by W. Pisano and C.S. Queriroz, Energy and Environmental Analysis, Inc., Boston, MA, July, 1977. NTIS PB 270 695

11020DNO03/72 - A Flushing System for Combined Sewer Cleansing: by Central Engineering Laboratories, FMC Corp., Santa Clara, CA, March, 1972. NTIS PB 210 858

11020DNO08/67 - Feasibility of a Periodic Flushing System for Combined Sewer Cleansing: by FMC Corp., Santa Clara, CA, August, 1967.
NTIS Only PB 195 223

Regulator/Concentrators

Swirl Regulator/Concentrator---

The dual-functioning swirl regulator/concentrator can achieve both flow control and good removals (90-100 percent, laboratory determined) of inert settleable solids (effective diameter 0.3 mm, s.g. 2.65) and organics (effective diameter 1.0 mm, s.g. 1.2). It should be noted that the laboratory test solids represent only the heavier fraction of solids found in CSO. Actual CSO contains a wider range of solids, so removals in field operations are closer to 40-50 percent.

Swirls have no moving parts. Flow is regulated by a central circular weir spillway, while simultaneously, solid/liquid separation occurs by way of flowpath induced inertial separation, and gravity settling. Dry-weather flows are diverted through the foul sewer outlet, to the intercepting sewer for subsequent treatment at the municipal plant. During higher flow storm conditions, 3-10 percent of the total flow, which includes sanitary sewage, storm runoff, and solids concentrated by swirl action, is diverted by way of the foul sewer outlet to the interceptor. The relatively clear, high-volume supernatant overflows the central circular weir, and can be stored, further treated, or discharged to a stream.

The swirl is capable of functioning efficiently over a wide range of CSO rates and has the ability to separate settleable light weight solids and floatable solids at a small fraction of the detention time normally required for sedimentation. A swirl unit is illustrated in Figure 5.

Suspended solids removals for the Syracuse, NY, prototype unit, as compared to hypothetical removals in a conventional regulator, are shown in Table 6. BOD_5 removals for the Syracuse unit are shown in Table 7 .(see EPA-600/2-79-134), Disinfection/Treatment of Combined Sewer Overflows).

Helical Bend Regulator/Concentrator---

The helical bend flow regulator is based on the concept of using the helical motion imparted to fluids at bends when a total angle of approximately 60 degrees and a radius of curvature equal to 16 times the inlet pipe diameter (D) are employed.

Figure 6 illustrates the device. The basic structural features of the helical bend are: the transition section from the inlet to the expanded straight section before the bend, the overflow side weir and scum baffle, and the foul outlet for concentrated solids removal, and controlling the amount of underflow going to the treatment works.

Dry-weather flow goes through the lower portion of the device, and to the intercepting sewer. As the liquid level increases during wet-weather, helical motion begins and the solids are drawn to the inner wall and drop to the lower

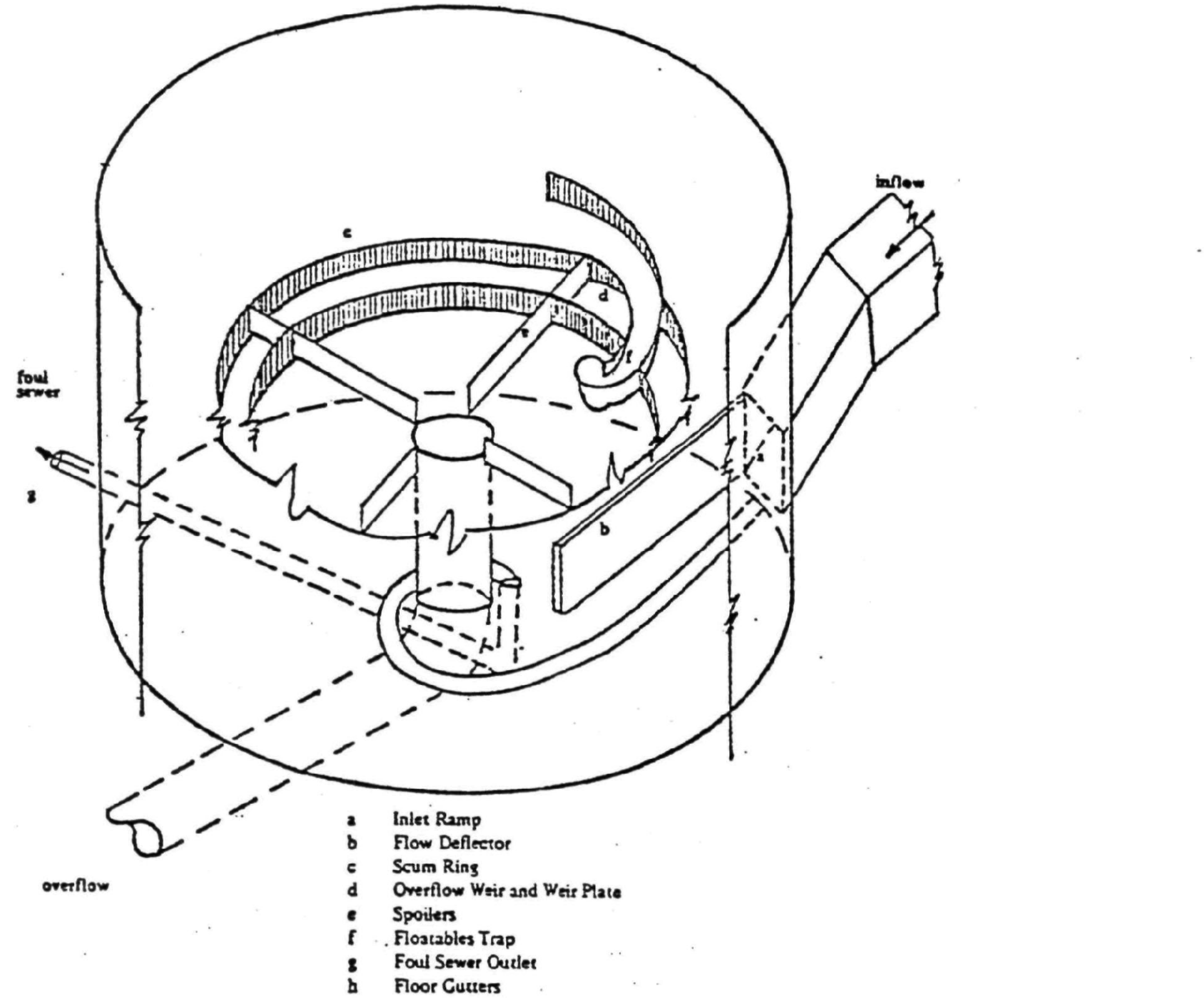


Figure 5. Isometric View of Swirl Regulator/Concentrator

(EPA-670/2-73-059)

Table 6. SS Removal

	SWIRL REGULATOR/CONCENTRATOR						CONVENTIONAL REGULATOR (hypothetical)			
	Average SS per storm, mg/l			Mass Loading, kg			Mass Loading kg			Swirl Net Removal Benefit (%)[c]
Storm #	Inf.	Eff.	(%) Rem.[a]	Inf.	Eff.	(%) Rem.[a]	Inf.	Underflow	(%) Rem.[b]	
2-1974	535	315	36	374	179	52	374	101	27	25
3-1974	182	141	23	69	34	51	69	33	48	3
7-1974	110	90	18	93	61	34	93	20	22	12
10-1974	230	164	29	255	134	48	256	49	19	29
14-1974	159	123	23	99	57	42	99	26	26	16
1-1975	374	167	55	103	24	77	103	66	64	13
2-1975	342	202	41	463	167	64	463	170	37	27
6-1975	342	259	24	112	62	45	112	31	28	17
12-1975	291	232	20	250	168	33	250	48	19	14
14-1975	121	81	33	83	48	42	83	14	17	25
15-1975	115	55	52	117	21	82	117	72	62	20

a Data reflecting negative SS removals at tail end of storms not included.

b For the conventional regulator removal calculation, it is assumed that the SS concentration of the foul underflow equals the SS concentration of the inflow.

c Calculated by subtracting the hypothetical percent SS removals in a conventional regulator, from the percent SS removals in a swirl regulator/concentrator.

(EPA-600/2-79-134; EPA-625/2-77-012)

Table 7. Swirl Regulator/Concentrator BOD_5 Removal

	Mass Loading, kg			Average BOD_5 per storm, mg/l		
Storm #	Inf.	Eff.	(%) Rem.	Inf.	Eff.	(%) Rem.
7-1974	26,545	4,644	82	314	65	79
1-1975	3,565	1,040	71	165	112	32
2-1975	12,329	6,164	50	99	70	29

(EPA-600/2-79-134; EPA-625/2-77-012)

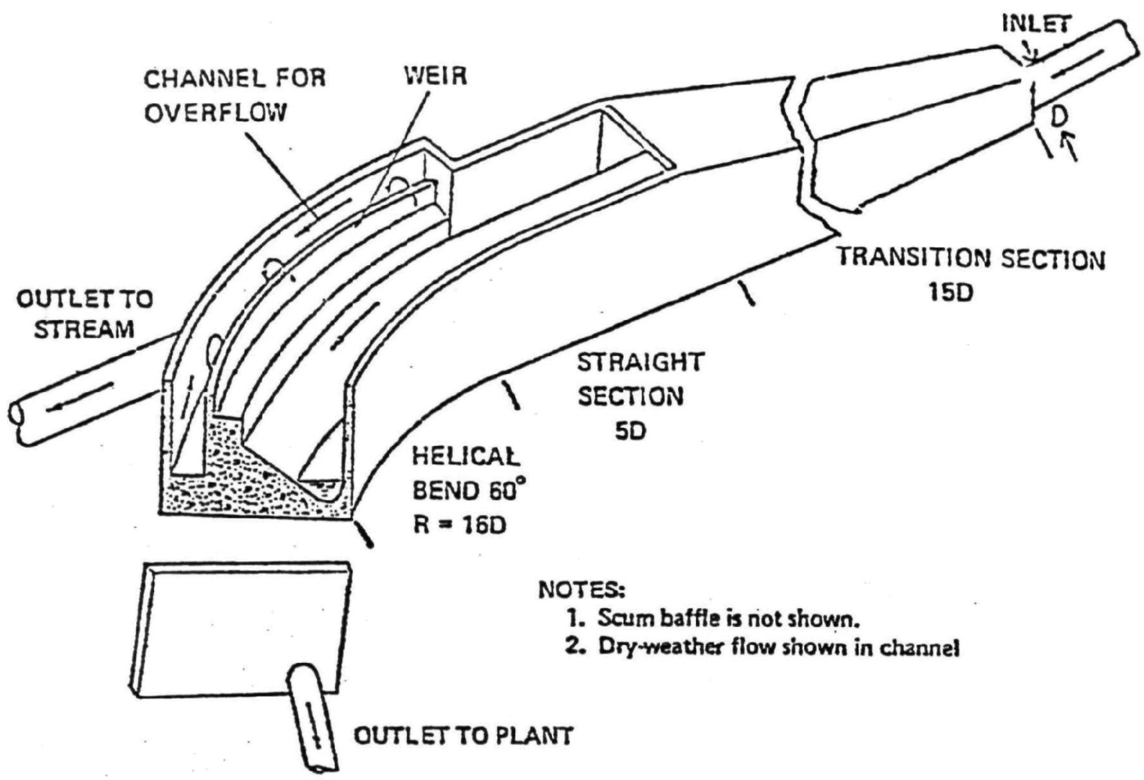


Figure 6. Isometric View Of Helical Bend Regulator/Concentrator (EPA-600/2-75-062)

level of the channel leading to the treatment plant. When the storm subsides, the velocity of flow increases, due to the constricted channel. This helps prevent the settling of solids. As with the swirl, the proportion of concentrated discharge will depend on the particular design. The relatively clean CSO passes over a side weir, and is discharged to the receiving water, or storage and/or treatment facilities. Floatables are prevented from overflowing by a scum baffle along the side weir, and collect at the end of the chamber. They are conveyed to the treatment plant when the storm flow and liquid level subside.

Based on laboratory tests, pollutant removals in a helical bend unit are comparable to those in a swirl (a full scale helical bend is currently being demonstrated in Boston, MA). Helicals and swirls are, in effect, upstream treatment devices for the removal of relatively heavy, coarse material, but they cannot be used to substitute for primary clarification.

A comparison of construction costs for helical bend and swirl regulator/ concentrators is presented in Table 8.

Table 8. Comparison of Construction Costs-Helical Bend and Swirl Regulator/Concentrators

Capacity	Swirl	Helical
1.42 m^3/s (50 ft^3/s)	$182,463	$ 445,472
2.83 m^3/s (100 ft^3/s)	295,886	835,055
4.67 m^3/s (165 ft^3/s)	410,952	1,176,968

Note: Land Costs not included

ENR = 3452

It should be noted that these costs do not reflect the real cost-effectiveness of swirls and helicals, since these units actually serve dual functions, i.e., flow control, and wastewater treatment. Even though the construction cost for the helical bend is higher than for the swirl, the helical may be more appropriate for a particular site, based on space availability and elevation difference between the interceptor and the incoming combined sewer (the helical requires a smaller elevation difference than the swirl). If there is not sufficient hydraulic head to allow dry-weather flow to pass through the facility, an economic evaluation would be necessay to determine the value of either pumping the foul sewer flow continuously, pumping the foul flow during storm conditions, or bypassing the facility during dry-weather conditions.

References

EPA-625/2-77-012 - Swirl Device for Regulating and Treating Combined Sewer Overflows: EPA Technology Transfer Capsule Report, Prepared by R. Field and H. Masters, USEPA, Edison, NJ ERIC 2012 (Cincinnati), 1977.

EPA-670/2-74-039 - Relationship between Diameter and Height for Design of a Swirl Concentrator as a Combined Sewer Overflow Regulator: by R.H. Sullivan et al., American Public Works Assoc., Chicago, IL, July, 1974. NTIS PB 234 646

EPA-670/2-73-059 - The Dual-Functioning Swirl Combined Sewer Overflow Regulator/Concentrator: by R. Field, USEPA, Edison, NJ, September, 1973. NTIS PB 227 182/3

EPA-R2-72-008 - The Swirl Concentrator a~ a Combined Sewer Overflow Regulator Facility: by R. Sullivan, American Public Works Association, Chicago, IL, September, 1972.
NTIS PB 214 687

EPA-600/2-75-062 - The Helical Bend Combined Sewer Overflow Regulator: by R.H. Sullivan et al., American Public Works Association, Chicago, IL, December, 1975.
NTIS PB 250 619

EPA-600/2-79-134 - Disinfection/Treatment of Combined Sewer Overflows, Syracuse, New York: by F. Drehwing et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, August, 1979.
NTIS PB 80-113459

Design Manual-Secondary Flow Pollution Control Devices: by R.H. Sullivan et al., APWA, Grant No. R-803157, (Publication Pending).

STORAGE

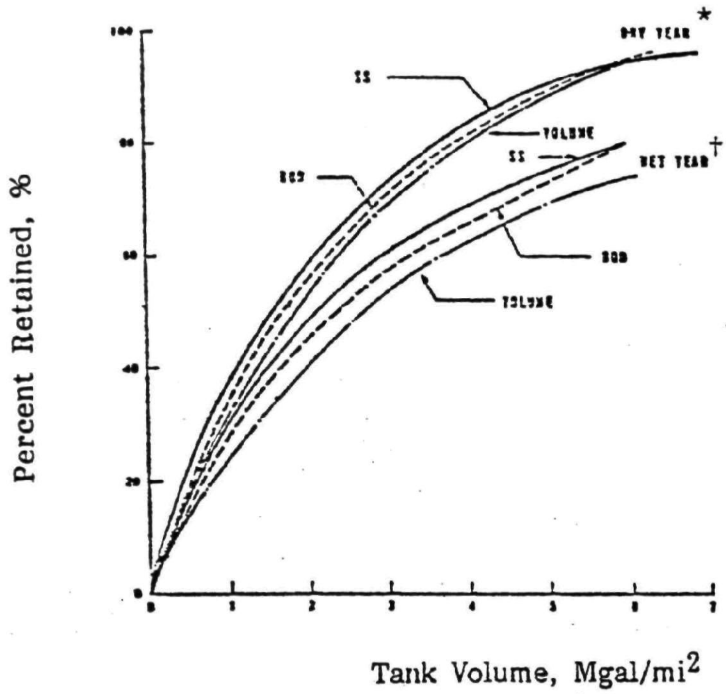
Because of the high volume and variability associated with CSO, storage is considered a necessary control alternative. Storage is also the best documented abatement measure currently practiced. Storage facilities are frequently used to attenuate peak flows associated with CSO. Storage must be considered, at all times, in system planning, because it allows for maximum use of existing dry-weather treatment plant facilities, and results in the lowest cost system in terms of treatment. The CSO is stored until the treatment plant can accept the extra volume. At that time, the CSO is discharged.

Storage facilities can provide the following advantages: (1) they respond without difficulty to intermittent and random storm behavior, and (2) they are not upset by water quality changes.

Figure 7 shows that there is an increase in BOD and SS percent removals, with an increase in tank volume per drainage area. Figure 8, however, demonstrates decreasing removal efficiencies per unit volume, as tank size increases. Also, beyond an optimum tank volume, the rate of cost increase for retaining the extra flow increases, therefore, it is not economical to design storage facilities for the infrequent storm. During periods when the tank is filled to capacity, the excess which overflows to the receiving water will have had a degree of primary treatment, by way of sedimentation.

Storage facilities can be classified as either in-line or off-line. The basic difference between the two is that in-line storage has no pumping requirements. In-line storage can consist of either storage within the sewer pipes ("in-pipe"), or storage in in-line basins. Off-line storage requires detention facilities (basins or tunnels), and facilities for pumping CSO to storage, or pumping the CSO to the sewer system.

Examination of storage options should begin with in-pipe storage. If this is not suitable, the use of in-line storage tanks should be considered, however, head allowances must be sufficient since no pumps will be used. Off-line storage should be considered last, since this will require power for pumping. Since the idea of storage is to lower the cost of the total treatment system, the storage capacity must be evaluated simultaneously with downstream treatment capacity so that the least cost combination for meeting water/CSO quality goals can be implemented. If additional treatment capacity is needed, a parallel facility can be built at the existing plant, or a satellite facility can be built at the point of storage.



*48.5 cm (19.1 in.) rainfall
†103.4 cm (40.7 in.) rainfall
runoff coefficient (C) = 0.5

Figure 7. Pollution and Volumetric Retention vs. Storage Tank Volume for Wet- and Dry-Years

(EPA-600/8-77-014)

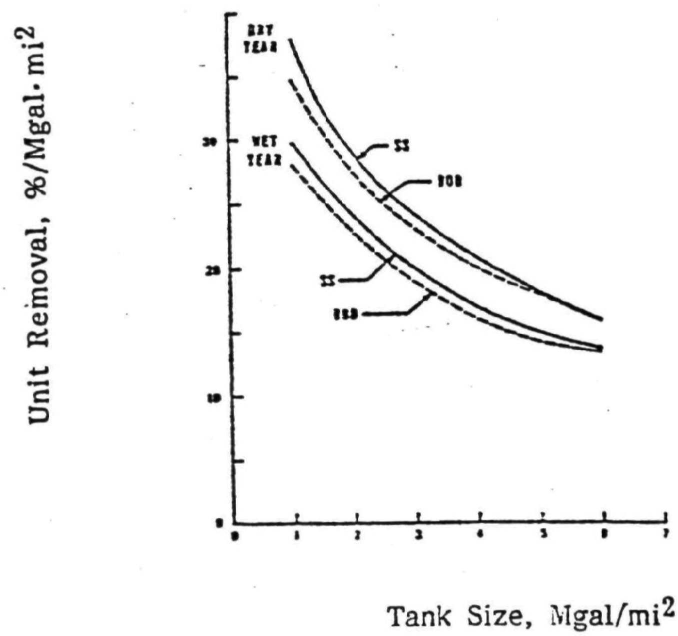


Figure 8. Unit Removal Efficiencies for CSO Detention Tanks

(EPA-600/8-77-014)

In-Pipe Storage

Because combined sewers are designed to carry maximum flows occurring, say, once in 5 years (50 to 100 times the average dry-weather flow), during most storms, there will be considerable unused volume within the conduits. In-pipe storage is provided by damming, gating, or otherwise restricting flow passage causing sewage to back-up in the upstream lines. The usual location to create the back-up is at the regulator, or overflow point, but the restrictions can also be located upstream.

For utilization of this concept, some or all of the following may be desirable: sewers with flat grades in the vicinity of the interceptor, high interceptor capacity, and extensive control and monitoring networks. This includes installation of effective regulators, level sensors, tide gates, rain gage networks, sewage and receiving water quality monitors, overflow detectors and flowmeters. Most of the systems are computerized, and to be safe, the restrictions must be easily and automatically removed from the flow stream when critical flow levels are approached or exceeded. Such systems have been successfully implemented in Seattle, and Detroit. In-pipe storage was also demonstrated in Minneapolis-St. Paul.

Costs associated with in-pipe storage systems are summarized in Table 9. Costs include regulator stations, central monitoring and control systems, and miscellaneous hardware.

Table 9. Summary of In-Pipe Storage Costs

Location	Storage capacity, Mgal	Drainage area, acres	Capital cost, $	Storage cost, $/gal	Cost per acre, $/acre	Annual operation and maintenance $/yr
Seattle, Washington						
Control and monitoring system	....	....	6,040,000	....	....	126,000
Automated regulator stations	....	....	6,730,000	....	....	380,000
	17.8	13,120	12,770,000	.73	974	506,000
Minneapolis-St. Paul, Minnesota	NA	64,000	5,200,000	....	81	....
Detroit, Michigan	140	89,600	4,850,000	.04	54	....

NA = not available.
ENR= 3452

$/acre x 2.47 = $/ha
$/gal x 0.264 = $/l
Mgal x 3 785 = m^3

(EPA-600/8-77-014)

Off-line Storage

Off-line storage facilities can be located at overflow points, or near dry weather treatment plants. Typical storage facilities include lagoons, and covered, or uncovered concrete tanks. Tunnels are also used where land is not available.

Costs for basin storage facilities are presented in Table 10, and construction cost curves are shown in Figure 9. Note that these curves do not include pumping facilities, so these curves are applicable to in-line basins. The costs for earthen basins include liners.

Innovative Storage Technology

Lake-Flow Balance System—

Karl Dunkers, an independent research engineer from Sweden, has developed, under the auspices of the Swedish EPA counterparts, an approach to lake protection against pollution from stormwater runoff. Instead of using conventional systems for equalization, i.e., concrete tanks or lined ponds, which are relatively expensive and require a lot of land area, the flow balance method uses a wooden pontoon tank system in the lake, which performs in accordance with the plug flow principle. This system is illustrated in Figure 10. The tank bottom is the lake bottom itself. The tank volume is always filled up, either with polluted stormwater runoff or with lake water. When it is raining, the stormwater runoff will "push" the lake water from one compartment to another. The compartment walls are of flexible PVC fiberglass cloth. When not receiving runoff, the system reverses by pump back and the lake water fills up the system. Thus, the lake water is utilized as a flow balance medium.

Sweden has invested in three of these installations so far. Two have been in operation for one to two years and a third was recently constructed. The systems seem to withstand wave-action up to .9m (3 ft) as well as severe icing conditions. If a wall is punctured, patching is easily accomplished. Maintenance has been found to be inexpensive.

So far, the lake-flow balance system has been demonstrated with urban runoff only. If used with CSO, consideration would have to be given to sludge handling and disposal. The Storm and Combined Sewer Section hopes to demonstrate this unique system with CSO in New York City, and at other locations in the United States. The estimated cost of this system is $525/linear meter ($160/linear foot) (ENR = 3452).

Self-Cleaning Storage/Sedimentation Basin—

In the city of Zurich, Switzerland, an in-line sedimentation-storage tank was designed to prevent solids shoaling after a storm, and provide for solids transport to the interceptor. The floor of the tank contains a continuous dry-weather channel, which is an extension of the tank's combined sewer inlet, that meanders from side to side (see Figure 11) through the tank. This channelized floor arrangement allows for complete sediment transport to the interceptor during both dry weather, and upon draw down after a storm event.

The dry-weather flow comes through the meandering bottom channel. During wet weather flows, the water level in the tank rises above the channel. If the storm intensity is low enough, there is complete capture, and if the storm intensity continues to rise, an overflow occurs through a weir at the tail

Table 10 . Summary of Basin Storage Costs[a]

Location	Storage capacity, Mgal	Drainage area, acres	Capital cost, $	Storage cost, $/gal	Cost per acre, $/acre	Annual operation and maintenance cost, $/yr
Akron, Ohio (1)	1.1	188.5	786,500	0.71	4,180	5,000
Milwaukee, Wisconsin						
Humboldt Avenue	3.9	570	3,062,000	0.78	5,370	88,000
Boston, Massachusetts						
Cottage Farm Detention and Chlorination Station[b]	1.3	15,600	11,210,000	8.63	720	140,000
Charles River Marginal Conduit Project (2)	1.2	3,000	16,380,000	13.65	5,500	170,000
New York City, New York						
Spring Creek Auxiliary Water Pollution Control Plant						
Storage	12.39	3,260	20,600,000	1.66	6,300	173,000
Sewer	13.00	. ...	... .	. ..	.....	....
	25.39	3,260	20,600,000	0.81	6,300	173,000
Chippewa Falls, Wisconsin						
Storage	2.82	90	1,290,000	0.45	14,300	4,700
Treatment	...	..	326,000	....	3,700	13,800
	2.82	90	1,616,000	0.45	18,000	18,500
Chicago, Illinois (3)						
Phase I Tunnels Under constr. or completed, and pumping stations	1,016		1,192,000,000(3a)	1.17		
Phase I Tunnels Remaining	1,033		1,074,000,000(3b)	1.04		
	2,049		2,266,000,000	1.11		7,800,000(3d)
Phase II Tunnels and Reservoirs	42,325		1,400,000,000(3c)	0.03		9,900,000(3d)
	44,374	240,000	3,666,000,000	0.08	15,275	17,700,000
Sandusky, Ohio (4)	0.36	14.86	900,000	2.49	60,400	10,700
Washington, D.C.(5)	0.20	30.0	1,500,000	7.61	50,800	5,800
Columbus, Ohio(6)						
Whittier Street	3.75	29,250[c]	10,600,000	2.83	360	.. ....
Cambridge, Maryland (7)	0.25	20	550,000	2.21	28,000	25,000

(Continued)

Table 10 (Concluded)

a. ENR = 3452, except as noted

b. Estimated values

c. Estimated area.

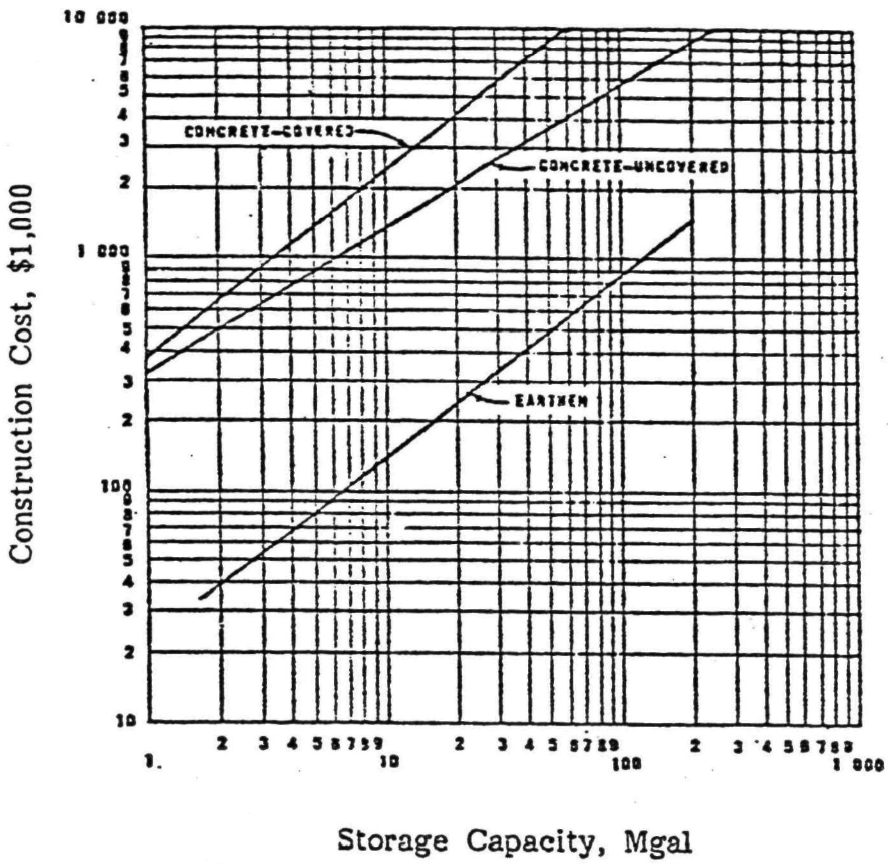
(1) EPA-600/2-76-272 (media-void space storage)
(2) Environmental Assessment Statement for Charles River Marginal Conduit Project in the Cities of Boston and Cambridge, MA. Commonwealth of Massachusetts, Metropolitan District Commission. September, 1974
(3) The Metropolitan Sanitary District of Greater Chicago, Personal Communication from Mr. Forrest Neil, Chief Engineer. September, 1981
  (3a) Actual award TARP Status Report July 1, 1981 (July 1981 dollars)
  (3b) Estimate as of January 1, 1981, TARP Status Report July 1, 1981 (January 1981 dollars)
  (3c) TARP Phase II estimate as of July 31, 1981, ENR = 3574
  (3d) Estimate (May 1980 dollars)

(4) EPA 11022ECV09/71 (underwater)
(5) EPA 11020DWF12/69 (underwater)
(6) EPA-600/2-77-064
EPA 11020FAL03/71
(7) EPA 11022DPP10/70 (underwater)

\$/acre x 2.47 = \$/ha
\$/gal x 0.264 = \$/l
Mgal x 3785 = m^3

(EPA-600/8-77-014)

end of the tank. A scum baffle prevents solids from overflowing. This arrangement allows for sedimentation to take place during a tank overflow condition and at the same time transport of solids that settle, by way of the bottom channel.



Mgal x 3 785 = m^3

Figure 9. Storage Basin Construction Costs (ENR = 2000)

(EPA-600/8-77-014)

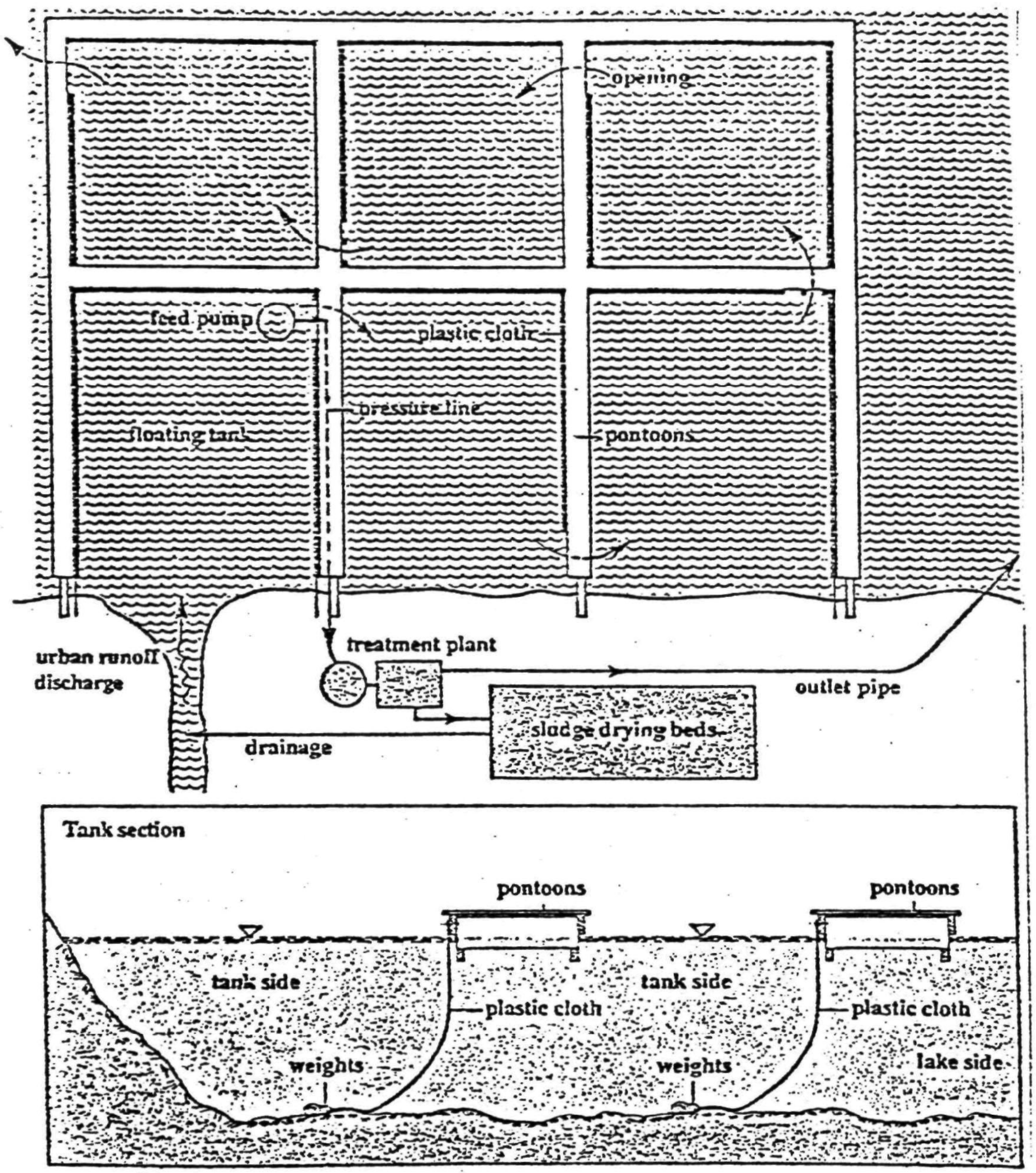


Figure 10. Lake-Flow Balance System

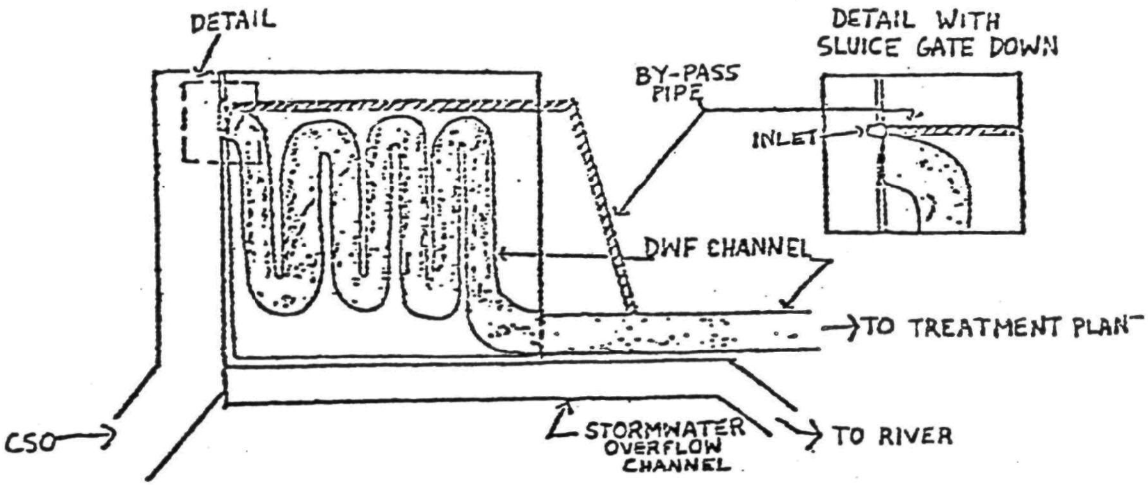


Figure 11. Self-Cleaning Storage/Sedimentation Basin

(Source: Trip Report, R. Field, Chief, Storm and Combined Sewer Section, USEPA-MERL, October, 1978).

References:

EPA-600/2-77-046 - Cottage Farm Combined Sewer Detention and Chlorination Station, Cambridge, MA: by Commonwealth of MA Metropolitan District Commission, MA, November, 1977.
NTIS ONLY PB 263 292

EPA-600/2-76-222a Wastewater Management Program, Jamaica Bay-Vol I. Summary Report: by D.L. Feurstein and W.O. Maddaus, City of New York, NY, September, 1976.
NTIS PB 260 887

EPA-600/2-76-222b Wastewater Management Program, Jamaica Bay-Vol II, Supplemental Data, NYC Spring Creek: by D.L. Feurstein and W.O. Maddaus, City of New York, NY, 1976.
NTIS ONLY PB 258 308

EPA-670/2-74-075 - Surge Facility for Wet- and Dry-Weather Flow Control: by H.L. Welborn, City of Rohnert Park, CA, November, 1974.
NTIS PB 238 905

EPA-600/2-75-071 - Detention Tank for Combined Sewer Overflow - Milwuakee, WI Demonstration Project: by Consoer, Townsend and Associates, Chicago, IL, December, 1975.
NTIS PB 250 427

11023—08/70 - Retention Basin Control of Combined Sewer Overflows: by Springfield Sanitary District, Springfield, IL, August, 1970.
NTIS PB 200 828

11020FAQ03/71 - Dispatching Systems for Control of Combined Sewer Losses: By Metro. Sewer Board, St. Paul, MN, March, 1971.
NTIS PB 203 678

EPA-R2-72-070 - Storage and Treatment of Combined Sewer Overflows: by the City of Chippewa Falls, WI, October, 1972.
NTIS PB 214 106

11022ELK12/71 - Maximizing Storage in Combined Sewer Systems: by Municipality of Metropolitan Seattle, WA, December, 1971.
NTIS ONLY PB 209 861

EPA-670/2-75-020 - Sewerage System Monitoring and Remote Control: by T.R. Watt et al., Detroit Metro Water Dept., Detroit, MI, May, 1975.
NTIS PB 242 107

EPA-670/2-74-022 - Computer Management of a Combined Sewer System: by C.P. Leiser, Municipality of Metropolitan Seattle, Seattle, WA, July, 1974.
NTIS PB 235 717

EPA-670/2-75-010 - Multi-Purpose Combined Sewer Overflow Treatment Facility, Mount Clemens, Michigan: by V.U. Mahida et al., Spalding, DeDecker & Associates, Inc., Madison Heights, MI, May, 1975.
NTIS PB 242 914

TREATMENT

PHYSICAL WITH/WITHOUT CHEMICAL

Physical-chemical processes are of particular importance in CSO treatment because of their adaptability to automatic operation (including almost instantaneous startup and shutdown), excellent resistance to shockloads, and ability to consistently produce a low SS effluent.

In this paper, physical-chemical systems will be limited to screening, filtration, chemical clarification, and dissolved air flotation.

Screening

Screens have been used to achieve various levels of SS removal contingent with three modes of screening process applications.

- Main treatment - screening is used as the primary treatment process.
- Pretreatment - screening is used to remove suspended and coarse solids prior to further treatment to enhance the treatment process or to protect downstream equipment.
- Dual use - screening provides either main treatment or pretreatment of stormwater and is used as an effluent polisher during periods of dry weather.

Screens can be divided into four categories: (1) bar screens (>25.4 mm (>in.) openings), (2) coarse screens (25.4-4.8 mm (1-3/16 in.)), fine screens (4.8-0.1 mm (3/16-1/250 in.)), and (4) microscreens (<0.1 mm (<1/250 in.)). No special studies have been made to evaluate bar and coarse screens in relation to CSO, so the basis for design should be the same as for their uses in dry-weather treatment facilities. Because CSO contains a significant amount of coarse debris, which is aesthetically undesirable, providing coarse screening as the minimum CSO treatment may be useful. Fine screens and microscreens are discussed, together because in most cases, they operate in a similar manner.

Several distinct types of screening devices have been developed and used for SS removal from CSO, and are described in Table 11.

Design parameters for static screens, microstrainers, drum screens, disc screens, and rotary screens are presented in Tables 12, 13, and 14.

Removal efficiency of screening devices is adjustable by changing the aperture size of the screen placed on the unit, making these devices very versatile. Solids removal efficiencies are affected by two mechanisms: (1) straining by the screen, and (2) filtering of smaller particles by the mat (schmutzdecke) deposited by the initial straining. The efficiencies of screens treating a waste with a typical distribution of particle sizes will increase as the size of screen opening decreases.

Table 11. Description of Screening Devices Used in CSO Treatment

Type of screen	General description	Process application	Comments
Drum screen	Horizontally mounted cylinder with screen fabric aperture in the range of 100 to 841 microns. Operates at 2 to 7 r/min.	Pretreatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Microstrainers[a]	Horizontally mounted cylinder with screen fabric aperture in the range of 23 to 100 microns. Operates at 2 to 7 r/min.	Main treatment	Solids are trapped on inside of drum and are backwashed to a collection trough.
Rotostrainer	Horizontally mouned cylinder made of parallel bars perpendicular to axis of drum. Slot spacing in the range of 250 to 2500 microns. Operates at 1 to 10 r/min.	Pretreatment	Solids are retained on surface of drum and are removed by a scraper blade.
Disc strainer	Series of horizontally mounted woven wire discs mounted on a center shaft. Screen aperture in the range of 45 to 500 microns. Operates at 5 to 15 r/min.	Pretreatment, main treatment, or post treatment of concentrated effluents	Unit achieves a 12 to 15% solids cake.
Rotary screen	Vertically aligned drum with screen fabric aperture in the range of 74 to 167 microns. Operates at 30 to 65 r/min.	Main treatment	Splits flow into two distinct streams: unit effluent and concentrate flow, in the proportion of approximately 85:15.
Static screen	Stationary inclined screening surface with slot spacing in the range of 250 to 1600 microns.	Pretreatment	No moving parts. Used for removal of large suspended and settleable solids.

a. A vertically mounted microstrainer is available, which operates totally submerged and operates at approximately 65 r/min. Aperture range 10 to 70 microns. Solids are moved from the screen by a sonic cleaning device.

(EPA-600/8-77-014)

The second most important condition affecting removal efficiencies, especially for microstrainers, is the thickness of filtered material on the screen. Whenever the thickness of this filter mat is increased, the suspended matter removal will also increase because of the decrease in effective pore size and the filtering action of the filtered mat. This will also increase head loss across the screen.

It was found, during experimental microstrainer operation in Philadelphia, that because of extreme variation of the influent SS concentration of CSO, removal efficiency would also vary, while effluent concentration remained relatively constant. For example, an effluent concentration of 10 mg/l SS would yield a reduction of 99.0 percent for an influent concentration of 1,000 mg/l (representative of "first-flush"),whereas, the SS reduction would be only 50 percent if the influent concentration were 20 mg/l (representative of tail end of storm). This phenomenon is apt to recur in other physical-chemical stormwater treatment operations. (R. Field and E. Struzeski, JWPCF, Vol. 44, No. 7, July, 1972).

Table 12. Design Parameters for Static Screens

Hydraulic loading, gal/min per ft of width	100-180
Incline of screens, degrees from vertical	35[a]
Slot space, microns	250-1 600
Automatic controls	None

a. Bauer Hydrasieves (TM) have 3-stage slopes on each screen: 25°, 35°, and 45°.

gal/min·ft x 0.207 = l/m·s

(EPA-600/8-77-014)

Table 13. Design Parameters for Microstrainers, Drum Screens, and Disc Screens

Parameter	Microstrainers	Drum screen	Disc screens
Screen aperture, microns	23-100	100-420	45-500
Screen material	Stainless steel or plastic	Stainless steel or plastic	wire cloth
Drum speed, r/min			
Speed range	2-7	2-7	5-15
Recommended speed	5	5	....
Submergence of drum, %	60-80	60-70	50
Flux rate, gal/min per ft^2 of submerged screen	10-45	20-50	20-25
Headloss, in.	10-24	6-24	18-24
Backwash			
Volume, % of inflow	0.5-3	0.5-3	...[a]
Pressure, lb/in^2	30-50	30-50	....

a. Unit's waste product is a solids cake of 12 to 15% solids content.

$gal/min \cdot ft^2 \times 2.445 = m^3/h \cdot m^2$
in. x 2.54 = cm
ft x 0.305 = cm
$lb/in.^2 \times 0.0703 = kg/cm^2$

(EPA-600/8-77-014)

Table 14. Design Parameters for Rotary Screens

Screen aperture, microns	
Range	74-167
Recommended aperture	105
Screen material	Stainless steel or plastic
Peripheral speed of screen, ft/s	14-16
Drum speed, r/min	
Range	30-65
Recommended speed	55
Flux rate, gal/ft^2-min	70-150
Hydraulic efficiency, % of inflow	75-90
Backwash	
Volume, % of inflow	0.02-2.5
Pressure, lb/in^2	50

ft/s x 0.305 = m/s
gal/ft^2-min x 2.445 = m^3/m^2-h
lb/in^2 x 0.0703 = kg/cm^2

(EPA-600/8-77-014)

Microstrainers and fine screens remove from 25 to 90 percent of the SS, and from 10 to 70 percent of the BOD_5, depending on the size of screens used, and the type of wastewater being treated.

At Philadelphia, polyelectrolyte addition (0.25-1.5 mg/l) improved the operating efficiency of the microstrainer. Suspended solids removal increased from 70 percent to 78 percent and the average effluent SS was reduced from 40 to 29 mg/l. The flux also increased from an average of 56.2 $m^3/m^2 \cdot h$ to (23 gal/ft^2-min) to 95.4 $m^3/m^2 \cdot h$ (39 gal/ft$^2 \cdot$ min). After an extensive laboratory coagulation study, moderately charged, high molecular weight cationic polyelectrolytes were found to be the most suitable for this application.

Microstrainer, drum screen, static screen, and rotary screen performances as a function of influent SS concentration, for several experimental projects, are shown in Figures 12, 13, 14, and 15, respectively.

Costs of screening facilities are presented in Table 15.

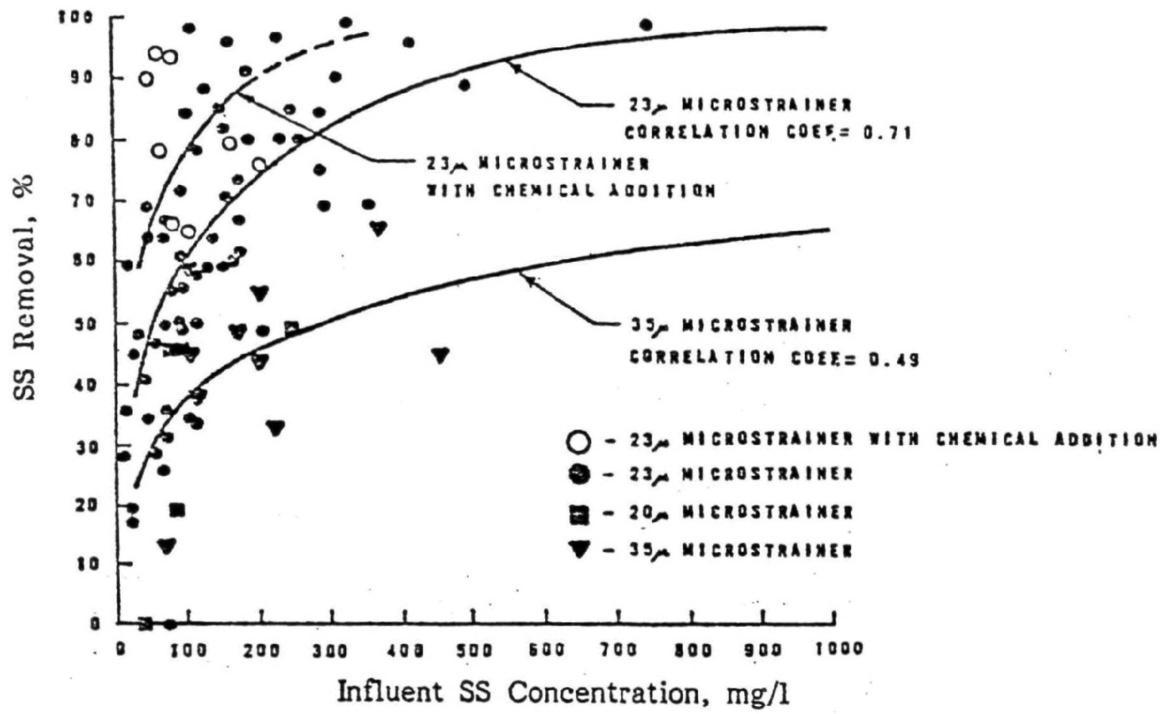


Figure 12. Microstrainer Performance as a Function of SS Concentration (EPA-600/8-77-014)

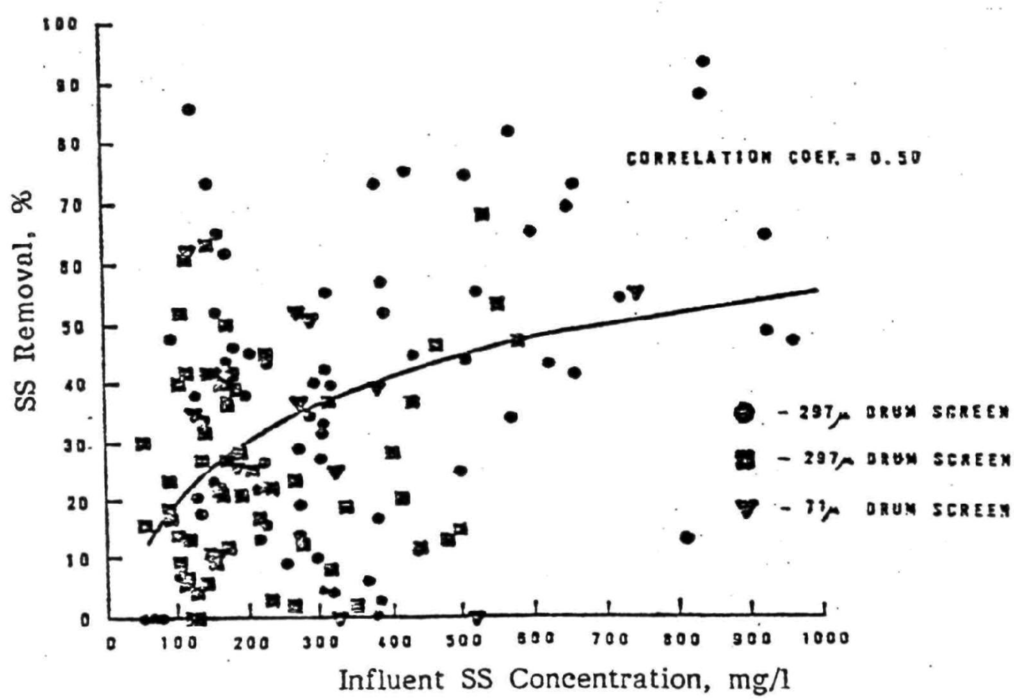


Figure 13. Drum Screen Performance as a Function of SS Concentration (EPA-600/8-77-014)

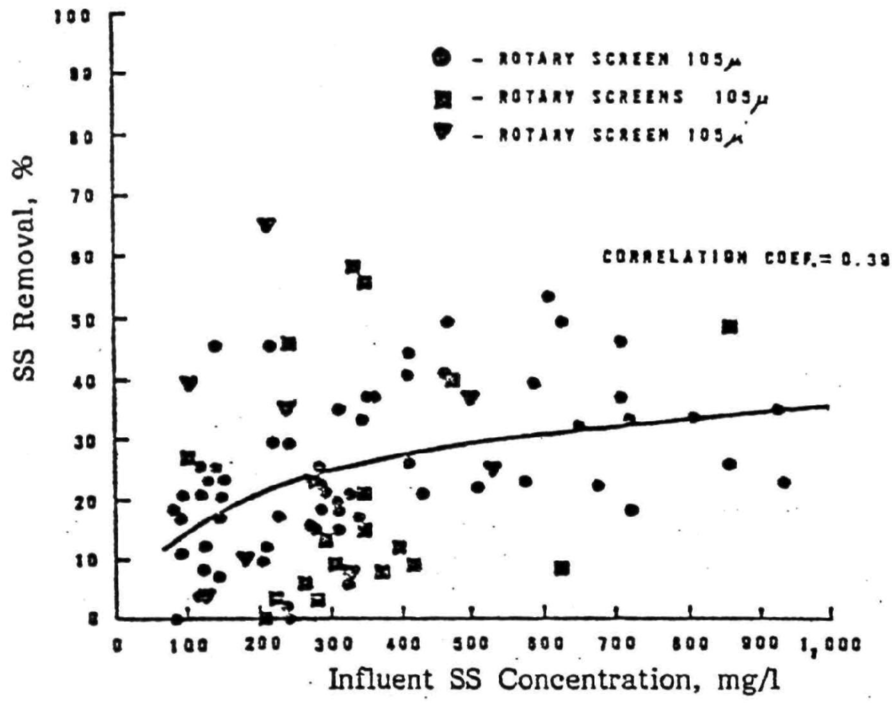


Figure 14. Static Screen Performance as a Function of SS Concentration

(EPA-600/8-77-014)

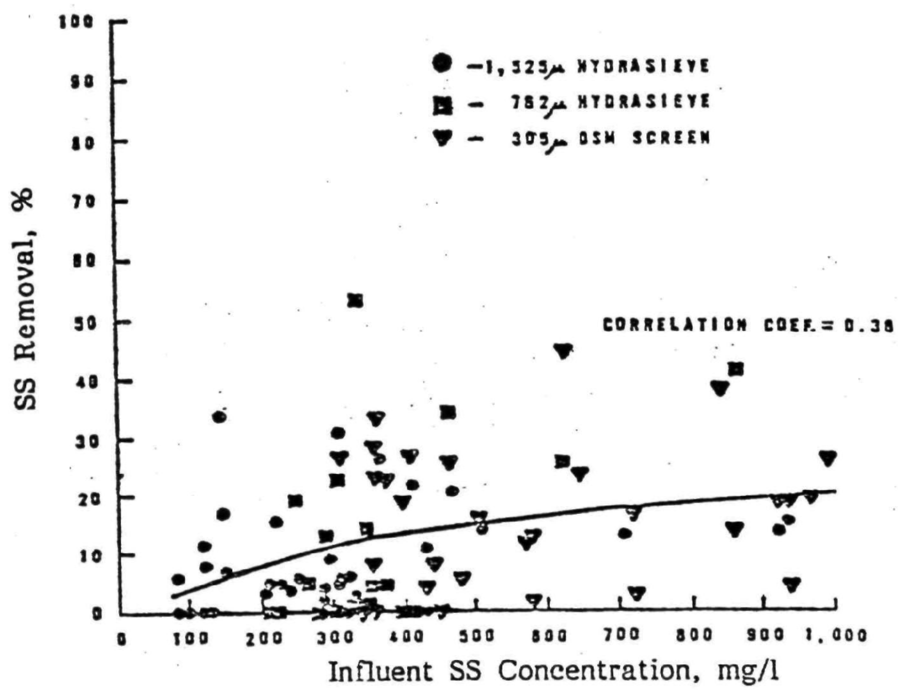


Figure 15. Rotary Screen Performance as a Function of SS Concentration

(EPA-600/8-77-014)

Table 15. Cost Summary of Selected Screening Alternatives[a]

Project location	Type of screen	Screening capacity, Mgal/d	Capital cost, $	Cost, $/Mgal/d	Annual operation and maintenance cost, $/1000 gal
Belleville, Ontario (1)	Rotary screen	1.8	57,800	3[illegible],100	0.143
		5.4	168,600	30,900	0.143
		7.2	221,600	30,700	0.143
	Static screen	0.75	25,700	34,300	0.073
		5.3	165,000	31,400	0.073
		7.5	225,600	30,000	0.073
Cleveland, Ohio(2),b,c	Drum screen	25	1,050,300	42,000	.....
		50	1,532,300	30,600	.....
		100	3,012,200	30,100	.....
		200	5,765,400	28,800	.....
Ft. Wayne, Indiana	Static screen	18	470,200	26,100	0.035
	Drum Screen	18	438,500	24,300	0.067
	Rotary screen	38	1,009,200	26,600	0.079
Mt. Clemens, Michigan(3)	Microstrainer	1.0	45,200	45,200	.....
Philadelphia, Pennsylvania	Microstrainer with chemical addition	7.4	156,700	21,200	0.083
	Microstrainer without chemical addition	7.4	255,300	34,500	0.085
Racine, Wisconsin	Drum Screen	3.9	39,000	10,000	.....
Seattle, Washington(4),b	Rotary screen	25	1,035,600	41,400	0.169
Syracuse, New York (5),c	Rotary Screen	5	223,500	44,700	.....
	Drum screen	10	443,600	44,400	.....

a. ENR =3452.
b. Estimated costs for several sizes of facilities.
c. Estimates include supplemental pumping stations and appurtenances

(1) Operational Data for the Belleville Screening Project. Ontario Ministry of the Environment. August 6, 1976.
(2) EPA 11023EYI04/72
(3) EPA-670/2-75-010
(4) EPA 11023FDD03/70
(5) EPA-600/2-76-286

Mgal/d x 0.0438 = m^3/s
$/1000 gal x 0.264 = $\$/m^3$

(EPA-600/8-77-014)

References

EPA-600/2-79-031b - Combined Sewer Overflow Abatement Program, Rochester, NY - Volume II - Pilot Plant Evaluations: by F. Drehwing et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, July, 1979.
NTIS PB 80-159262

EPA-670/2-74-049 - Microstraining and Disinfection of Combined Sewer Overflows- Phase III: by M.B. Maher, Crane Co., King of Prussia, PA, August, 1974.
NTIS PB 235 771

11023EVO06/70 - Microstraining and Disinfection of Combined Sewer Overflows: by Cochrane Div., Crane Co., King Of Prussia, PA, June, 1970.
NTIS PB 195 67

EPA-R2-73-124 - Microstraining and Disinfection of Combined Sewer Overflows-Phase II: by G.E. Glover, and G.R. Herbert, Crane Co., King of Prussia, PA, January, 1973.
NTIS PB 219 879

EPA-600/2-79-106a - Screening/Flotation Treatment of Combined Sewer Overflows: Volume II: Full-Scale Operation, Racine, WI: by T.L. Meinholz, Envirex, Inc., Milwaukee, WI, August, 1979.
NTIS PB 80-130693

EPA-600/2-77-069a - Screening/Flotation Treatment of Combined Sewer Overflows, Volume I - Bench-Scale and Pilot Plant Investigations: by M.K. Gupta et al., Envirex, Environmental Science Div., Milwaukee, WI, August, 1977.
NTIS PB 272 834

EPA-600/2-79-085 - Combined Sewer Overflow Treatment by Screening and Terminal Ponding - Fort Wayne, IN: by D.H. Prah and P.T. Brunner, City of Fort Wayne, IN, August, 1979.
NTIS PB 80-119399

11020FDC01/72 - Screening/Flotation Treatment of Combined Sewer Overflows: by the Ecology Division, Rex Chainbelt, Inc., January, 1972.
No NTIS Number

11023FDD07/71 - Demonstration of Rotary Screening for Combined Sewer Overflows: by City of Portland, Dept. of Public Works, Portland, OR, July, 1971.
NTIS PB 206 814

11023FDD03/70 - Rotary Vibratory Fine Screening of Combined Sewer Overflows: by Cornell, Howland, Hayes and Merryfield, Corvallis, OR, March, 1970.
NTIS PB 195 168

11020EXV07/69 - Strainer/Filter Treatment of Combined Sewer Overflows: by Fram Corporation, East Providence, RI, July, 1969.
NTIS ONLY PB 185 949

Screening/Dual Media High-Rate Filtration

Dual Media high-rate filtration (DMHRF) (>20$m^3/m^2 \cdot h$ (> 8 gal/$ft^2 \cdot$min)) removes small size particulates that remain after screening, and floc remaining after polyelectrolyte and/or coagulant addition. Principal advantages of the proposed system are: high treatment efficiencies, automated operation, and limited space requirements.

To be most effective, filtration through media that are graded from coarse to fine in the direction of filtration is desirable. A uniform size, single specific gravity medium filter cannot conform to this principle since backwashing of the bed automatically grades the bed from coarse to fine in the direction of washing; however, the concept can be approached by using a two layer bed. A typical case is the use of coarse anthracite particles on top of less coarse sand. Since anthracite is less dense than sand, it can be coarse and still remain on top of the bed after the backwash operation. Another alternative would be an upflow filter, but these units have limitations in that they cannot accept high filtration rates.

The principal parameters to be evaluated in selecting a DMHRF system are media size, media depth and filtration rate. Since much of the removal of solids from the water takes place within filter media, their structure and composition is of major importance. Too fine a medium may produce a high quality effluent but also may cause excessive head losses and extremely short filter runs. On the other hand, media that is too coarse may fail to produce the desired clarity of the effluent. Therefore, the selection of media for DMHRF should be made by pilot testing using various materials in different proportions and at different flow rates. Depth of media is limited by head loss and backwash considerations. The deeper the bed, the greater the head loss and the harder it is to clean. On the other hand, the media should be of sufficient depth so as to be able to retain the removed solids within the depth of the media for the duration of the filter run at the design flux rate without permitting breakthrough. The design filtration flux must be such that the effluent will be of a desired quality without causing excessive head loss through the filter, which in turn requires frequent backwashing. At high flux, shear forces seem to have significant effect on solids retention and removal.

Several DMHRF pilot study installations have been demonstrated for control of CSO pollution. These facilities have used 15.2, 30.5, and 76.2 cm (6, 12, and 30 in.) diameter filter columns with anthracite and sand media, together with various dosages of coagulants and/or polyelectrolytes. Descriptions of the DMHRF facilities are summarized in Table 16.

Suspended solids removal by DMHRF was found to vary directly with influent SS concentrations and inversely with flux or hydraulic loading rate. Experimental results have shown that SS removals from CSO increase appreciably with appropriate chemical additions (New York City; Cleveland).

DMHRF treatment of CSO at New York City's dual-use facility, (40 $m^3/m^2 \cdot h$ (16 gal/$ft^2 \cdot$min) constant flux) provided overall average SS removals of 61 percent across the filter and 66 percent across the system with an average influent SS concentration of 182 mg/l. Average SS removals for the three testing modes (no chemicals, polymer only, polymer and alum) and test ranges are shown in Table 17.

Table 16. Description of CSO-DMHRF Pilot Plant Demonstration Facilities[a]

Project Location	Process description	No. of filter columns	Diameter of columns, in.	Pretreatment facilities	Filter media*	Period of operation
Cleveland, Ohio	Pilot deep bed, dual media high rate filtration, with chemical addition. Facilities include pretreatment, storage, and filtration.	3 1	6 12	420 micron drum screen	5 ft of No. 3 anthracite over 3 ft of No. 612 sand	1970 to 1971
New York City, New York, Newtown Creek	Pilot deep bed, dual media high rate filtration, with polyelectrolyte addition. Facilities include pretreatment, storage, and filtration. Dry-weather and combined sewer flow is pumped from grit chamber of Newtown Creek plant.	1 2	30 6	420 micron rotostrainer later replaced with a 420 micron disc strainer	5 ft of No. 3 anthracite over 2 ft of No. 612 sand	1975 to present
Rochester, New York	Pilot deep bed, dual media high rate filtration with chemical addition.	3	6	swirl separator	5 ft of No. 1-1/2 or No. 2 anthracite over 3 ft of No. 1220 sand	1975 to 1976

a. Systems operate at flux rates ranging from 20 to 73 $m^3/m^2 \cdot h$ (8 to 30 $gal/ft^2 \cdot min$).

in. x 2.54 = cm
ft x 0.305 = m

*Media	Effective Size (mm)	Uniformity Coefficient
No. 3 Anthracite	4.0	1.5
No. 2 Anthracite	1.78	1.63
No. 1½ Anthracite	0.98	1.73
No. 612 Sand	2.0	1.32
No. 1220 Sand	0.95	1.41

(EPA-600/8-77-014)

Table 17. CSO-DMHRF Average SS Removals (New York City)

	Plant Influent (mg/l)	Filter Influent (mg/l)	Filter Effluent (mg/l)	Filter Removals (%)	System Removals (%)
No chemicals	175	150	67	55	62
Poly only	209	183	68	63	67
Poly & alum	152	143	47	67	69

(EPA-600/2-79-015)

A measure of the capability of a filter to remove SS, which is useful for predicting removals and filter-run cycle, is the specific capture, or mass capture. This can be expressed as pounds of solids removed per filter surface, or pounds of solids removed per media volume. Table 18 presents average SS mass captures obtained across the filter (New York City) during CSO tests of at least 3 hours duration, and the average for tests S-13, 14 and 16 which used more optimal chemical feeds and occurred during the storms of greatest intensity. It should be noted that these mass capture values are specific to the Newtown Creek filter and the test conditions.

Table 18. DMHRF Average Mass Capture Of CSO (New York City)

	Capture per Filter Surface		Capture per Media Volume	
CSO Test Nos.	lb/ft^2/run*	lb/ft^2/hr	lb/ft^3/run**	lb/ft^3/hr
S4B, S9-16	3.7	0.76	0.54	0.11
S-13, 14, 16	5.2	1.2	0.76	0.17

* 1 lb/ft^2 = 4.88 kg/m^2
** 1 lb/ft^3 = 16.02 kg/m^3

(EPA-600/2-79-015)

BOD removals (New York City) from CSO averaged 32 percent across the filter and 41 percent across the system with an average influent BOD_5 of 136 mg/l. The removals improved with chemical additions. Average BOD_5 removals for the three testing modes and test ranges are shown in Table 19.

Table 19. CSO-DMHRF Average BOD_5 Removals (New York City)

	Plant Influent (mg/l)	Filter Influent (mg/l)	Filter Effluent (mg/l)	Filter Removals (%)	System Removals (%)
No chemicals	164	131	96	27	41
Poly only	143	129	84	35	41
Poly & alum	92	85	53	38	43

(EPA-600/2-79-015)

It should be noted that the nature of the CSO tested (for example, the presence of dissolved industrial organic contaminants), may account for variable BOD_5 removals.

Limited tests were also run (New York City) to determine heavy metals reduction. These results, shown in Table 20, represent composite samples.

Table 20. Removal Of Heavy Metals by DMHRF

	Cadmium	Chromium	Copper	Mercury	Nickel	Lead	Zinc
Average removal, %[a]	56	50	39	0	13	65	48

a. Concentration basis

(EPA-600/2-79-015)

Design parameters for DMHRF are shown in Table 21.

Costs of DMHRF facilities are summarized in Table 22. These costs are based on facilities similarly designed to that of the Cleveland demonstration project.

Comparison with alternate treatment systems show that DMHRF is cost-competitive with conventional sedimentation facilities for dual process (sanitary and CSO), or CSO treatment, yet DMHRF has only 5-7 percent the area requirements. For strict CSO treatment, DMHRF is competitive with dissolved air flotation and microstraining processess.

Table 21. Design Parameters for DMHRF

Filter media depth, ft	
No. 3 anthracite	4-5
No. 612 sand	2-3
Effective size, mm	
Anthracite	4
Sand	2
Flux rate, gal/ft^2·min	
Range	8-40
Design	24
Headloss, ft	5-30
Backwash	
Volume, % of inflow	4-10
Air	
Rate, standard ft^3/min·ft^2	10
Time, min	10
Water	
Rate, gal/ft^2-min	60
Time, min	15-20

ft x 0.305 = m
gal/ft^2·min x 2.445 = m^3/m^2·h
standard ft^3/min·ft^2 x 0.305 = m^3/m^2·min

(EPA-600/8-77-014)

Table 22. Summary of Costs[a] for DMHRF Facilities

Plant capacity Mgal/d	Construction costs, $[b]		Construction costs, $/Mgal·d		Operation and maintenance costs, $/yr	
	24 gal/ft^2·min	16 gal/ft^2·min	24 gal/ft^2·min	16 gal/ft^2·min	24 gal/ft^2·min	16 gal/ft^2·min
25	2,485,000	2,900,000	99,400	116,000	76,000	78,000
50	3,745,000	4,522,000	74,900	90,400	95,000	98,000
100	6,870,000	8,388,000	68,700	83,900	169,000	176,000
200	11,668,000	13,843,000	58,300	69,200	223,000	231,000

a. ENR = 3452
b. Includes low lift pumping station, prescreening, and chemical addition facilities; and excludes engineering and administration.

Mgal/d x 0.0438 = m^3/s
gal/ft^2·min x 2.445 = m^3/m^2·min

(EPA-600/8-77-014)

References

EPA-600/2-79-015 - Dual Process High-Rate Filtration of Raw Sanitary Sewage And Combined Sewer Overflows: (Newtown Creek) by H. Innerfeld et al., New York City Dept. of Water Resources, New York, NY, April, 1979.
NTIS PB 296 626/AS

11023EYI04/72 - High-Rate Filtration of Combined Sewer Overflows: (Cleveland), by R. Nebolsine et al., Hydrotechnic Corp., New York, NY, April, 1972.
NTIS PB 211 144

EPA-600/2-79-031b - Combined Sewer Overflow Abatement Program, Rochester, NY - Volume II: Pilot Plant Evaluations: by F.J. Drehwing et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, July, 1979.
NTIS PB 80-159262

EPA-R2-73-222 - Ultra High-Rate Filtration of Activated Sludge Plant Effluent: by R. Nebolsine et al., Hydrotechnic Corp., New York, NY, April, 1973.
NTIS ONLY

Screening/Dissolved Air Flotation

Dissolved air flotation (DAF) is a unit operation used to separate solid particles or liquid droplets from a liquid phase. Separation is brought about by introducing fine air bubbles into the liquid phase. As the bubbles attach to the solid particles or liquid droplets, the buoyant force of the combined particle and air bubble is great enough to cause the particle to rise. Once the particles have floated to the surface, they are removed by skimming. The most common process for forming the air bubbles is to dissolve air into the waste stream under pressure, then releasing the pressure to allow the air to come out of solution. The pressurized flow carrying the dissolved air to the flotation tank is either (1) the entire stormwater flow, (2) a portion of the stormwater flow (split flow pressurization), or (3) recycled DAF effluent.

Higher overflow rates 3.2-25 $m^3/m^2 \cdot h$ (1.3-10.0 $gal/ft^2 \cdot min$) and shorter detention times (0.2-1.0 h) can be used for DAF than for conventional settling (1.7 $m^3/m^2 \cdot h$ (0.2-0.7 $gal/ft^2 \cdot min$); 1.0-3.0 h). This process has a definite advantage over gravity sedimentation when used on CSO since particles with densities both higher and lower than the liquid can be removed in one skimmimg operation. Dissolved air flotation also aids in the removal of oil and grease, which are not as readily removed during sedimentation.

The principal parameters that affect removal efficiencies are (1) overflow rate, (2) amount of air dissolved in the flows, and (3) chemical addition. Chemical addition has been used to improve removals, and ferric chloride has been the chemical most commonly added.

A treatment system consisting of screening, followed by DAF has been found to be an effective method of reducing pollutants in CSO. The basis of this system is that screening will remove particles that are too heavy for the air bubbles to carry. Average reported percent removals (Milwaukee - 18,925 m^3/d (5 Mgal/d) pilot), with and without chemical addition, are listed in Table 23.

Table 23. Percent Removals Achieved with Screening/DAF (Milwaukee)

	Without Chemical Flocculant Addition	With Chemical Flocculant Addition
BOD	35 ± 8	60 ± 11
COD	41 ± 8	57 ± 11
SS	43 ± 7	71 ± 9
Volatile SS	48 ± 11	71 ± 9
Nitrogen	29 ± 14	24 ± 9

(EPA-600-77-069a)

The chemical flocculant addition required to achieve the above stated removals was 20 mg/l ferric chloride, and 4 mg/l cationic polyelectrolyte. At the Racine prototype plants (54,126 m^3/d and 168,054 m^3/d (14.3 and 44.4 Mgal/d)), 40 mg/l ferric chloride, and 2 mg/l cationic polyelectrolyte were used.

The percent removals (concentration basis) are presented in Table 24.

Table 24. Percent Removals Achieved with Screening/DAF (Racine)

	Percent Removal	
	Site I	Site II
BOD	57.5	65.4
TOC	51.2	64.7
Suspended Solids	62.2	73.3
Volatile Suspended Solids	66.8	70.9
Total Phosphorus	49.3	70.0

(EPA-600/2-79-106a)

The results from Site II are better than Site I because the hydraulic loading was usually lower at Site II than at Site I, resulting in lower overflow rates and longer tank detention times at Site II.

Typical design parameters for DAF facilities are presented in Table 25.

Table 25. DAF Design Parameters

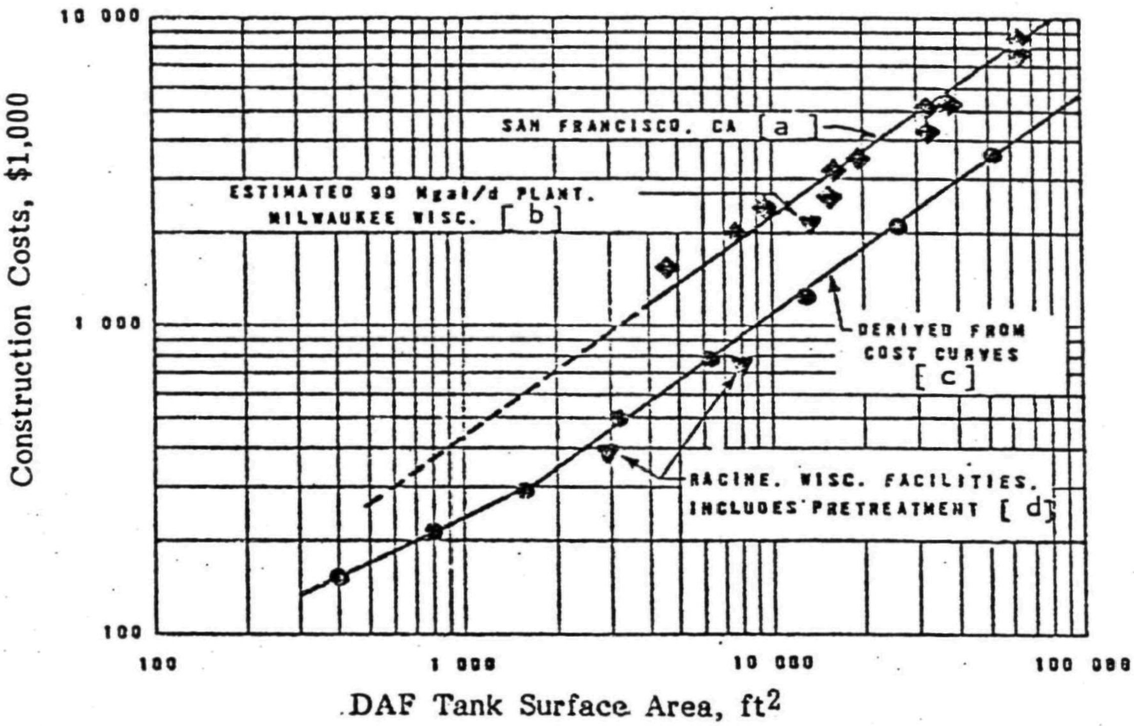
Overflow rate, gal/ft^2·min	
Low rate	1.3-4.0
High rate	4.0-10.0
Horizontal velocity, ft/min	1.3-3.8
Detention time, min	
Flotation cell range	10-60
Flotation cell average	25
Saturation tank	1-3
Mixing chamber	1
Pressurized flow, % of total flow	
Split flow pressurization	20-30
Effluent recycle pressurization	25-45
Air to pressurized flow ratio, standard ft^3/min·100 gal	1.0
Air to solids ratio	0.05-0.35
Pressure in saturation tank, lb/in^2	40-70
Float	
Volume, % of total flow	0.75-1.4
Solids concentration, % dry weight basis	1-2

gal/ft^2·min x 2.445 = m^3/m^2·h
ft/min x 0.00508 = m/s
standard ft^3/min·100 gal x 0.00747 = m^3/min·100 l
lb/in^2 x 0.0703 = kg/m^2

(EPA-600/8-77-014)

For the two full scale CSO test sites in Racine, WI, capital costs (including land) were $763,882 and $1,472,165 for 54,126 and 168,054 m^3/d (14.3 and 44.4 Mgal/d) facilities, respectively (ENR = 3452). Construction cost curves (ENR = 2000) for DAF facilities, based on the experienced cost of the demonstration facilities, are presented in Figure 16.

The operation and maintenance cost (ENR = 3452) for the systems was $0.11/cu m ($0.40/1,000 gal). It was thought that these costs would be reduced to $0.06/cu m ($0.21/1,000 gal) by process and procedural modifications as described in EPA-600/2-79-106a. The major reason for the high operation and maintenance cost is the cost of labor for maintenance of the sites and cleanup of the sites after a system operation. These costs were $0.07/cu m ($0.26/1,000 gal), or 65 percent of the total. Therefore, maintenance becomes the major cost item in the full-scale application of screening/DAF for the treatment of CSO.



a. Evaluation did not include screening; EPA-600/2-75-033.
b. Screening/flotation; EPA-600/2-77-069a.
c. Does not include screening; EPA-600/2-76-286.
d. Screening/flotation; EPA-600/2-79-106a.

$ft^2 \times 0.0929 = m^2$

$Mgal/d \times 0.0438 = m^3/s$

Figure 16. DAF Construction Costs (ENR = 2000)

(EPA-600/8-77-014)

References

EPA-600/2-79-106a - Screening/Flotation Treatment of Combined Sewer Overflows: Volume II: Full-Scale Operation, Racine, WI: by T.L. Meinholz, Envirex, Inc., Milwaukee, WI, August, 1979.
NTIS PB 80-130693

EPA-600/2-77-069a - Screening/Flotation Treatment of Combined Sewer Overflows, Volume I - Bench-Scale and Pilot Plant Investigations: by M.K. Gupta et al., Envirex, Environmental Science Division, Milwaukee, WI, August, 1977.
NTIS PB 272 834

EPA-600/2-75-033 - Treatment of Combined Sewer Overflows by Dissolved Air Flotation: by T.A. Bursztynsky et al., Engineering Science, Inc., Berkeley, CA, September, 1975.
NTIS PB 248 186

11020FDC01/72 - Screening/Flotation Treatment of Combined Sewer Overflows: Rex Chainbelt, Inc., Milwaukee, WI, January, 1972.
No NTIS Number

BIOLOGICAL TREATMENT

Biological treatment is a means for stabilizing dissolved organic matter, and removing nonsettleable colloidal solids. This can be accomplished either aerobically, or anaerobically. Several biological processes have been applied to CSO treatment. These include: contact stabilization, trickling filters, rotating biological contactors (RBC), and treatment lagoons. Descriptions of these processes and typical combined sewage treatment installations are provided in Tables 26 and 27.

Biological treatment processes are generally classified as secondary treatment processes, capable of removing between 70 and 95 percent of the BOD_5 and SS from waste flows at dry-weather design flowrates and loadings. When biological treatment processes are used for combined sewage treatment, removal efficiencies are lower (the percent organic matter is smaller for CSO solids than for dry-weather solids), and are controlled to a large degree by hydraulic and organic loading rates. Most biological systems are susceptible to overloading conditions and shock loads as compared to physical treatment processes. However, RBC's have achieved high removals at flows 8 to 10 times dry-weather design flows.

Typical pollutant removals for contact stabilization, trickling filters, and RBC's (wet-weather loading conditions), are presented in Table 28. These processes include primary (except contact stabilization) and final clarification. Final clarification greatly influences the overall performance of the system by preventing the carryover of biological solids produced by the processes. Pollutant removal efficiencies by treatment lagoons have varied from highs of 85 to 95 percent to negative values due to excessive algae production and carryover. In addition to the type of lagoon and the number of cells in series (stages), several major factors that influence removal efficiencies include: (1) detention time, (2) source of oxygen supply, (3) mixing, (4) organic and hydraulic loading rates, and (5) algae removal mechanisms. A single cell storage/oxidation lagoon in Springfield, IL, averaged 27 percent BOD_5 removal and 20 percent SS removal; however, fish kills in the receiving water were greatly reduced as compared to that prior to the construction of the facility. Multiple cell facilities with algae control systems constructed at Mount Clemens, MI, and Shelbyville, IL, provide 75 to 90 percent SS and BOD_5 removal efficiencies during wet-weather conditions.

An operational problem common to all stormwater biological systems is that of maintaining a viable biomass to treat flows during wet-weather conditions. At New Providence, NJ, trickling filters are operated in series during dry weather, and in parallel during wet weather. This type of operation maintains a viable microorganism population during dry weather and also provides greater capacity for the wet-weather flows. For processes that borrow biomass from dry-weather facilities or allow the biomass to develop, a lag in process efficiency may be experienced as the biomass becomes acclimated to the changing waste strength and flowrate. Also, because of the limited ability of biological systems to handle fluctuating and high hydraulic shock loads, storage/detention facilities preceding the biological processes may be required.

Table 26. Description of Biological Processes Used in CSO Treatment

Biological process	Process description	Source of Biomass	Requires additional treatment	Type of additional treatment
Contact stabilization	Process is a modified activated sludge process in which the absorption phase, or contact, and the oxidation phase (stabilization) takes place in two separate tanks. Sludge is wasted from the stabilization tank to maintain constant biomass concentrations.	From conventional activated sludge treatment facility.	Yes	Secondary clarification
Trickling filters	Standard trickling filter process in which a biological growth is supported on a stationary medium and the stormwater distributed over the surface and allowed to flow through the media. Process can include standard rate or deep bed plastic media designs.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Rotating biological contactors	Process operates on the same principle as trickling filters; however, the biological growth is supported on large diameter, closely spaced disks which are partially submerged and rotate at slow speeds.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Treatment lagoons				
Oxidation ponds	Shallow aerobic ponds which rely on surface reaeration for oxygen supply to maintain biological uptake of organics. Sedimentation also occurs in oxidation ponds.	Allowed to generate for each storm.	Optional	Final clarification, screening, or sand filtration
Aerated lagoons	Similar to oxidation ponds except they are deeper and rely on artificial means of oxygen supply such as surface aerators or diffused air systems. System operates under aerobic conditions.	Allowed to generate for each storm.	Optional	Final clarification, screening, or sand filtration
Facultative lagoons	Facultative lagoons are the deepest of the lagoons and rely on surface reaeration. The lagoons have three distinct layers: aerobic near the surface due to algae and reaeration, a transition zone, and an anaerobic zone near the bottom sludge deposits. The biological oxidation and anaerobic stabilization occur simultaneously.	Allowed to generate for each storm.	Optional	Final clarification, screening, or sand filtration

(EPA-600/8-77-014)

Table 27. Summary of Typical Biological CSO Treatment Installations

Project location	Type of biological treatment	Tributary area, acres	Design capacity, Mgal/d	Major process components	No. of units	Total size	Period of operation
Kenosha, Wisconsin	Contact stabilization	1,200	20	Contact tank	2	32,700 ft^3	1972 to 1975
				Stabilization tank	2	97,900 ft^3	
Milwaukee, Wisconsin	Rotating biological contactors	35	0.05[a]	3 ft diameter RBC units	24	28,300 ft^2	1969 to 1970
Mt. Clemens, Michigan							
Demonstration system	Treatment lagoons in series with recirculation between storms	212	1.0[b]	Storage/aerated lagoon	1	750,000 ft^3	1972 to 1975
				Oxidation lagoon	1	1,100,000 ft^3	
				Aerated lagoon	1	930,000 ft^3	
Citywide full scale system	Storage/treatment lagoons in series with recirculation between storms	1,471	4.0[b]	Aerated storage basin	1	4,440,000 ft^3	Under construction
				Aerated lagoon	1	500,000 ft^3	
				Oxidation lagoon	1	1,100,000 ft^3	
				Aerated/oxidation lagoon	1	922,000 ft^3	
New Providence, New Jersey	Trickling filters	.....	6.0	High-rate plastic media	1	36 ft diameter	1970 to present
				High-rate rock media	1	65 ft diameter	
Shelbyville, Illinois	Treatment lagoons:						
	Southeast site	44	28[c]	Oxidation lagoon	1	255,600 ft^3	1969 to present
	Southwest site	450	110	Detention lagoon plus 2-cell facultative lagoon	1	2,782,700 ft^3	1969 to present
Springfield, Illinois	Treatment lagoon	2,208	67	Storage/oxidation lagoon	1	6,330,000 ft^3	1969 to present

a. Design based on average dry-weather flow; average wet-weather flow = 1 Mgal/d.

b. Design flowrate through lagoon systems. Total flowrate to facilities is 64 Mgal/d for the demonstration project and 260 Mgal/d for citywide system.

c. Estimated using a 50% runoff coefficient at a rainfall rate of 1.95 in./h.

acres x 0.405 = ha
Mgal/d x 0.0438 = m^3/s
ft^3 x 0.0283 = m^3
ft^2 x 0.0929 = m^2
ft x 0.305 = m
in./h x 2.54 = cm/h

Table 28. Typical Wet-Weather BOD and SS Removals for Biological Treatment Processes

Biological treatment process	Expected range of pollutant removal, % BOD	COD
Contact stabilization	70-90	75-95
Trickling filters	65-85	65-85
RBC[a]	40-80	40-80

a. Removal reflects flow ranges from 30 to 10 times dry-weather flow.

(EPA-600/8-77-014)

General maintenance problems experienced by wet-weather biological facilities are similar to those experienced at conventional biological installations. Winter operation of mechanical surface aerators have had some serious drawbacks, including icing, tipping, or sinking. Other methods of providing the required oxygen that show promise and have been demonstrated at many dry-weather facilities include diffused air systems and submerged tube aerators.

A comparison of construction, and operation and maintenance costs for biological treatment systems and treatment lagoons is presented in Table 29. Costs of final clarification are included where control of solids and sludge produced by the biological treatment system are required. Costs also include pumping, disinfection, and algae control systems, when applicable. Engineering, administration, and land costs are not included in the estimates; however, land costs may be the controlling economic factor in the evaluation of lagoon treatment systems and therefore must be evaluated for each specific location. Biological CSO treatment systems are integrated with or are a part of dry-weather treatment facilities. Cost estimates of the wet-weather portion of these facilities were separated from total costs of the total treatment systems. The cost of the in-line RBC at Milwaukee, WI, was used together with an estimated cost for a final clarifier to develop an estimated cost of a complete RBC treatment system. The final clarifier cost was based on one 19.8 m (65 ft) diameter clarifier with a surface loading rate of 2.04 $m^3/m^2 \cdot h$ (1,200 $gal/ft^2 \cdot d$). Costs of lagoon treatment systems vary widely, and are a function of the type of lagoon (oxidation, aerated, or facultative), the number of cells, and the miscellaneous equipment requirements including: aeration equipment, disinfection equipment, instrumentation, pumping, and algae control provisions. Costs for many of these CSO facilities are based on only one installation of each biological treatment process. Therefore, these costs should be considered only coarse estimates and may be greatly influenced by the degree of integration with dry-weather treatment required to produce a viable system. These costs can be used as a preliminary guide, but detailed analyses should be performed to compare and evaluate biological treatment alternatives with other methods of treatment and control.

Initial capital investments of integrated dual use facilities can be reduced by apportioning part of the costs to the dry-weather facility. The cost reduction is in proportion to the net benefit that the wet-weather facility provides to the overall treatment efficiency during dry-weather periods. A description of this evaluation is presented in Section 4 of EPA-600/8-77-014, Urban Stormwater Management and Technology: Update and User's Guide.

Table 29 . Summary of Capital and Operation and Maintenance Costs for Biological Treatment Alternatives[a]

Project location	Type of biological treatment	Peak plant capacity, Mgal/d	Construction cost, $	Cost/ capacity, $/Mgal·d	Cost/ tributary area, $/acre	Annual operation and maintenance cost, ¢/1,000 gal (except as noted)
Kenosha, Wisconsin	Contact stabilization	20	2,354,300	117,700	1,970	23.8
Milwaukee, Wisconsin[b]	Rotating biological contactor	4.3	516,100	119,400	14,740	7.6
Mount Clemens, Michigan						
Demonstration system	Aerated treatment lagoons	64	1,105,300	17,300	5,230	34.5
Citywide system	Storage/aerated treatment lagoons	260	9,902,100	38,000	6,730	32.8
New Providence, New Jersey[c]	High-rate trickling filter	6	819,900	136,600	.....	21.2
Shelbyville, Illinois						
Southeast site	Oxidation lagoon	28	74,900	2,680	1,730	2,640/yr[d]
Southwest site	Storage and facultative lagoons	110	582,900	5,300	1,300	9,980/yr[d]
Springfield, Illinois	Oxidation lagoon	67	303,800	4,500	140	3,630/yr

a. ENR = 3452
b. Includes estimate of final clarifier.
c. Includes plastic media trickling filter, final clarifier, plus one-half of other costs.
d. Based on estimated man-day labor requirements.

Mgal/d x 0.0438 = m^3/s
acres x 0.405 = ha
¢/1,000 gal x 0.264 = ¢/m^3

(EPA-600/8-77-014)

References

EPA-670/2-74-050 - Combined Sewer Overflow Treatment by the Rotating Biological Contactor Process: by F.L. Welsh, and D.J. Stucky, Autotrol Corp., Milwaukee, WI, June, 1974.
NTIS PB 231 892

EPA-670/2-75-019 - Biological Treatment of Combined Sewer Overflow at Kenosha, WI: by R.W. Agnew et al., Envirex, Milwaukee, WI, April 1975.
NTIS PB 242 126

EPA-670/2-73-071 - Utilization of Trickling Filters for Dual-Treatment of Dry and Wet-Weather Flows: by P. Homack et al., E.T. Killam Assoc., Inc., Millburn, NJ, September, 1973.
NTIS PB 231 251

EPA-670/2-74-079 - An Evaluation of Three Combined Sewer Overflow Treatment Alternatives (Shelbyville, IL): by J.W. Parks et al., Urbana, IL, August, 1974.
NTIS ONLY PB 239 115

11023—08/70 - Retention Basin Control of Combined Sewer Overflows: by Springfield Sanitary District, Springfield, IL, August, 1970.
NTIS PB 200 828

EPA-670/2-75-010 - Multi-Purpose Combined Sewer Overflow Treatment Facility, Mount Clemens, Michigan: by V.U. Mahida et al., Spalding, DeDecker & Associates, Inc., Madison Heights, MI, May, 1975.
NTIS PB 242 914

Post-Construction Evaluation of the East Chicago, Indiana CSO Treatment Lagoon: by D.J. Connick et al., Environmental Design & Planning, Inc., Allston, MA, Grant No. 11023 FAV (Publication Pending).

ADVANCED TREATMENT

In this report, advanced treatment includes high gradient magnetic separation, and powdered activated carbon-alum coagulation. The high pollutant removals achievable with these processes may not always be necessary, or cost-effective for CSO, but if a high degree of treatment is required, the processes may become more attractive.

High Gradient Magnetic Separation

High gradient magnetic separation (HGMS) is a new treatment technology applied to CSO management. In its simplest form, the high gradient magnetic separator consists of a canister packed with a fibrous ferromagnetic material which is magnetized by a strong external magnetic field (coils surround the canister). An iron frame increases the efficiency of the electromagnetic coils. The device operates in a sequence of feed and flush modes. The magnetic particles are trapped on the edges of the magnetized fibers while the non-magnetic particles and slurry fluid pass through the canister. The matrix offers only a small hydraulic resistance to the feed flow, occupying less than 5 percent of the canister volume (95 percent void volume). When the matrix has become loaded with magnetic particles, the particles are easily washed from the matrix by reducing the magnetic field to zero and opening valves and backflushing. High gradient magnetic separation may also be used to remove non-magnetic contaminants from water. This is accomplished by binding finely divided magnetic seed particles, such as magnetic iron oxide (magnetite), to the non-magnetic contaminants, thus creating a "magnetic handle" ("indirect filtration" or "seeded water treatment"). Binding of the magnetic seed is accomplished in two general ways: adsorption of the contaminant to magnetic seed and chemical coagulation (alum). Particles ranging in size from soluble through settleable (>0.001μ) may be removed with this process. Design parameters for HGMS are presented in Table 30.

Table 30. Preliminary Design Parameters for High Gradient Magnetic Separators

Magnetic field strength, kG[a]	0.5-1.5
Maximum flux rate, gal/ft^2-min	100
Minimum detention time, min	3
Matrix loading, g solids/g of matrix fiber	0.1-0.5
Magnetite addition, mg/l	100-500
Magnetite to suspended solids ratio	0.4-3.0
Alum addition, mg/l	
Range	90-120
Average	100
Polyelectrolyte addition, mg/l	0.5-1.0

a. kG = kilogauss

gal/ft^2·m x 2.445 = m^3/m^2·h

(EPA-600/8-77-014)

Magnetic separation can provide the rapid filtration of many pollutants from water, with a small expenditure of energy. Removal is much more efficient than with sedimentation because the magnetic forces on fine particles may be many times greater than gravitational forces. To date, only bench scale tests and a pilot plant scale system of 1 to 4 l/min (0.26 to 1.06 gal/min) have been operated.

Typical pollutant removals are shown in Tables 31, 32, and 33.

Table 31. Removal of Solids by HGMS for CSO and Raw Sewage Samples[a]

	Removal, %[b]	
Solids parameter	CSO	Raw sewage
SS	95	91
Settleable solids	99+	99+
Apparent color, PCU	87	82
Turbidity, FTU	93	88

a. All samples concentration basis except as noted.

b. Operated at 1 to 4 l/min (0.26 to 1.06 gal/min), (3 to 12 min residence times).

(EPA-600/8-77-014)

Table 32. Removal of Biological and Chemical Constituents by HGMS

Pollutant parameter	Avg. removal, %
BOD	92
COD	74
Total coliforms on EMB agar at 37°C	99.3
Fecal coliforms on EMB agar at 37°C	99.2
Algae	99.9
Virus, bacteriophage T_7	100
Virus, polio	99-100

(EPA-600/8-77-014)

Table 33. Removal of Heavy Metals by HGMS

	Heavy metal constituent						
	Cadmium	Chromium	Copper	Mercury	Nickel	Lead	Zinc
Average removal, %	43	41	53	71	0-67	0-67	84

(EPA-600/8-77-014)

Costs of HGMS have been evaluated for a 94625 m^3/d (25 Mgal/d) facility, and are summarized in Table 34. Capital costs include pretreatment, chemical addition, thickening and dewatering equipment, pumps, backflush system, instrumentation, and disinfection system. Operation and maintenance costs include chemicals, labor, electrical utilities, and maintenance.

Table 34. Construction and Operation and Maintenance Costs[a] for a 25 Mgal/d HGMS Facility

Construction cost	
Total, $	3,647,000
$/Mgal·d	145,800
Operation and maintenance cost	
$/yr	938,900
$/1,000 gal treated	0.21

a. ENR = 3452

Mgal/d x 0.0438 = m^3/s
1,000 gal x 3.785 = m^3

(EPA-600/8-77-014)

Powdered Activated Carbon - Alum Coagulation

Several combined sewage treatment demonstration projects have evaluated the benefits of chemical aids to process operations, but only one pilot operation representing a complete physical chemical treatment system has been implemented. It was demonstrated at a 379 m^3/d (100,000 gal/d) pilot unit in Albany, NY. In this project, raw municipal sewage and CSO were mixed with powdered activated carbon, to remove dissolved organics. Alum was then added to aid in subsequent clarification. Addition of polyelectrolyte was followed by a short flocculation period. Solids were separated from the liquid stream by gravity settling, and the effluent was then disinfected and discharged, or filtered (tri-media), then disinfected prior to discharge. Carbon regeneration in a fluidized bed furnace and alum recovery from the calcined sludge were also demonstrated, as was reuse of the reclaimed chemicals. Average carbon losses per regeneration cycle were 9.7 percent. Average removals in excess of 94 percent COD, 94 percent BOD, and 99 percent SS were consistently achieved (without filtration) in treating combined sewage.

Representative capital and operation and maintenance costs for a physical-chemical treatment plant designed for raw stormwater treatment, projected from data developed during the Albany project, are summarized in Table 35.

Table 35. Estimated Capital and Operation and Maintenance Costs for a Physical-Chemical Treatment Plant

Capital Costs,* $				Operation and Maintenance costs, ¢/1000 gal			
1 Mgal/d	10 Mgal/d	25 Mgal/d	100 Mgal/d	1 Mgal/d	10 Mgal/d	25 Mgal/d	100 Mgal/d
309,180	3,091,800	6,289,400	18,416,600	3.3	32.5	26.9	20.2

ENR = 3452

Mgal/d x 0.0438 = m^3/s
¢/1000 gal x 0.264 = ¢/m^3

* Capital costs include screens, grit chambers, overflow facilities, pipe reactor vessels, pumps, chemical storage, carbon slurry tanks, sludge storage, agitators, flocculators, tube settlers, filtration, chlorination, carbon regeneration/sludge incineration, fluidized bed furnace, chemical make-up system, 10 percent contingencies, and land. Operation and maintenance costs include all materials, power, and labor. Plant is designed for raw stormwater treatment.

(EPA-670/2-74-040)

Reference

EPA-R2-73-149 - Physical-Chemical Treatment of Combined and Municipal Sewage: by A.J. Shuckrow et al., Pacific NW Lab., Battelle Memorial Institute, Richland, WA, February, 1973.
NTIS PB 219 668

DISINFECTION

Conventional municipal sewage disinfection generally involves the use of chlorine gas or sodium hypochlorite as the disinfectant. To be effective for disinfection purposes, a contact time of not less than 15 minutes at peak flow rate and a chlorine residual of 0.2 to 2.0 mg/l are commonly recommended.

Disinfection of CSO is generally practiced at treatment facilities to control the discharge of pathogens, and other microorganisms in receiving waters. However, an approach other than that used for the conventional municipal sewage is required, mainly because such flows have characteristics of intermittent, high flow rate, high SS content, wide temperature variation, and variable bacterial quality.

Several other aspects of disinfection practices require consideration for CSO treatment applications:

- A residual disinfecting capability may not be feasible for CSO (and all wastewater) discharges. Recent work indicates that chlorine residuals and compounds discharged to natural waters may be harmful to aquatic life.
- The coliform count is increased by surface runoff in quantities unrelated to pathogenic organism concentration. Total coliform levels may not be the most useful indication of disinfection requirements and efficiencies.
- Discharge points requiring disinfection are often at outlying points on the sewer system and require unmanned, automated installations.

The disinfectant used at a facility for treatment of CSO should be adaptable to intermittent use. Other considerations include the disinfection effectiveness, and the safety and ease of feeding. Table 36 shows disinfectants that might be used for stormwater disinfection. Chlorine and hypochlorite will react with ammonia to form chloramines and with phenols to form chlorophenols. These are toxic to aquatic life and the latter also produce taste and odor in the water. Chlorine dioxide does not react with ammonia and completely oxidizes phenols. Ozone is also effective in oxidizing phenols.

High-rate disinfection refers to achieving either a given percent or a given bacterial count reduction through the use of (1) decreased disinfectant contact time, (2) increased mixing intensity, (3) increased disinfectant concentration, (4) chemicals having higher oxidizing rates, or (5) various combinations of these. Where contact times are less than 10 minutes, usually in the range of 1 to 5

Table 36. Characteristics of Principal Stormwater Disinfection Agents

Characteristic	Chlorine	Hypochlorite	Chlorine dioxide	Ozone
Stability	Stable	6 month half-life	Unstable	Unstable
Reacts with ammonia to form chloramines	Yes	Yes	No	No
Destroys phenols	At high concentrations	At high concentrations	Yes	Yes
Produces a residual	Yes	Yes	Short lived[a]	No
Affected by pH	More effective at pH <7.5	More effective at pH <7.5	Slightly	No
Hazards	Toxic	Slight	Toxic, explosive	Toxic

a. Chlorine dioxide dissociates rapidly

(EPA-600/8-77-014)

minutes, adequate mixing is a critical parameter, providing complete dispersion of the disinfectant and forcing disinfectant contact with the maximum number of microorganisms. The more physical collisions high-intensity mixing causes, the lower the contact time requirements. Mixing can be accomplished by mechanical flash mixers at the point of disinfectant addition and at intermittent points, or by specially designed plug flow contact chambers containing closely spaced, corregated parallel baffles which create a meandering path for the wastewater (EPA-670/2-73-077).

High-rate disinfection was shown to be enhanced beyond the expected additive effect by sequential addition of Cl_2 followed by $C10_2$ at intervals of 15 to 30 seconds (EPA-670-2-75-021; EPA-600/2-76-244). A minimum effective combination of 8 mg/l of Cl_2 followed by 2 mg/l of $C10_2$ was found as effective in reducing total and fecal coliforms, fecal streptococci, and viruses to acceptable target levels as adding 25 mg/l Cl_2 or 12 mg/l $C10_2$ individually. It was surmised that the presence of free Cl_2 in solution with chlorite ions ($C10_2^-$), (the reduced state of $C10_2$), may cause the oxidation of $C10_2^-$ back to its original state. This process would prolong the existence of $C10_2$, the more potent disinfectant.

Ozone has a more rapid disinfecting rate than chlorine and also has the further advantage of supplying additional oxygen to the wastewater. The increased disinfecting rate for ozone requires shorter contact times, and results in a lower capital cost for a contactor, as compared to that for a chlorine contact tank. Ozone does not produce chlorinated hydrocarbons or a long-lasting residual as chlorine does, but it is unstable and must be generated on-site just prior to application. Thus, unlike chlorine, no storage is required. In tests on CSO in Philadelphia (see "microscreening and disinfection" reports listed at the end of this Disinfection section), equivalent disinfection was obtained using either 3.8 mg/l of ozone or 5 mg/l of chlorine.

Because of the characteristic intermittent operation associated with treatment of CSO, reduction of construction cost with a potential increase in operating costs often results in overall minimum costs. In the case of chlorination facilities, as applied to treatment of CSO, the construction costs associated with contact basins having conventional contact time of 15 to 30 minutes are high and difficult to justify. Therefore, consideration should be given to higher mixing intensities, to make better usage of the chlorine and/or higher chlorine dosages and smaller, shorter detention time contact basins to effect the same end results.

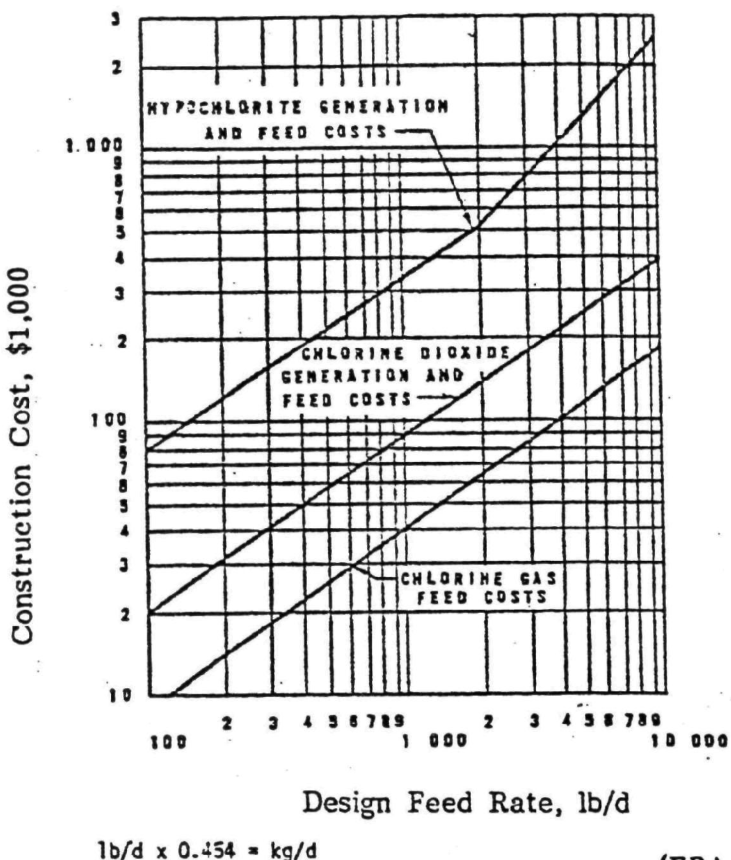
Disinfectant costs for CSO treatment are higher than those for sewage treatment. This is the result of smaller total annual disinfectant volume requirements, increased disinfectant concentration requirements, and higher unit operation and maintenance costs for CSO treatment facilities. These costs could be reduced by using the facilities in conjunction with dry-weather flow treatment plants, whenever possible.

Curves comparing generation and feed costs for chlorine gas, chlorine dioxide, and hypochlorite generation disinfection systems for CSO have been developed and are presented in Figure 17. These costs (ENR = 2000) include manufactured equipment, labor, piping, housing, electrical and instrumentation, and miscellaneous items. No allowance for land was included.

Capital and operating costs for several CSO and stormwater disinfection facilities are presented in Table 37.

As previously mentioned, conventionally long contact times may not be economical. Short term contact times with more intense mixing, using a basin and mixer similar to those used in coagulant mixing, can effect the same disinfection results. Construction costs curves for high intensity mixing/chlorine contact basins are presented in Figure 18. Power requirement curves, for high intensity mixing, are presented in Figure 19.

The capital costs for different disinfection agents and methods resulting from the Philadelphia study are shown in Table 38. The capital costs for ozone generation are usually the highest of the most commonly used processes. Ozone operation costs are very dependent on the cost of electricity and the source of the ozone (air or pure oxygen).



(EPA-600/8-77-014)

Figure 17. Chlorine Disinfection Feed Facilities Cost Curves (ENR = 2000)

Table 37. Cost Data on Chlorine Gas and Hypochlorite Disinfection[a]

Location, agent and source	Capital Cost, $	Operating cost, $/yr	Cost/lb available Chlorine, $
Akron, Ohio[b]			
Sodium hypochlorite			
Purchased	762,000	40,200	0.26-0.46
Cambridge, Massachusetts and Somerville, Massachusetts[b]			
Sodium hypochlorite			
Purchased	--	--	0.67
On-site generation	--	--	0.35
New Orleans, Lousiana[c]			
Sodium hypochlorite			
On-site generation	1,000,000	500,000	0.21
Saginaw, Michigan[d]			
Chlorine gas	280,000	4,000	0.60
Sodium hypochlorite			
Purchased	34,000	11,000-20,000	0.31-.54
On-site generation	165,000-278,000	8,100-9,000	0.48-0.69
South Essex Sewerage District, Massachusetts[e]			
Chlorine gas	1,151,000	402,000	0.06
Sodium hypochlorite			
Purchased	728,000	630,000	0.08
On-site generation			
Sea water	2,900,000	280,000	0.06
Brine	2,900,000	523,000	0.09

a. ENR = 3452
b. Combined sewer overflow disinfection
c. Storm sewer discharge disinfection
d. Combined sewer overflow disinfection at use rate of 42,000 lb/yr of chlorine
e. Sewage treatment plant effluent disinfection at use rate of 24,000 lb/day of chlorine

$/lb x 2.2 = $/kg

(EPA-670/2-74-040)

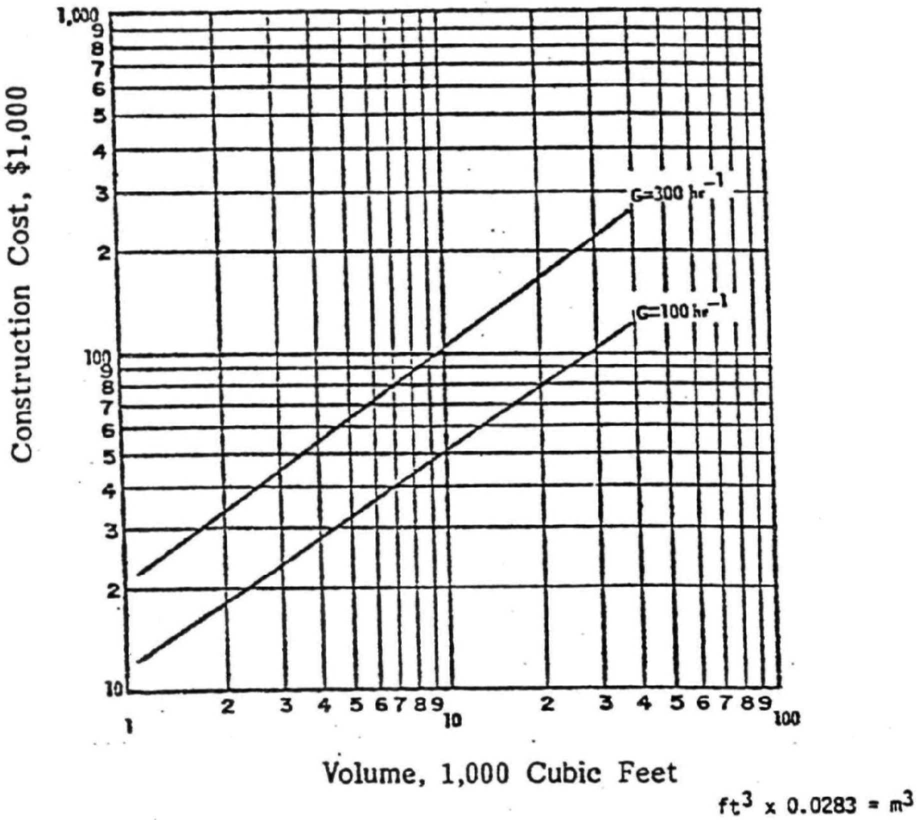


Figure 18. High Intensity Mixing/
Chlorine Contact Basin Cost (ENR = 2000)

(EPA-600/2-76-286)

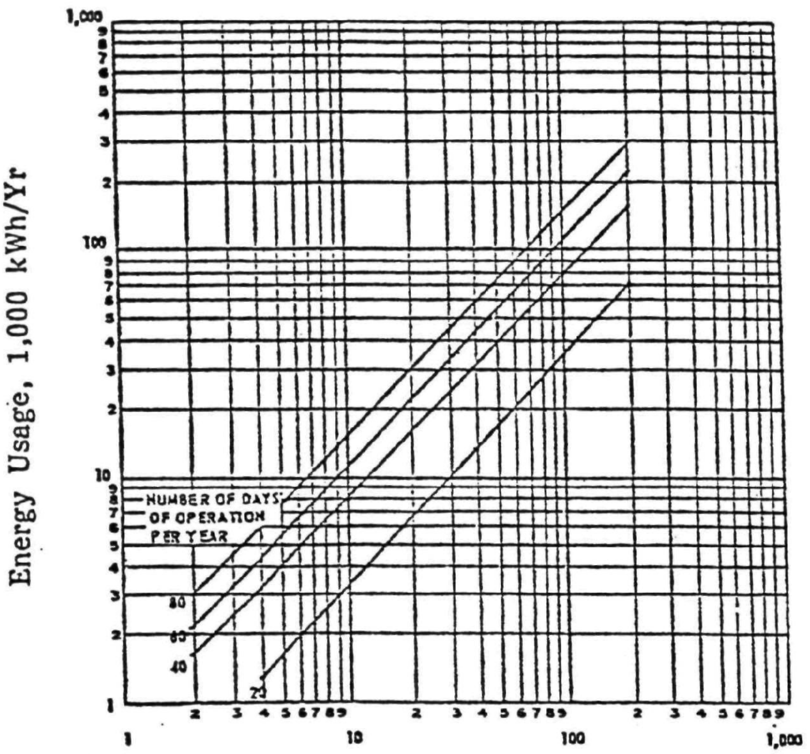


Figure 19. High Intensity Mixing (Chlorine Contact) Power Requirements

(EPA-600/2-76-286)

Table 38. Comparison of Estimated Capital Costs for 3 Different Disinfection Methods[a]

Disinfection method	Capital cost, \$/mgd
2-Minute ozone contact (chamber with once-through oxygen-fed ozone generator)[b]	22,460
2-Minute chlorine contact (chamber with hypochlorite feeder)[c]	2,625
5-10 Minute conventional chlorine contact[d]	2,920

a. ENR = 3452

b. Unit cost of ozone at \$9.00/lb from oxygen @ \$0.33/lb; dosage of 3.8 ppm; Otto plate type generator.

c. Unit cost of hypochlorite at \$0.73/lb available chlorine; dosage of 15 ppm.

d. Unit cost at \$0.73/lb available chlorine; dosage of 5 ppm.

\$/mgd x 0.0228 = \$/l/sec
\$/lb x 2.2 = \$/kg

(EPA-670/2-74-040)

References

EPA-670/2-75-021 - Bench-Scale High-Rate Disinfection of Combined Sewer Overflows with Chlorine and Chlorine Dioxide: by P.E. Moffa et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, April, 1975.
NTIS PB 242 296

11023EVO06/70 - Microstraining and Disinfection of Combined Sewer Overflows: by Cochrane Div., Crane Co., King of Prussia, PA, June, 1970.
NTIS PB 195 67

EPA-R2-73-124 - Microstraining and Disinfection of Combined Sewer Overflows-Phase II: by G.E. Glover, and G.R. Herbert, Crane Company, King of Prussia, PA, January, 1973.
NTIS PB 219 879

EPA-670/2-74-049 - Microstraining and Disinfection of Combined Sewer Overflows-Phase III: By M.B. Maher, Crane Company, King of Prussia, PA, August, 1974.
NTIS PB 235 771

EPA-600/2-79-031b - Combined Sewer Overflow Abatement Program, Rochester, NY - Volume II Pilot Plant Evaluations: by F.J. Drehwing et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, July, 1979.
NTIS PB 80-159262

EPA-600/2-79-134 - Disinfection/Treatment of Combined Sewer Overflows, Syracuse, New York: by F. Drehwing et al., O'Brien & Gere Engineers, Inc., Syracuse, NY, August, 1979.
NTIS PB 80-113459

EPA-670/2-73-077 - Combined Sewer Overflow Seminar Papers November, 1973.
U.S. EPA and N.Y.S. - DEC
NTIS PB 231 836

EPA-600/2-76-244 - Proceedings of Workshop on Microorganisms in Urban Stormwater: by R. Field et al., Edison, NJ, (March 24, 1975), November, 1976.
NTIS PB 263 030