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FOREWORD

By Staff

Highway Research Board

This report is recommended to highway design engineers, maintenance engineers, and others concerned with highway safety. It contains a compilation of recommended practices for locating, designing, and maintaining guardrails and median barriers, as selected from a comprehensive literature review, a state-of-the-art survey, and the advice of a selected group of acknowledged experts. It is believed that this report will contribute to the effort toward producing safer highways.

There is a pressing need on the part of highway design engineers for a choice of effective guardrail and median barrier installations. Although the problem is one currently receiving extensive attention, it is recognized that considerable time will elapse before all work to identify or develop effective systems will be completed. Several sources previously generated much usable information that needed to be consolidated so as to provide an up-to-date, concise instructional manual that can provide *immediate* "how-to-do-it" guidance for the highway design engineer with respect to the various features of the commonly used, tried and proven systems now in existence that should be recognized as interim standards until research has satisfied the ultimate needs in this area.

This report presents the results of synthesizing a great deal of information concerning guardrails and median barriers collected as a part of NCHRP Project 15-1(2), "Guardrail Performance and Design," and provides recommended standards for nationwide consistency of practice by highway design engineers as related to warrants, design, and maintenance.

The agency worked jointly with a special NCHRP advisory group—consisting of John L. Beaton, California Division of Highways; Malcolm D. Graham, New York Department of Transportation; James D. Lacy, BPR; and Paul C. Skeels, General Motors Proving Ground—which exercised its responsibility to advise and counsel as to the contents of this report. Although the entire report content was originated by the agency, each recommendation has the consensus endorsement of the advisory group. Where recommendations are founded on less than clear-cut evidence, the judgment of the advisory group prevailed. It should be recognized that where no consensus of the advisory group was evident, no recommendation is presented.

Inasmuch as this report is intended as a design aid, references and supporting documentation have generally not been cited in order to preserve a clear, straightforward presentation. It should be noted also that the included standard designs certainly will be refined and upgraded in the future and the designer is obligated to periodically obtain the latest revisions.

The method of presentation is mainly graphic, with several drawings and tables. Example problems are included in the appendixes to demonstrate the warranting procedure.

This report covers the first phase of a 30-month research effort under NCHRP Project 15-1(2). Continuing work includes mathematical modeling, physical analog studies, and full-scale crash tests for various guardrail and median barrier systems, including end treatments and transitional zones. The next report is scheduled for publication in early 1970.

Treatability of Recreational Vehicle Wastewater in Septic Systems at Highway Rest Areas

CHARLES A. BROWN, KEVIN E. KIERNAN, JOHN F. FERGUSON, and MARK M. BENJAMIN

ABSTRACT

Recreational vehicle (RV) owners commonly use chemical toilet additives containing formaldehyde to minimize odors from their wastewater holding tanks. The purpose of this study is to determine the character and treatability of this wastewater using conventional septic tank-drainfield systems at highway rest areas. RV wastewater is a high-strength waste. Mean concentrations from 72 samples are 5-day biochemical oxygen demand (BOD_5) 3110 mg per liter, chemical oxygen demand (COD) 8230 mg per liter, total suspended solids (TSS) 3120 mg per liter, and volatile suspended solids (VSS) 2640 mg per liter, with a formaldehyde concentration of 170 mg per liter. The average volume per vehicle is 62 liters. Because RV wastewater is highly concentrated, sludge and scum accumulation and pumpout interval should be considered in addition to hydraulic residence time when sizing septic tanks for RV waste. A model for sludge and scum accumulation is developed based on the concept that some organic material in sludge and scum is readily degradable and compactible, some is degradable and compactible with extended residence time, and some material is inert and not compactible.

Recreational vehicles (RVs) including campers, trailers, motor homes, and fifth wheelers have become popular as a means of transportation and shelter for people on vacations and weekend trips. During the summer, about 16 percent of the traffic using Interstate highway rest areas in Washington is composed of RVs.

Many RVs have built-in toilets and holding tanks. It is common practice to empty the holding tank after a few days on a long trip or at the end of a short trip. RV holding tank disposal stations are provided at some private and public campgrounds, some service stations, and, in some states, at selected highway rest areas.

Many people use additives in their holding tanks to minimize odors and to prevent clogging of their drain lines. Common commercial additives for RV holding tanks contain formaldehyde or pH buffers or enzymes. Formaldehyde inhibits biological degradation, thereby preventing the formation of odorous compounds. pH buffers prevent odors by maintaining the solution pH in a range where most odorous volatile compounds dissociate into ionic, nonvolatile species. Enzymes are used to increase the rate of biological degradation in order to liquefy solids and prevent clogging. Other ingredients in commercial RV additives include surfactants, dyes, and perfumes. Some people add other chemicals, usually soaps and surfactants, to their holding tanks instead of commercial preparations.

Common sewage treatment systems for RV wastewater include septic tank-drainfield systems, sewage lagoons, and activated sludge treatment plants. A few sites have holding tanks, and the waste is transported elsewhere for treatment. Some operators of RV disposal stations report that the chemicals in the additives upset their system. Others report that this high-strength waste overloads their system (1).

PROCEDURE

Wastewater from 72 recreational vehicles was collected and sampled at RV dump stations in western Washington to determine average values for volume, composition of waste, and formaldehyde concentration. Fifty-three vehicles were sampled at the Sea-Tac Rest Area on Northbound Interstate 5 near Tacoma, Washington. Fourteen vehicles were sampled at the Silver Lake Rest Area on Southbound Interstate 5 near Everett, Washington. Five vehicles were sampled at the Thousand Trails Campground near LaConner, Washington.

The RV owners usually discharge their holding tanks through a 10.2-cm diameter flexible plastic hose that is connected to the holding tank outlet. To collect waste as it was being dumped, a second hose was coupled to the owner's hose and connected to a heavy-duty, kitchen-style garbage disposal. The outlet of the disposal was connected with tygon tubing to a 19-liter-per-minute positive displacement, Vanton Flexiliner pump. The pump discharged into a 210-liter barrel.

All black (toilet waste) and gray (washwater) water that the owner wished to dump, as well as any water that the owner used to rinse the holding tank and hose, was collected in the barrel. Thus, the sample had about the same composition as the water that the owner would typically discharge at an RV dump station. The volume of total wastewater and rinse water was measured, and a sample was put on ice and brought back to the laboratory. In the laboratory, a volume-proportional composite sample was created from between 1 and 6 individual samples.

Septic tank water samples were collected from the RV disposal septic tank systems at Wenberg State Park in Snohomish County and Dash Point State Park in King County. Drainfield water samples at Wenberg were obtained through a lysimeter plate, which was buried in the drainfield soil about 30 cm horizontally away from, and about 15 cm below the bottom of, a gravel-filled trench. At Dash Point, a hole about 90 cm deep was dug about 30 cm away from a gravel-filled drainfield trench. Septic tank water was allowed to seep out of the saturated soil and collect in the hole.

Water samples were put on ice and brought to the laboratory where they were stored at 5°C until analyzed. They were analyzed for total and volatile suspended solids, total and soluble chemical oxygen demand, and total 5-day biochemical oxygen demand using Standard Methods (2). Soluble COD samples were obtained by filtering the wastewater through 0.45 micron membrane filters. Samples filtered through 0.45 micron filters also were analyzed for formaldehyde using the chromotropic acid method (3). Sludge and scum samples were taken from the Wenberg septic tank and analyzed for total and volatile solids concentration.

To determine potential toxic effects of formaldehyde on anaerobic bacterial cultures, anaerobic toxicity assays (ATAs) were conducted (4). A mesophilic anaerobic culture was maintained in an incubator at 34°C. The culture was daily fed 660 mg per liter of COD (acetate and propionate) and 8 mg per liter of formaldehyde. The ATA was conducted in 250-ml serum bottles. Forty-eight ml of anaerobic culture and nutrient media and a dose of formaldehyde were put into a serum bottle and spiked with 2.0 ml of organic feed consisting of 75.0 mg acetate and 26.5 mg propionate. The bottles were sealed with serum caps and placed in an incubator at 34°C. Gas production was measured periodically using glass syringes with 20-gauge needles. Average cumulative gas production for several replicates was plotted, and toxicity was indicated if test bottles had significantly less gas production than controls, which contained all the same ingredients but no formaldehyde.

RESULTS

A summary of the analytical results for RV wastewater characterization is given in Table 1. Analytical results for septic tank water samples are given in Table 2 and results from the drainfield water samples are given in Table 3.

TABLE 1 Average RV Wastewater Characteristics

Number of Samples	72
Volume, liters	62 ± 10[a]
standard deviation[b]	43
Total Suspended Solids, mg l^{-1}	3120 ± 490
standard deviation	2120
Volatile Suspended Solids, mg l^{-1}	2460 ± 410
standard deviation	1780
Total COD, mg l^{-1}	8230 ± 1430
standard deviation	6140
Soluble COD, mg l^{-1}	2930 ± 560
standard deviation	2350
Total BOD_5, mg l^{-1}	3110 ± 530
standard deviation	2200
Formaldehyde mg l^{-1}	
All RV Users	170 ± 60
standard deviation	250
Formaldehyde Additive Users Only	250 ± 60
standard deviation	180

Note: mg l^{-1} = milligrams per liter.

[a]Ranges given are the error of the mean value at a 95% confidence level.
[b]Standard deviation for individual RV samples.

Wenberg septic tank sludge total solids concentration was 8.5 percent and volatile solids concentration was 5.3 percent; scum total solids concentration was 19.1 percent and volatile solids concentration was 13.1 percent. Results for the anaerobic toxicity assays for formaldehyde-dosed cultures are shown in Figure 1.

TABLE 2 Septic Tank Water Analytical Results–Wenberg State Park Septic Tank

11- 9-80	Compartment #1	Compartment #2	Compartment #3
Scum, cm	46	0	--
Sludge, cm	30	30	--
Total COD, mg l^{-1}	1620	--	--
3-10-81			
Scum, cm	38	0	--
Sludge, cm	20 to 36	15	--
Total COD, mg l^{-1}	5360	2500	--
Soluble COD, mg l^{-1}	3290	1850	--
TSS, mg l^{-1}	700	80	--
VSS, mg l^{-1}	550	70	--
Temperature, °C	12	12	--
pH	6.9	7.05	--
Formaldehyde, mg l^{-1}	5	5	--
8-20-81			
Scum, cm	58	0	0
Sludge, cm	30	25	18
Total COD, mg l^{-1}	3180	2870	2870
Soluble COD, mg l^{-1}	1900	1980	1820
BOD_5, mg l^{-1}	1780	1490	1430
TSS, mg l^{-1}	460	170	170
VSS, mg l^{-1}	410	140	150
Formaldehyde, mg l^{-1}	5.5	6.8	8.7
9- 9-81	Dash Point State Park Distribution Box		
Total COD, mg l^{-1}		2310	
BOD_5, mg l^{-1}		1360	
TSS, mg l^{-1}		300	
VSS, mg l^{-1}		240	
Formaldehyde, mg l^{-1}		9.2	

Note: mg l^{-1} = milligrams per liter.

[a]Septic tank had three compartments in series with volumes of 3780, 2530, and 1250 liters, respectively.

TABLE 3 Drainfield Water Analytical Results

	Dash Point 9-9-81	Wenberg 9-14-81
Total COD, mg per liter	1,880.0	1,240.0
Soluble COD, mg per liter	—	870.0
BOD_5, mg per liter	910.0	460.0
Formaldehyde, mg per liter	6.0	4.8

DISCUSSION

Wastewater Characteristics

The data in Table 1 indicate that RV wastewater is a very high-strength waste with a BOD_5 of 3110 mg per liter and a TSS of 3120 mg per liter. Variability in waste strength among vehicles is high as evidenced by the large standard deviations. These results are generally consistent with other studies of recreational wastewaters (5-7) as indicated in Table 4.

Waste strengths and volumes for typical domestic wastewater and for highway rest area restroom wastewater measured by several investigators are given in Table 5. These values permit comparison with the high strength RV waste characteristics and are important when considering combining RV dump station waste with rest area or domestic waste in treatment

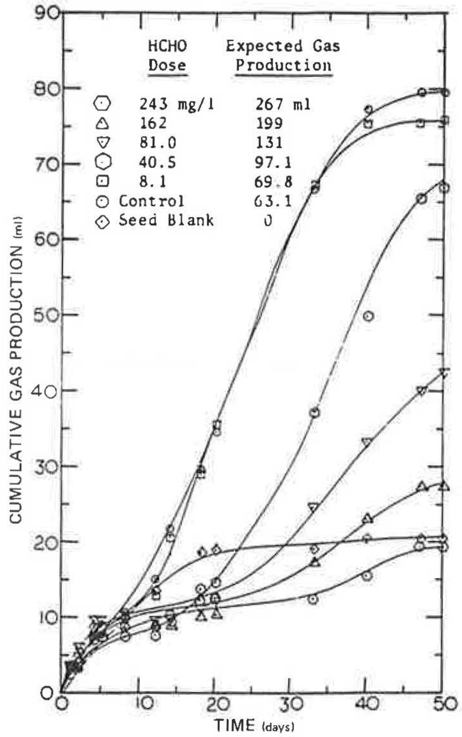

FIGURE 1 Response of anaerobic toxicity assay cultures to formaldehyde.

systems. Rest area waste strength is typical of weak-to-average domestic waste.

Formaldehyde preparations are by far the most popular additives in use today with 67 percent of the RV owners using them. Average formaldehyde concentration for wastewater from formaldehyde users was 250 mg per liter.

A significant portion of RV users were not using any additive--usually people on a short weekend trip. Phenol-based products were not found in either this survey or on the shelves of some Washington retail RV accessories stores. Only one person was found using a zinc-based additive, which is no longer on the market. In the early and mid-1970s, zinc was the most common active ingredient in additives (5-7). In 1978 California prohibited the sale or use of zinc and other nonbiodegradable additives. In response, manufacturers switched to other active ingredients, usually formaldehyde-based. The manufacture and sale of zinc products has apparently disappeared completely from the RV additive market.

Disposal Station Usage Rate

To estimate usage rates of RV disposal stations in Washington, short-term traffic counts were made by people stationed at sites throughout the state on various weekdays and weekends during the summer of 1981 and on Labor Day, 1981. On Labor Day, 68 RVs used the two disposal stations at the Sea-Tac Rest Area between 3:00 and 6:00 p.m. Generally, the flow of RVs through the stations was heavy and steady, and a short line of RVs had formed. Thus, the maximum usage rate for a station is estimated to be 11.3 RVs per hour.

The following scenario of a busy day gives the expected maximum wastewater generation rate for a disposal station and may be used for design purposes. Although lights are sometimes provided, few people use the disposal station at night. Assume that people begin using the station regularly at 8:00 a.m. on a holiday morning and the usage rate is one-half the maximum rate until about noon. From noon until 5:00 p.m., assume that usage is at the maximum of 11.3 RVs per hour. Finally, assume that evening use between 5:00 and 9:00 p.m. tapers off to one-half the maximum rate again. This gives a realistic maximum usage rate for a very busy day of about 100 RVs per day and corresponds to a wastewater volume of 6200 liters per day.

RV Septic Tank Effluent Characteristics

The data in Table 1 indicate that effluent from an RV wastewater septic tank is very strong in total and soluble COD and BOD_5 and has high total and volatile suspended solids concentrations.

TABLE 4 Literature Review of Recreational Wastewater Characterization

Reference	(5)			(6)	(7)	Present Study
Wastewater Type	RV Black	RV Gray	RV Combined	RV	Powerboats, Sailboats, and Houseboats	RV including rinse
	(excluding rinse water)					
Number of Samples	140	140	140	14	43	72
Volume, l per vehicle	38	38	38	--	--	62±10[a]
TSS, $mg\ l^{-1}$	4200	550	3850	1120-20500	2430±980[a]	3120±490
VSS, $mg\ l^{-1}$	3743	481	3329	1020-18400	1910±800	2640±410
COD, $mg\ l^{-1}$	11684	2390	6209	5600-22000	6140±1780	8230±1430
BOD_5, $mg\ l^{-1}$	11700	1870	3080	1838-7590	2560±900	3110±530
Formaldehyde, $mg\ l^{-1}$	276	16	18	_b	_b	170±60
Zinc, $mg\ l^{-1}$	8	0.5	9	1.7-4.6	150±100	_b
Phenol, $mg\ l^{-1}$	1.4	0.13	0.5	_b	_b	_b

Note: $mg\ l^{-1}$ = milligrams per liter.

[a]Ranges given are the error of the mean value at a 95% confidence level.

[b]No analyses made for these components.

TABLE 5 Rest Area and Typical Domestic Wastewater Characterization

Reference	(8)	(9)	(10)	(11)	(12)	(13)
Wastewater Type	Rest Area	Rest Area	Rest Area	Rest Area	Domestic (Medium Strength)	Domestic
Volume						
liters per person-day	19	--	--	13	280	380
liters per vehicle	--	--	21	--	--	--
TSS, mg l^{-1}	56-230	165	124 to 224	--	220	180-300
VSS, mg l^{-1}	--	--	--	--	165	140-230
COD, mg l^{-1}	--	405	203 to 383	--	500	550-700
BOD_5, mg l^{-1}	110-204	165	78 to 210	--	220	160-280
Nitrogen, mg l^{-1} N	--	140	--	--	40	40-50
Phosphorous, mg l^{-1} P	--	29	--	--	8	10-15

Note: mg l^{-1} = milligrams per liter.

For comparison, domestic wastewater septic tank effluent characteristics (14) are given in Table 6. These data were derived from a survey of four conventional septic tank systems servicing individual residences in Snohomish and Pierce counties in Washington.

Formaldehyde levels in both RV septic tank water and drainfield water were found to be about 5 to 10 mg per liter. If there was no mechanism for formaldehyde removal in the tank, a concentration of 170 mg per liter would be expected, which is the average concentration found in RV holding-tank water. The anaerobic toxicity results show substantial reduction in biological activity at 50 to 150 mg per liter formaldehyde and no significant reduction in activity at levels of 5 to 10 mg per liter. If there was biological degradation of formaldehyde, degradation would be expected to continue until formaldehyde concentrations were reduced below 5 to 10 mg per liter. Formaldehyde is probably removed from septic tank systems by nonbiological mechanisms as well as by biodegradation. It appears that, for reasons not well understood at this time, formaldehyde removal ceases in anaerobic systems when formaldehyde concentration drops to about 5 mg per liter.

A sample of sludge from the Wenberg septic tank was placed in a glass flask and small gas bubbles were observed rising from the sludge, confirming the presence of biological activity. Thus, at the formaldehyde levels in RV septic tank water, biological activity is not totally eliminated, though it may be inhibited.

Septic Tank Design Practices

The primary function of a septic tank is to provide removal of suspended solids by settling or flotation. Other important functions include biological decomposition of solids and storage of sludge and scum.

Several design manuals are available for guidance in septic tank design. In these manuals, septic tanks are sized to provide adequate detention time for solids removal based on experience.

The Washington Highway Hydraulic Manual (1972) (23) simply requires a 24-hr minimum detention time:

$$V = Q \quad (1)$$

where V is septic tank volume in liters, and O is design flow rate in liters per day.

The following equation (15) is given for septic tank design at highway rest areas:

$$V = 4{,}250 + 0.75\ Q \quad (2)$$

where V is septic tank volume in liters, with a 5700-liter minimum; and Q is design flow rate in liters per day. They state that the design flow rate should be 1.25 times the average daily rate.

Nomographs were developed for septic tank sizing (7,15) that specify a 36-hr minimum detention time:

$$V = 1.5\ Q \quad (3)$$

Additional design constraints include a minimum volume of 5700 liters.

A 24-hr liquid detention time is required at maximum sludge path and scum accumulation (16). For flows between 2800 and 5700 liters per day, the tank may be sized for a 36-hr detention time as in Equation 3. This allows 33 percent of the tank volume to be used for sludge and scum storage. For flows between 5700 and 57 000 liters per day, Equation 2 may be used. The Washington State Department of

TABLE 6 Typical Effluent from Domestic Wastewater Septic Tanks (8)

	System					Standard Deviation
	Number 1	Number 2	Number 3	Number 4	Average	
COD, mg per liter	189	251	486	265	300	130
BOD_5, mg	112	123	241	123	150	60
TSS, mg	26	27	70	23	37	23
VSS, mg	15	19	55	17	27	19

Transportation (WSDOT) no longer uses Equation 1 for design. Instead, criteria from the Washington State Department of Social and Health Services, which are similar to those suggested by Otis et al. (16), are used (17).

Figure 2 shows septic tank size as a function of design flow rate for each of these design correlations. Figure 3 presents these correlations showing detention time as a function of daily flow.

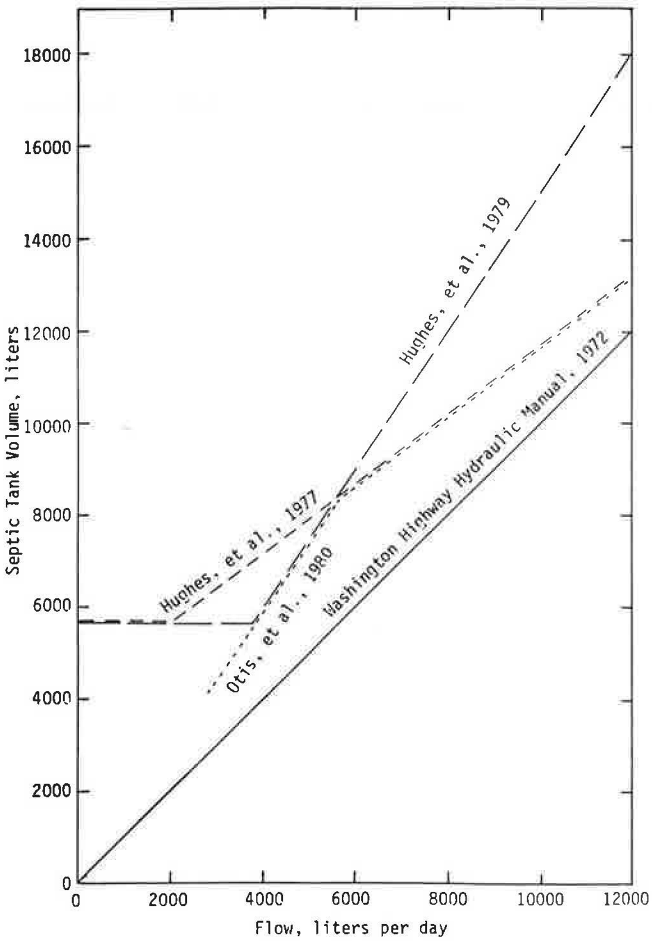

FIGURE 2 Septic tank design equations.

Each of these septic tank sizing equations is based on providing hydraulic detention time for settling of solids. None addresses sludge and scum accumulation or designed service intervals between pumpout. Common practice is to pump domestic waste septic tanks every 3 to 5 years without measuring sludge or scum accumulation (12,16).

The tank should be pumped no later than when the bottom of the scum layer is within 7.5 cm of the outlet or when the sludge level is within 20 cm of the outlet (16). This recommendation does not appear consistent with the septic tank sizing Equation 3 and a minimum 24-hr hydraulic detention time. For a typical 1.0-meter-deep tank, this recommendation allows the tank to be three-quarters full of sludge and scum. However, Equation 3 coupled with a minimum 24-hr detention time provides for only one-third of the tank volume to be filled with sludge and scum.

Because RV wastewater contains very high concentrations of suspended solids as well as formaldehyde (which may inhibit anaerobic digestion of sludge and scum) solids accumulation in RV waste septic tanks will be substantially greater than in domestic waste septic tanks treating an equal volume of wastewater. Therefore, sludge and scum accumulation and pumpout interval should be considered in addition to hydraulic residence time when sizing septic tanks for RV waste.

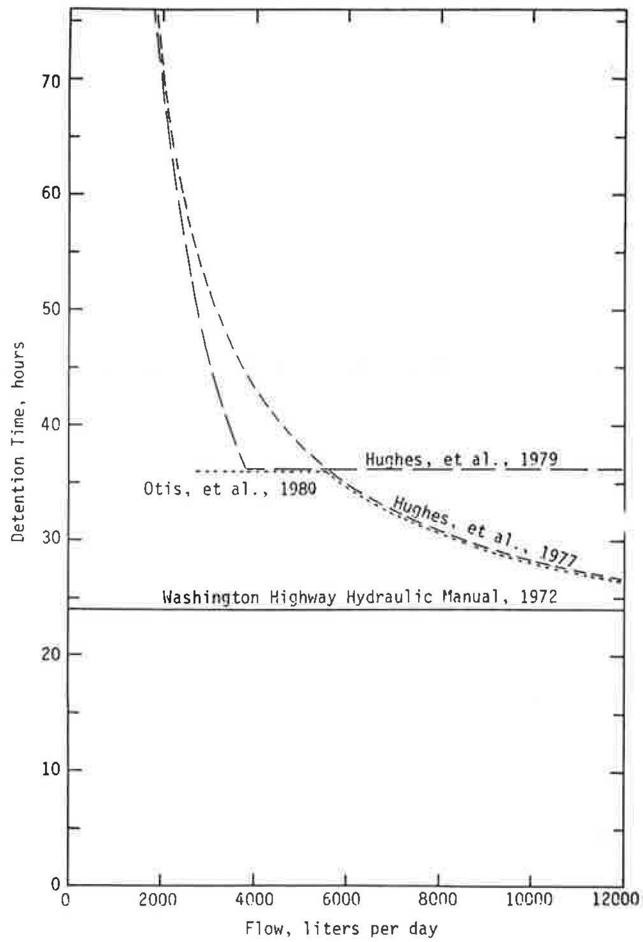

FIGURE 3 Septic tank detention time.

Sludge and Scum Accumulation in Domestic Septic Tanks

As sludge and scum accumulate in a septic tank, the effective liquid volume and detention time decrease. With large accumulations, sludge scouring increases, treatment efficiency decreases, and suspended solids pass through the tank. One cause of clogged drainfields is failure to pump out the septic tank.

Sludge and scum quantities in 300 operating domestic septic tanks were measured. This yielded mean values for accumulation volumes for a number of septic tanks with a specified service life since the last pumpout (18). These data are shown in Figure 4.

A simple first order kinetic model for sludge and scum degradation in septic tanks can be developed, assuming that the sludge removal rate is proportional to the amount of sludge in the tank. Such a model does not work well for extended residence times because no provision is made for refractory materials. In this study, an accumulation model was developed based on the concept that some organic material in sludge and scum is readily degradable and compactible, some is degradable and compactible with extended residence times, and some material is inert and not compactible.

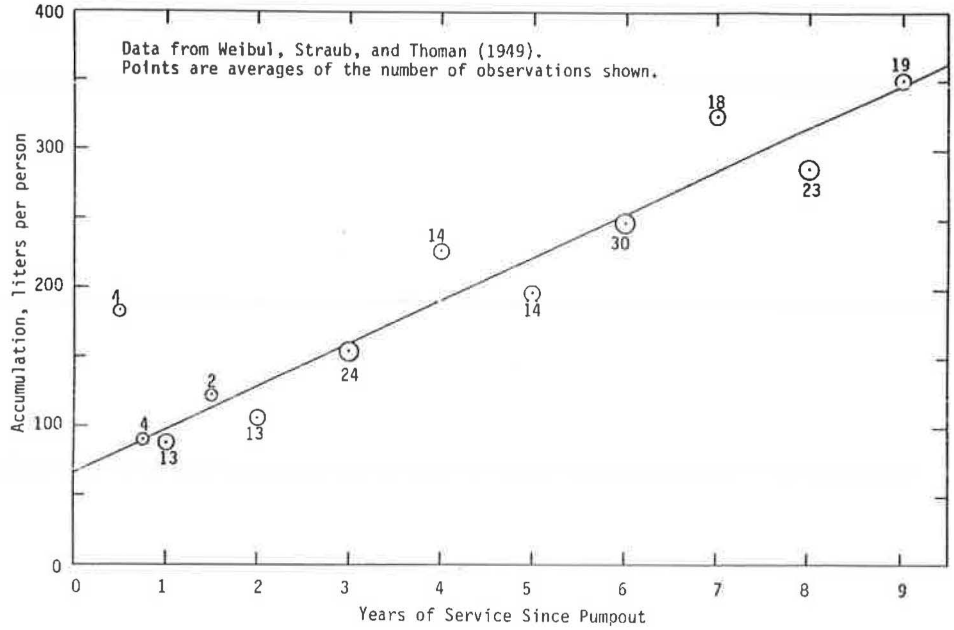

FIGURE 4 Domestic septic tank sludge and scum accumulation.

By material balance, the volume of sludge and scum in a septic tank is the difference between the volume input and the volume removed by degradation and compaction. Removal of sludge and scum with outflow is neglected as shown in the following equation:

$$\text{Volume accumulated} = \text{Volume input} - \text{Volume removed by degradation and compaction} \quad (4)$$

The volume of the input during a time period t is given by

$$V_i = r_i \, \Delta t \quad (5)$$

where

V_i = volume of sludge and scum input;
r = volumetric rate of sludge and scum input; and
Δt = duration of input.

For a given incremental volume, V_i, of sludge and scum entering the tank during one day, the initial rate of degradation will be relatively fast. At short residence times, it is assumed that the first order rate model applies, so the volume of this increment that disappears is proportional to both the volume of the original increment and the residence time:

$$V_{r,\ \text{short}\ t_R} = a \, t_R \, V_i \quad (6)$$

where

V_r = volume removed from the incremental input volume, V_i, by degradation and compaction;
t_R = residence time of the incremental volume; and
a = constant.

For long residence times, the volume that has disappeared from the original increment will be proportional only to the original volume of the increment:

$$V_{r,\ \text{long}\ t_R} = (a/b) \, V_i \quad (7)$$

where a and b are constants. The volume removed after long residence times will be the fraction of sludge and scum that is ultimately degraded or compacted. The inert, noncompactible fraction will remain accumulated in the tank.

The dependence of volume removed from an incremental input volume on residence time can be modeled using Equation 8:

$$V_r = a \, t_R/(1 + b \, t_R) \, V_i \quad (8)$$

Note that at short residence times ($bt_R \ll 1$), Equation 8 reduces to Equation 6; at long residence times, Equation 8 reduces to Equation 7. Thus, Equation 8 is consistent with the limiting cases that comprise the conceptual model.

The total volume removed from the tank from time 0 to time t, designated $V_r(t)$, will be the sum of the volumes removed from each incremental input volume:

$$V_r(t) = \Sigma \, V_r = \sum_{i=1}^{n} [a \, t_{R,n}/(1 + b \, t_{R,n})] V_i \quad (9)$$

where $t_{R,n}$ is residence time of the nth incremental volume. Substituting Equation 5 gives:

$$V_r(t) = \Sigma \, [a \, t_{R,n}/(1 + b \, t_{R,n})] \, r_i \, \Delta t \quad (10)$$

Using differential input times, Equation 10 becomes:

$$V_r(t) = \int_0^t [a \, t/(1 + b \, t)] \, r_i \, dt \quad (11)$$

Integrating gives:

$$V_r(t) = [a \, r_i \, t/b] - [a \, r_i/b^2] \ln(1 + b \, t) \quad (12)$$

The difference between the input volume given by Equation 5 and the volume removed given by Equation 12 gives the volume of the accumulation:

$$V(t) = r_i \, t - [a \, r_i \, t/b] + [a \, r_i/b^2] \ln(1 + b \, t) \quad (13)$$

The value for the sludge and scum input rate, r_i, in Equation 13 can be determined using data for domestic wastewater. Typical wastewater parameters for residences using on-site sewage treatment systems are 166 liters per person per day and 200 to 290 mg per liter TSS (16). This results in a TSS loading of 34 to 49 g per person per day. Values of 166 liters per person per day and 220 mg per liter of TSS will be used as typical septic tank input parameters, giving a TSS loading of 37 g per person per day.

About two-thirds of the solids accumulation in the tanks was sludge and one-third of the volume was scum (18). Measured values for solids concentration of septic tank sludge and scum in this study were 8.5 percent and 19 percent, respectively. Therefore, each liter of total accumulation contains approximately 57 g solids in 0.67 liters of sludge and 63 g solids in 0.33 liters of scum. Assuming a density of 1 g per cubic centimeter for the solids, there are 120 g solids per liter of combined sludge and scum accumulation.

Based on these values, the input rate of solids into a septic tank is 13 400 g per person per year. Because the solids concentration of sludge and scum in the tank is 120 g solids per liter, the sludge and scum input rate, r_i, is 111 liters per person per year.

Weibul's sludge and scum accumulation data, shown in Figure 4, can be used to estimate values for the constants a and b in Equation 13. Values of a and b were chosen by trial and error to give the minimum sum of the squares of the difference between each data point and calculated accumulation from the model. This resulted in values of 1.9 per year and 2.5 per year for the constants a and b, respectively. With these constants, Equation 13 becomes:

$$V(t) = 26\ t + 34\ \ln\ (1 + 2.5\ t) \tag{14}$$

where V(t) is accumulation at time t, liters; and t is service time since last pumpout, years. This model is plotted with Weibul's data in Figure 5.

The data indicate that after a couple of years, the accumulation rate is practically constant with time. This indicates that accumulation of removable solids after a year or two is a small term in the mass balance compared to the accumulation of nonremovable solids.

The ratio of the constants a to b is 0.76. From Equation 7, this implies that three-quarters of the input sludge and scum volume will be ultimately removed by degradation and compaction. It is concluded that Equation 14 provides a reasonable model for domestic septic tank sludge and scum accumulation.

Declining Rate Model Applied to RV Waste

The declining rate model for septic tank accumulation can be applied to RV wastewater by adjusting the constants a, b, and r_i. Table 1 gives the suspended solids concentration of RV wastewater as 3120 mg per liter and the volume per vehicle as 62 liters. Thus, the suspended solids loading per vehicle is 190 grams. Assuming 120 grams of solids per liter of sludge plus scum, this results in 1.6 liters of sludge and scum per RV. On the basis of a unit loading of one RV tank per day, this gives an input rate of 590 liters of sludge and scum per year. It is assumed that the fraction of RV sludge and scum ultimately removable is the same as domestic waste, so the ratio of a to b is still 0.76.

The initial rate of biodegradation is proportional to the constant a. If formaldehyde in RV wastewater inhibits the rate, but not the ultimate extent, of anaerobic digestion of the solids, the values of a and b decrease proportionately (5,19). However, the magnitude of any initial inhibition is unknown. The effect that various degrees of inhibition of the initial degradation rate would have on sludge and scum accumulation is shown in Figure 6. Fifty percent inhibition means that the value of a for RV waste is one-half of the value of a for domestic waste.

The model for sludge and scum accumulation using these constants is given by Equation 15:

$$V_{t,RV} = 140\ t + 448/b\ [\ln(1 + b\ t)] \tag{15}$$

where

$V_{t,RV}$ = accumulation, liters;
t = time since last pumpout, years; and
b = 2.47 x (1 - % inhibition/100).

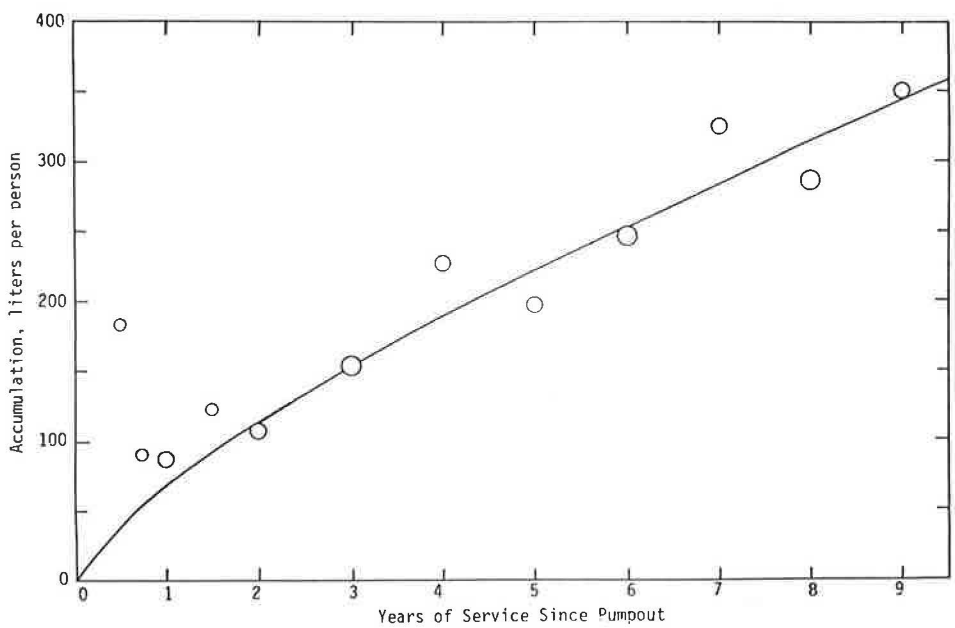

FIGURE 5 Comparison of declining degradation rate model.

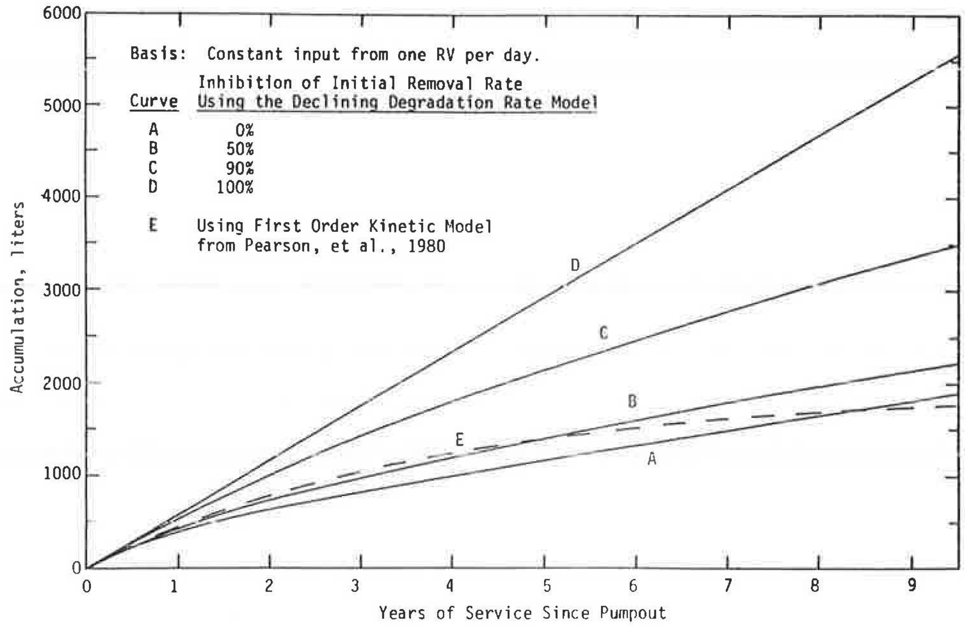

FIGURE 6 RV wastewater septic tank sludge and scum accumulation.

Figure 6 shows the resulting sludge and scum accumulation in an RV waste septic tank based on this model. The curves are based on one RV input per day. To adjust to any other basis, the accumulation is multiplied by the desired daily RV input rate. For comparison, Figure 6 shows sludge and scum accumulation based on a first order kinetic model.

Although the anaerobic toxicity assays were not really designed to give kinetic information, the gas production rates during the growth phases may be used to obtain rough estimates of inhibitory effects on degradation rate. For formaldehyde concentrations of 0, 40, 80, 160, and 240 mg per liter, gas production rates during the growth phases were 2.5, 2.0, 1.1, 0.7, and 0.5 ml per day, respectively. This corresponds to 0, 20, 57, 74, and 80 percent inhibition in gas production rate for the respective formaldehyde concentrations. Therefore, an assumption of 90 percent reduction in the initial removal rate in RV waste septic tanks would be a conservative estimate for design purposes. Recall that although formaldehyde concentration in RV tanks was 170 mg per liter, it was quickly reduced by physical, chemical, or biological reactions to much lower levels in bench scale and in operating septic tanks.

Recommendation for Sizing RV Waste Septic Tanks

Septic tanks for RV wastewater should be sized with consideration for both hydraulic detention time and solids accumulation. Because RV waste is very concentrated, there will be much more sludge and scum accumulated for a given quantity of water than in domestic tanks. The relationships given in engineering manuals are based only on hydraulic detention time and do not address accumulation or pumpout interval.

The hydraulic detention time should be 24 hr at the maximum sludge and scum accumulation (16). This detention time should be for the maximum daily flow rate. Thus, a septic tank for RV wastewater can be sized by adding the volume required for a minimum 24-hr detention time to the volume required for sludge and scum at the designed service period before pumpout. The resulting septic tank sizing equation using 90 percent reduction of the initial degradation rate is given by adding Equations 1 and 15:

$$V = Q_{max} + n/365\ [140\ t + 1{,}800\ \ln\ (1 + 0.25\ t)] \quad (16)$$

where

V = septic tank size, liters;
Q_{max} = designed peak flow rate for system, liters per day;
n = designed average number of RVs per year; and
t = designed service interval between pumpout, years.

This relationship is plotted in Figure 7 for average use rates of 1,000, 5,000, and 10,000 RVs per year and a maximum daily wastewater flow rate of 6200 liters per day.

Figure 7 demonstrates the importance of considering sludge and scum accumulation when sizing septic tanks. At 1,000 vehicles per year, the hydraulic flow rate term in Equation 16 dominates. However, at 5,000 RVs per year, the accumulation term becomes increasingly important for more than 1 year of service time, and at 10,000 RVs per year, the accumulation term dominates Equation 16 after 1 year of service time.

Drainfield Design

Where soil conditions are suitable, subsurface soil absorption is a simple, effective method of treating septic tank effluent. Partially treated wastewater is discharged below the ground surface where it is absorbed and treated by soil as it percolates to the groundwater.

Several different designs of subsurface soil absorption systems may be used including trenches, beds, seepage pits, mounds, fills, and artificially drained systems. All of these systems are covered excavations filled with porous media with a means for introducing and distributing the wastewater throughout the system. The following discussion

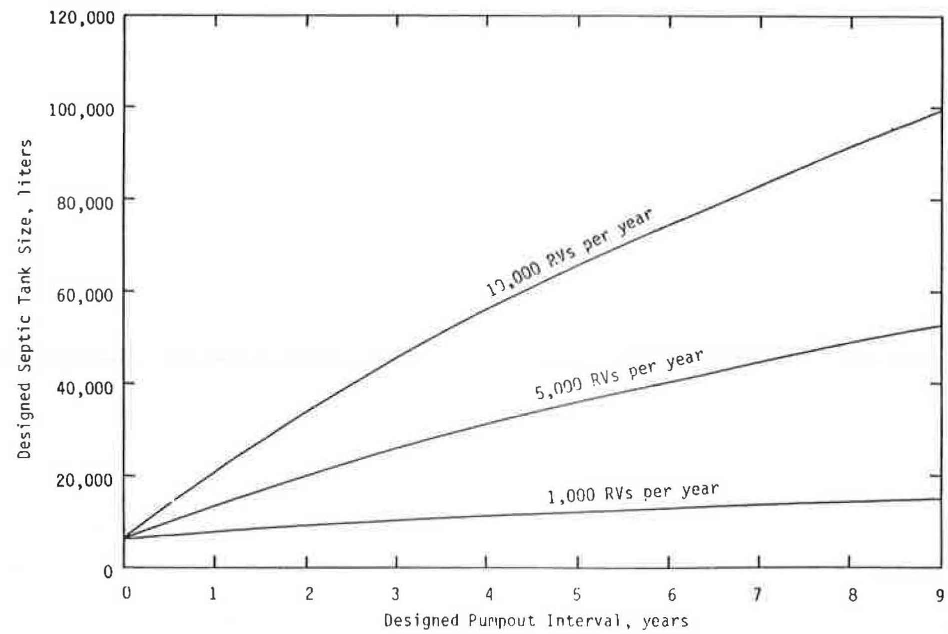

FIGURE 7 Septic tank volume for RV disposal stations.

concentrates on the trench drainfield system, because it is the most commonly used soil absorption system (16).

Continuous application of wastewater causes a clogging mat to form at the soil infiltrative surface. This mat slows the movement of water into the soil. This can be beneficial because it helps to maintain unsaturated soil conditions below the mat. Fortunately, the mat seldom seals the soil completely. The size of a drainfield must be based on the infiltration rate through the clogging mat that ultimately forms. Formation of the clogging mat depends primarily on loading pattern and soil conditions, although other factors may be important (16).

The clogging mat, when viewed under a microscope, looks like a mass of sewage solids consisting of bacteria, protozoa, cellulose pieces, nematodes, and bacterial slime. It is a living layer that responds to temperature change, food load, oxygen availability, and other environmental factors. Between dosings, the mat gradually dries, cracks, and shrinks in volume. The permeability of the mat varies from time to time and place to place within the trench.

The clogging process is related to the rate of biological growth and therefore to the food and solids load. It might be assumed that a linear relationship exists between increased BOD_5 and solids concentrations and increased clogging. However, studies have demonstrated only small differences in clogging rate over a range of wastewater qualities.

The following relationship adjusts required drainfield area to loading (20,21):

$$\text{Adjusted area required} = \text{Area required for standard septic tank pretreatment} \times (BOD_5 + TSS/250)^{1/3} \quad (17)$$

where BOD_5 and TSS are expressed in mg per liter, and 250 mg per liter is the sum of BOD_5 plus TSS for standard septic tank effluent (Table 6 gives this sum as 187 mg per liter for typical septic tank effluent). This relationship is valid only for domestic sewage and does not apply to soils with low permeability. The wastewater carrying capacity of soils with low permeability may be governed by the hydraulic or flow capacity of the soil rather than the clogging mat.

Sizing Drainfields for Servicing RVs

RV septic tank effluent is very strong in COD and BOD, has high suspended solids concentrations, and contains 5 to 10 mg per liter formaldehyde. Because of the high strength of this effluent, it is possible that a drainfield size based on standard application rates will fail prematurely. Some sizing factor should be applied to drainfields receiving this high strength effluent.

A linear relationship for increasing drainfield area with increasing wastewater strength would provide a constant nutrient loading per square meter of drainfield, but this approach is too restrictive. For RV septic tank effluent, which has a total BOD_5 and TSS concentration 8.6 times stronger than typical domestic septic tank effluent, a linear relationship would require a sizing factor of 8.6. Although such a sizing factor would provide the same mass of nutrients per square meter of drainfield clogging mat, and hence a similar clogging mat density as found in domestic system drainfields, the hydraulic flowrate per square meter for an RV system would only be 12 percent of the flowrate that could be transmitted through such a clogging mat. Also, the work of Laak (20) and of Daniel and Bouma (21) does not support a linear relationship between required area for prevention of clogging and wastewater strength. Therefore, an appropriate drainfield sizing factor lies somewhere between 1.0 and 8.6.

Although it is an overextension of the correlation, Equation 17 might be used to give some indication of an appropriate sizing factor for RV septic effluent. Using the BOD_5 and TSS values for RV effluent given in Table 3, the sizing factor becomes:

$$\text{Sizing factor} = (1{,}430 + 170/250)^{1/3}$$
$$\text{Sizing factor} = 1.9 \quad (18)$$

Therefore, for lack of a better correlation, it is recommended that drainfields for RV septic tank effluent be double the recommended size for domestic septic tank effluent. This subject should receive further attention.

CONCLUSIONS AND RECOMMENDATIONS

RV wastewater is a very high-strength waste. Average total suspended solids, COD and BOD_5 values in this study were 3120 mg per liter, 8230 mg per liter, and 3110 mg per liter, respectively. The average volume of wastewater plus rinse discharged was 62 liters per vehicle.

Measured formaldehyde levels in septic tanks receiving RV wastes were about 5 to 10 mg per liter. BOD_5 of the effluent was about 1430 mg per liter. Total suspended solids were reduced to 170 mg per liter. Biological activity in the septic tank was evident from gas bubbles produced by the sludge.

Removal efficiencies for RV disposal septic tanks are higher than for domestic wastewater septic tanks. However, effluent from RV wastewater tanks is still about ten times stronger in BOD_5 and four times stronger in suspended solids than effluent from domestic tanks.

Several septic tanks sizing equations are used in design manuals. All are based on hydraulic detention times of about 24 to 36 hr. None addresses sludge and scum accumulation or pumpout interval.

A model was developed for sludge and scum accumulation in domestic septic tanks. The model, given by Equation 14, is based on a declining rate of degradation where some organic material in sludge and scum is readily degradable and compactible, some is degradable and compactible with extended residence times, and some material is inert and not compactible.

Because RV waste has a very high solids concentration and because anaerobic degradation may be inhibited by formaldehyde, sludge and scum accumulation should be considered when sizing septic tanks for RV disposal stations. Equation 16 was developed by applying the domestic sludge and scum accumulation model to RV waste.

The strong effluent from RV wastewater tanks may promote growth of a clogging mat and shorten the life of a drainfield. At the present time, it is recommended that drainfields for RV waste be twice as large as given by standard sizing criteria for domestic wastewater flowrates. This subject of drainfield sizing for concentrated effluent should be investigated further.

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Improved Percolation Test for Septic Tank Leach Field Systems

WILLIAM A. GROTTKAU and FRANK PEARSON

ABSTRACT

Septic tank systems are used at 50 percent of roadside rest areas in the United States for onsite disposal of wastewater generated from restrooms and from recreational vehicle waste holding-tank dump stations. The percolation test aids the sizing of septic tank leach fields by determining the percolation value for the soil, an index of the rate of seepage of water into the soil. The widely used Public Health Service percolation test procedure defines many aspects of the test, though some details are either discretionary or broadly defined. Comparative percolation tests were conducted to determine whether factors permitted to vary in the Public Health Service procedure could affect test results. Such factors investigated were: (a) test hole cross-sectional size; (b) method of excavation of test hole; (c) surface preparation of test hole; and (d) protection of interior surface of test hole. Based on findings of these comparative tests, certain precautions during testing are recommended to eliminate some causes of variation in test results, and a calculation is developed for adjusting raw data from percolation tests for the particular size of the test hole used. An improved percolation test method is proposed.

Figure 1 shows the distribution of waste disposal methods used at roadside rest areas in each Federal Highway Administration (FHWA) region and nationwide (1). Of 422 roadside rest areas surveyed nationwide, 50 percent were provided with septic tank systems, each treating waste flows up to 15,000 gallons per day (GPD). Although discharge of roadside rest area waste to municipal sewers is often favored where feasible, only in 6 percent of cases was this method actually employed, presumably because most roadside rest areas are in remote locations (1).

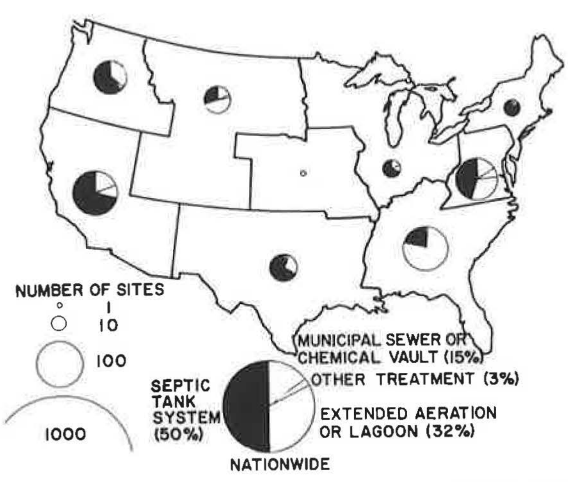

FIGURE 1 Roadside rest area wastewater disposal method methods according to FHWA region.

CHARACTERISTICS OF SEPTIC TANK SYSTEMS

Septic tank systems are relatively low in cost, easy to operate and maintain, and can tolerate fluctuations in loading and periods of nonuse. Where septic tank-leach field systems fail, failure is often manifested by surfacing of partially treated waste in the leach field. Common causes of such failure are:

1. Seepage following high precipitation;
2. Hydraulic overloading of the septic tank and leach field;
3. Failure to pump the septic tank with the result that septage overflows to clog the leach field;
4. Inadequate design of the leach field; and
5. Poor leach field construction.

PREDESIGN INVESTIGATIONS

The rate at which septic tank effluent will percolate into subsoil beneath the leach field is so site-specific that published or existing information can rarely be safely substituted for on-site investigations. Site investigations are made to evaluate the percolation characteristics of subsoil beneath the leach field trenches, and also to locate the maximum groundwater level under the leach field. A subsoil is considered suitable for a leach field if: (a) at the level of the leach field trench floor, the percolation value is between 5 and 30 min per in., and (b) groundwater remains at least 3 ft below the leach field trench floor (1). Where adverse subsoil or groundwater conditions exist, a sand filter might substitute for a leach field; sand filters are used in 15 percent of roadside rest area septic tank systems nationwide (1).

To assist in defining subsoil percolation characteristics, research was conducted by Van Kirk, Grottkau et al. (2) to develop a leach field percolation test procedure that appears more reputable than the Public Health Service procedure (3). The research concept was that some discretionary or broadly defined aspects of the Public Health Service percolation test procedure may affect test results. Based on findings of this research, a percolation test procedure was developed (2) that is consistent with, but more controlled than, Public Health Service and Environmental Protection Agency procedures (3,4).

EFFECT OF TEST HOLE BORE ON SOIL PERCOLATION VALUE

Existing Practice

The Public Health Service percolation test procedure (3) does not specify a particular cross-sectional shape nor plan dimensions for the percolation test hole. The flexibility permitted by that procedure in selecting the plan dimensions of the test hole evidently resulted from findings of a series of comparative percolation tests that showed no statistically significant variation of percolation value with test-hole size (5). However, these tests were all conducted in tight soil with a percolation value > 60 min/in., outside the FHWA recommended range of 5 to 30 min/in. (1).

Other Observations of Percolation Value Versus Test Hole Size

Other results indicate that for percolation tests in holes of differing sizes in a given soil, percolation value varies approximately directly with the bore of the hole. At Tempe, Arizona, percolation values were determined in three 3.3-in. bore holes and three 13-in. bore holes. Mean percolation rates were found to be 1.9 min/in. in the 3.3-in. holes, and 6.0 min/in. in the 13-in. bore holes (6). The ratio of these percolation values is 6.0/1.9=3.2, which compares to the diameter ratio of 13/3.3=3.9. At Portola, California, the percolation value measured in twenty 5-in. bore holes averaged 2.0 times the percolation value measured in paired 12-in. bore holes (7). Again, the diameter ratio of 12/5=2.4 only slightly exceeds the 2.0 ratio of percolation values. This pattern of observations can be explained theoretically.

Theoretical Effect of Test Hole Geometry on Test Results

Consider a vertical cylindrical test hole of a horizontal cross-section denoted A, and sectional perimeter C, so that the cross-sectional hydraulic radius is R=A/C. For a circular-section hole, the hydraulic radius is one-quarter of the diameter, that is, R=D/4. Water seeps through the wall and floor soil interface of the test hole at a particular interfacial velocity, v. This velocity is assumed to depend on the depth of submergence of the point in question, h, according to a power law, $v=kh^n$, where k is the constant and n is the exponent. Exponent values of 0.0, 0.5, and 1.0 are considered here, recognizing that the velocity of flow through porous media is commonly written as proportional to hydraulic gradient raised to an exponent that ranges from 0.5 for turbulent flow to 1.0 for laminar flow (8).

The decrease rate of the water volume stored in the test hole equals the total rate of water seepage through the floor and walls of the hole, as represented by:

$$Ae'dH/dt = k(AH^n + \int_0^H Ch^n dh) = kAH^n \{1+H/[R(n+1)]\} \quad (1)$$

where

- e' = hole porosity (presently taken as unity);
- H = depth of water in hole;
- t = time; and
- h = depth of submergence of an elemental annular slice of the hole wall surface.

By integrating Equation 1 (9), expressions for the time variation of water level can be obtained for n = 0.0, 0.5, and 1.0, respectively, by:

$$H_t = (R + H_0)[(R + H_T)/(R + H_0)]^{t/T} - R$$

$$H_t = 1.5R \tan^2 \{(t/T)[\mathrm{arctg}\sqrt{H_T/(1.5R)} - \mathrm{arctg}\sqrt{H_0/(1.5R)}] + \mathrm{arctg}\sqrt{H_0/(1.5R)}\}$$

$$H_t = 2R/\{(1+2R/H_0)[(1+2R/H_T)/(1+2R/H_0)]^{t/T} - 1\} \quad (2)$$

where H_t equals water depth at time t. Profiles of water level versus time computed by Equations 2-4 are reasonably linear and independent of exponent n for small changes in water level, as Figure 2 shows.

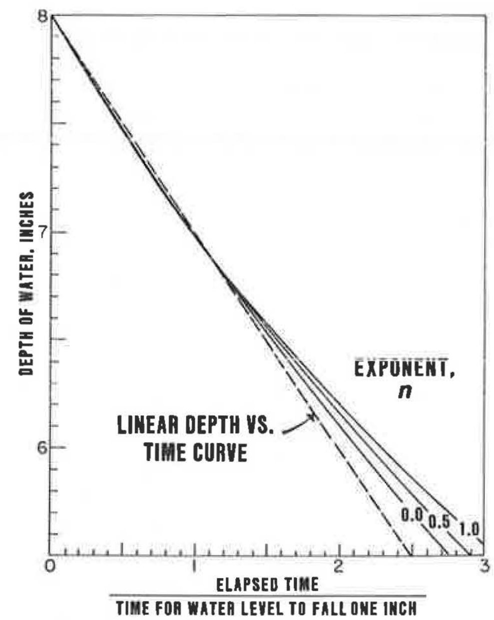

FIGURE 2 Example test hole depth versus time profiles by Equation 2.

However, percolation value will vary between tests in holes of differing cross-sectional size, that is, differing R. Given two holes of hydraulic radii, R_1 and R_2, with common initial water depths H_0, the respective water depths at any time during simultaneous percolation tests, H_1 and H_2, are related by:

$$H_2 = (R_2 + H_0)[(R_1 + H_1)/(R_1 + H_0)]^{R_1/R_2} - R_2$$

$$H_2 = 1.5R_2 \tan^2 \{\sqrt{(R_1/R_2)}[\mathrm{arctg}\sqrt{H_T/(1.5R_1)} - \mathrm{arctg}\sqrt{H_0/(1.5R_1)}] + \mathrm{arctg}\sqrt{H_0/(1.5R_2)}\}$$

$$H_2 = 2R_2/\{(1+2R_1/H_1)[(1+2R_2/H_0)/(1+2R_1/H_0)]^{R_1/R_2} - 1\} \quad (3)$$

For n = 0, 0.5, and 1.0, respectively.

Depths of water in the test hole at the start and end of the percolation test are H_0 at time 0 and H_T at time T, so the percolation value indicated by the test results is:

$$P = T/[e'(H_0 - H_T)] \quad (4)$$

Consequently, relative percolation values in test holes of different sizes are computed by substituting Equation 3 in Equation 4 written as:

$$P_2 = P_1[e'_1(H_0 - H_1)]/[e'_2(H_0 - H_2)] \quad (5)$$

where e'_1, e'_2, P_1, and P_2 are porosities and percolation values in test holes of hydraulic radii, R_1

and R_2, respectively. By combining Equations 3 and 5, a percolation value measured in a test hole of hydraulic radius R_1 may be adjusted to the equivalent value for a test hole of hydraulic radius R_2.

A simpler method of adjusting percolation test data for test-hole size uses the property that the depth versus time profile is fairly linear for small changes in water level as illustrated in Figure 2, so that Equations 1 and 4 can be combined to:

$$1/P \simeq e' dH/dt = kH^n \{1+H/[R(n+1)]\} \tag{6}$$

Then for n = 0:

$$P_2/P_1 \simeq (1+H/R_1)/(1+H/R_2) \tag{7}$$

If $H \approx H_0 = 8$ in. as recommended later herein, and P_2 is the percolation value for a 12-in.-bore test hole, then:

$$P_2/P_1 \simeq [1+8/(0.25D_1)]/[1+8/(0.25x12)] = 0.27+8.7/D_1 \tag{8}$$

where P_1 is the percolation value as measured in a D_1-in. bore hole.

Table 1 contains ratios of the percolation value in a 12-in.-diameter test hole to the percolation value in a test hole of lesser bore, computed by the preceding equations. Leach field design criteria are based on percolation values as determined in 12-in. test holes (10), so determinations in smaller holes should be adjusted to values for a 12-in. hole. Equations 3-8 predict higher percolation values in 12-in. test holes than in smaller holes, so percolation value determinations from smaller holes that are used for design without adjustment will produce an underdesigned leach field.

Equation 8, the simplest of the adjustment equations, generally overadjusts the results of a small-bore-hole percolation test. Equation 8 thus produces a safer design than other equations, provided the actual depth of water in the hole at the beginning of the test does not exceed 8 in. For initial water depths other than 8 in., Equation 7 safely approximates the adjustment factor. The data in Table 1 demonstrate that variations in test conditions--such as the initial depth of water, and the fall in water level during the test--may explain some of the variability in field determinations of percolation value. Further significant effects might be demonstrated by exploring a more exact analytical framework than Equation 1 provides, coupled with field investigations.

MAINTAINING THE PERVIOUS SOIL STRUCTURE IN PERCOLATION TESTING

The continued ability of a leach field to remove wastewater that it receives depends on establishing and maintaining an adequate wastewater seepage rate from the leach field into the subsoil. Failure of this seepage process can be caused by (a) the inherent impermeability of the subsoil, (b) intrusion of groundwater into the leach field, (c) destruction of the pervious structure of the subsoil, or (d) clogging of the subsoil by waste solids or biological growths.

The first two of these factors are identified through routine site investigations that include percolation tests. During these percolation tests, care is needed to maintain the pervious structure of the soil. Similar care is needed during construction and operation of the leach field. Otherwise, in percolation testing as in construction of the leach field, an otherwise suitable subsoil can become impermeable by compaction or smearing of the infiltrative subsoil interfaces, or by erosion of fines to the floor of the open excavation.

Augering of Test Hole

Power augering of a percolation test hole compacts excavated soil into the walls of the hole to a greater extent than hand augering. Compaction of soil into the walls of a percolation test hole during power-augering reduces the water seepage rate, thus increasing the percolation value.

TABLE 1 Factors to Adjust Percolation Values to Equivalent 12-Inch Bore Test Hole Percolation Values[a]

Initial depth of water in test hole, inches	Diameter of test hole, inches	Approximate solution for P_1/P_2 by Eq. 5c	Fall in water level during percolation test: One inch			Four inches		
			More exact solution for P_1/P_2 for assumed n					
			n = 0.0 Eq. 3a	n = 0.5 Eq. 3b	n = 1.0 Eq. 3c	n = 0.0 Eq. 3a	n = 0.5 Eq. 3b	n = 1.0 Eq. 3c
8	2	4.62	4.40	3.90	3.50	3.62	2.93	2.43
	4	2.45	2.36	2.16	2.00	2.04	1.77	1.57
	6	1.72	1.68	1.58	1.50	1.52	1.38	1.29
	8	1.36	1.34	1.29	1.25	1.26	1.19	1.14
	10	1.14	1.14	1.12	1.10	1.10	1.08	1.06
	12	1.00	1.00	1.00	1.00	1.00	1.00	1.00
24	2	4.62	5.35	5.08	4.83	5.06	4.68	4.33
	4	2.45	2.74	2.63	2.53	2.62	2.47	2.33
	6	1.72	1.87	1.82	1.77	1.81	1.74	1.67
	8	1.36	1.44	1.41	1.38	1.41	1.37	1.33
	10	1.14	1.17	1.16	1.15	1.16	1.15	1.13
	12	1.00	1.00	1.00	1.00	1.00	1.00	1.00

[a] Multiply tabulated P_1/P_2 value by measured percolation value to obtain equivalent 12-inch bore test hole percolation value.

TABLE 2 Effect of Augering Method on Percolation Rate (2)

Test location	Soil analysis, percent by weight			Percolation value, minutes per inch		Ratio of power auger percolation rate to hand auger percolation rate
	Sand	Silt	Clay	Power auger	Hand auger	
Transportation Laboratory,	39	39	22	46,53	0.3	38 (mean)
California Department				61,61	1.0	
of Transportation,				61,92	2.9	
Sacramento, California				122,122	3.8	
				122,122	4.4	
				122,182,375	6.1	
Dean Creek	33	52	15	14	0.7	43 (mean)
proposed roadside rest,				17	0.7	
near Garberville, Calif.				24	0.9	
				80	1.0	
Auburn Lake	--	--	--	>60	2.4	25
trails development,				80	0.8	100
near Cool, California				120	4.3	28
				240	10	24
				240	50	5

Table 2 summarizes results of tests at three locations to compare percolation values between power-augered holes and hand-augered holes. Percolation values measured in holes that were power-augered for their full depth averaged about 30 times higher than percolation values measured at the same sites in holes that were hand-augered for the final foot or more of depth. To minimize compaction of soil in the walls of the lower portion of a percolation test hole where the test is conducted, it is recommended that the final foot or more of hole depth be hand-augered.

Interior Surface Preparation of Test Hole

As mentioned earlier, the permeability of a cohesive subsoil can be sharply reduced as a result of smearing of tooled surfaces during excavation, or due to erosion of fines that can clog soil pores particularly on the floor of a ponded excavation. To minimize these possible effects before conducting a percolation test, hand-augered surfaces should first be scraped to roughen possibly smeared soil surfaces, and loose soil should be removed from the test hole.

Armoring of Test Hole

Protection is usually needed to avoid water scour or structural collapse of the carefully prepared surfaces of the percolation test hole during testing. The best way to accomplish this is by armoring the bottom of the test hole with a 2-in.-deep layer of 0.25-in.-sized pea gravel, and the walls with an approximately 0.75-in.-thick annular layer of pea gravel retained by a vertical length of perforated pipe. A piece of perforated pipe about 6 in. longer than the depth of the test hole should be centrally set on end on the bed of pea gravel, and more pea gravel should be placed between the pipe and walls of the hole.

Percolation values were compared between armored and unarmored test holes. The data in Table 3 indicate that in a cohesive soil (clay loam) the mean percolation value in 12 unarmored test holes was about 16 times the mean percolation in 6 armored test holes. Evidently, armoring of test holes protected their interior surfaces from scouring or collapse. Water added to an unarmored hole in clay loam produced a suspension of clay that appeared responsible for clogging soil pores. The data in Table 3 indicate an opposite trend, however, for granular soil, of a slightly higher percolation value in armored holes than unarmored holes; but this trend was statistically insignificant.

Gravel and perforated pipe occupy space in an armored test hole, so voids space (as measured by the volume of water needed to fill the hole) is less than if armoring materials were removed. Voids occupy the entire capacity of an unarmored hole so the porosity is unity. The porosity of an armored test hole is the voids fraction of the portion of capacity of the same hole without armoring that lies within the range of water level of the percolation test, which for a circular-section hole reduces to:

$$e' = e[1-(O/D)^2] + (I/D)^2 \quad (9)$$

where

e' = hole porosity;
e = pea gravel porosity;
D = test hole diameter; and
O and I = outside and inside diameters of perforated pipe, respectively.

With an armored test hole porosity of e' and unit porosity for an unarmored standard 12-in. test hole, then the joint correction for the hole size and armoring of the test hole results from combining Equations 8 and 9 by:

$$P_2/P_1 \simeq K = (0.27+8.7/D)/\{e[1-(O/D)^2] + (I/D)^2\} \quad (10)$$

TABLE 3 Effect of Pea Gravel Armoring of Test Hole on Percolation Rate

Test location and soil classification	Soil analysis, percent by weight				Percolation value, minutes per inch				Ratio of unarmored hole percolation value to armored hole percolation value
					Without armoring		With armoring		
	Gravel	Sand	Silt	Clay	Values	Mean (CV%)	Values	Mean (CV%)	
Transportation Laboratory, California Department of Transportation, Sacramento, California (clay loam)	0	39	39	22	20,24	50	0.3	3.1	16
					27,30	(57)	1.0	(70)	
					34,40		2.9		
					48,60		3.8		
					60,60		4.4		
					80,120		6.1		
Camp Roberts, northbound roadside rest area, near Paso Robles, California (sandy gravel)	25	57	6	8	2.0,4.0	4.1	3.3	9.3	0.4
					4.1,4.3	(27)	4.8	(71)	
					5.0,5.1		11.4		
							17.8		

where

K = correction factor;
P_2 = percolation value corrected to a 12-in.-diameter unarmored test hole, min/in.; and
P_1 = percolation value observed in a D-in. diameter armored test hole with an initial water depth of 8 in., min/in.

Presoaking and Adding Water To the Test Hole

Overnight presoaking of a percolation test hole before starting the test will allow cohesive soils to swell, and it will establish pseudo-steady-state seepage from the hole as during operation of a leach field at the site.

A domestic toilet-type float valve can be adapted to maintain a steady depth of water in the test hole during the presoaking period, provided water pressure at the site is adequate to operate the valve. Water should be introduced gently and to the bottom of the hole to avoid scouring the soil. For manual filling of the hole, the water supply hose can be connected to a valved section of 3/8-in.-diameter soft copper tubing long enough for a gentle stream to be directed to the bottom of the hole.

Water Level Measurement

Percolation testing involves measuring the fall in water level in a prepared test hole during a timed interval. The Public Health Service procedure (3) recommends measuring the fall in water level with the aid of two stakes: a movable vertical pointed stake, and a fixed horizontal reference stake fastened above the test hole to posts on either side of the hole. At the start and end of the timed test interval the vertical stake is supported with its point in contact with the water surface and scribed against the horizontal reference stake. The fall in water level over the timed interval is then measured as the distance between the marks scribed on the vertical stake.

This method was found to be rather awkward in practice and gave slightly variable results with discrepancies between replicate readings by different observers averaging 3/16 in. (6). A float gauge was found easier to use and was judged more accurate for indicating water level changes in the test hole. Such a gauge was fabricated from a plastic bottle, small enough to fit inside the perforated pipe, with a rod calibrated in inches (increasing downwards) fastened into the neck of the bottle. The gauge floats in the test hole, rising and falling with varying water level in the hole. Changes in water level in the test hole are read as differences between readings on the calibrated rod against an adjacent fixed reference point.

This float gauge may also be used as an aid to adjustment of the depth of water over the pea gravel surface to a specified value (6 in.) at the start of each timed interval in the percolation test. This is accomplished by reading the gauge first when depressed to rest on the pea gravel, then again when released to float on the water. Water is added to, or removed from, the hole until the reading with the floating gauge exceeds that for the depressed gauge by an amount equal to the specified depth of water over the pea gravel (6 in.) minus the draft of the gauge. (The draft of the gauge is the minimum depth of water needed to float the gauge, measured one time for a particular gauge. To measure its draft, the gauge is placed in an empty bucket and water is trickled in until the gauge begins to float, whereupon the draft is measured as the depth of water in the bucket without removing the gauge.)

Percolation Test Procedure and Results

A test procedure is proposed in the following section of this paper, based on the preceding considerations. In this procedure, the time for the test-hole water level to fall a measured amount ($\leq$ 1 in.) is recorded and adjusted by Equation 10 according to specific details of construction of the test hole. This simulates test conditions in a 12-in.-bore open test pit similar to that used by Ryon (10), upon whose work leach-field design criteria are based (3).

As was indicated in Tables 2 and 3, water seeps more rapidly from test holes into subsoil if precautions are taken to reduce compaction of soil into the test hole walls during excavation of the hole, and if the interior surfaces of the test hole are roughened, loose material removed, and the prepared surfaces protected by armoring. A higher rate of seepage translates into economy in leach field design, provided precautions to maintain the pervious structure of the soil are as stringent during construction of the leach field as during percolation testing. Otherwise, the ability of a cohesive subsoil to accept water or wastewater can be seriously impaired.

Leach Field Construction Considerations

Precautions necessary during construction of a leach field to protect the pervious structure of the subsoil include: (a) working only in dry weather and above groundwater; (b) closing a leach field trench overnight; (c) hand removal after machine excavation of any smearing or consolidation of the trench walls; (d) removing loose material from the trench floor before placing gravel; and (e) using only clean, uniformly graded gravel protected from contamination by fines before use and during use.

TEST METHOD FOR DETERMINING SOIL PERCOLATION VALUE

Scope

This test is an aid to sizing septic tank leach field systems. The test determines the percolation value of a soil, an inverse index of the tendency for water to seep into the soil. Percolation value is determined from measurements of the fall in water level in a prepared hole in the soil over a timed interval.

Apparatus

Some of the following items are illustrated in Figure 3:

- Six-in. diameter hand auger,
- Hole scraper (Figure 3a),
- Hole cleanout tool (Figure 3b),
- Stopwatch,
- Supply of water, for example, tanker truck,
- One per test hole of each of the following items:
  - Float valve (perhaps adapted from a toilet cistern valve as in Figure 3c), to be operable at pressure of available water supply;
  - Perforated PVC pipe, 4-1/2-in. outer diameter, about 6 in. longer than depth of hole (Figure 3d);

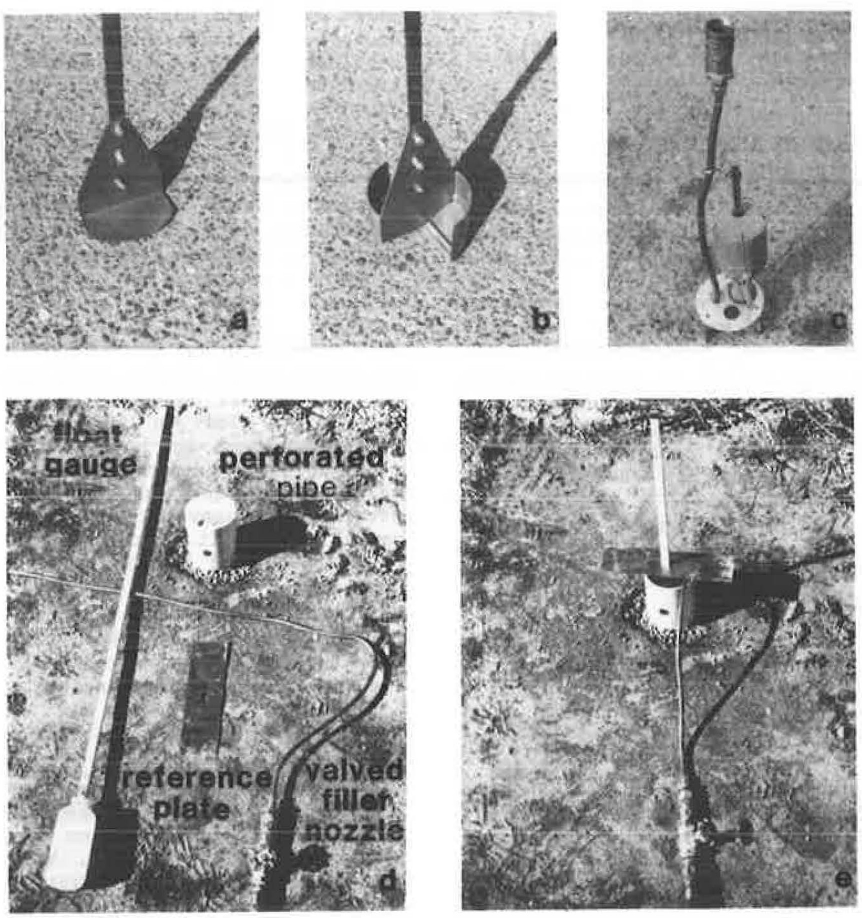

FIGURE 3 Some items of equipment for percolation testing: (a) scraper tool; (b) hole cleanout tool; (c) float value; (d) unassembled test apparatus; (e) assembled test apparatus.

- Float gauge fabricated from plastic bottle, with rod calibrated in inches (increasing downwards) fastened into the neck (Figure 3d);
- Reference plate, for example, 12 in. x 6 in. x 16g steel, slotted slightly larger than float gauge rod (Figure 3d);
- Valved filler nozzle of 3/8-in. soft copper tubing about 2 ft longer than the perforated pipe, connected through a gate valve and hoses to the water supply (Figure 3d); and
- Pea gravel, sized approximately 1/4 in., about 1/6 ft^3.

Determine the porosity of the pea gravel, and measure the inside and outside diameters of the perforated pipe, and the draft of each float gauge. Compute the correction factor by Equation 10.

Visual Inspection of Soil Profile

Excavate by backhoe or borings and document the vertical profile of soil strata at the site, noting particularly conditions that may impede drainage from the leach field, such as hardpan (4). Confirm that groundwater levels remain at least 3 ft below the leach field invert level. If monitoring of the maximum groundwater level is necessary, simple ways to record the high water mark in a test pit are: (a) sprinkle a conspicuous floatable powder in the pit, for example, cork dust; or (b) place in the pit a stake with a water-soluble coating, for example, blackboard chalk.

Preparation of Test Holes

Usually six or more test holes are dug and distributed to represent conditions over the entire leach field. More test holes may be needed later if some are found to have percolation values outside the FHWA-recommended range of 5 to 30 min/in. (1). The following procedure applies for each test hole and is illustrated in Figure 4:

- Machine augering is permissible to within 12 in. from the bottom of the test hole (Figure 4a);
- Hand-auger for the final 12 in. or more of depth (Figure 4b);
- Scrape the lowest 12 in. of sidewall and remove loose soil from the hole (Figure 4c);
- Place a 2-in. depth of pea gravel in the hole (Figure 4d);
- Centrally set a perforated pipe on end in the hole (Figure 4e);
- Backfill the annular space between the perforated pipe and the hole walls with pea gravel for 12 in. of depth (Figure 4f);
- Install float valve and connect to water supply (Figure 4g);
- Presoak hole by maintaining constant water level in hole at least 6 in. above pea gravel at bottom of hole, for 18 hr or more (Figure 4h);
- Adjust depth of water over pea gravel to 6 in. with float gauge in place (Figure 4i);
- Repeat the following procedure at least three times until a stable percolation value is obtained:
  - Adjust the water level to 6 in. above the surface of the layer of pea gravel in the bottom of the hole, and start the stopwatch at zero (Figure 4j);
  - Record the time in minutes for the water level to fall an inch or measured fraction of an inch (Figure 4k); and
- Calculate (Figure 4l):

Percolation value = correction factor x time, in minutes/fall in water level, in inches.

Table 4 contains an example of data collection and reduction. A stabilized percolation value in the range 5 to 30 min/in. is considered suitable for a leach field, provided groundwater does not rise closer than 3 ft below the invert of the leach field trenches (1).

TABLE 4 Example Percolation Test Data

Parameter	Value		
Test location	Translab, Sacramento		
Test date	September 11, 1982		
Test made by	J. Van Kirk		
Weather	Clear, sunny, 70-75 F		
Type of soil	Silty loam		
Presoaking period, hr	24		
Test hole diameter, D, in.	6		
Perforated pipe OD, O, in.	4 1/2		
Perforated pipe ID, I, in.	4 1/4		
Pea gravel porosity, e	0.4		
Test hole number	1	2	3
Test hole depth, in.	43	40	42
Initial gauge reading, in.[a]	13 3/8	12 1/4	13
Interval of test readings	1 in.	10 min	30 min
Test reading #1	2m 39s	13 in.	13 3/4 in.
#2	2m 50s	13 1/8 in.	13 5/8 in.
#3	2m 55s	13 in.	13 7/8 in.
#4	2m 53s	13 in.	13 7/8 in.
#5	2m 54s	13 in.	13 7/8 in.
Raw percolation value,	2.9	10/(13-12 1/4)	30/(13 7/8 -13)
min/in.		= 13.3	= 34.3
Correction factor[b]	x 2.54	x 2.54	x 2.54
Percolation value, min/in.	= 7.4	= 34	= 87

[a] Water added to give this gauge reading before each test interval.

[b] $K = (0.27 + 8.7/D)/\{e[1-(O/D)^2] + (I/D)^2\}$

$= (0.27+8.7/6)/\{0.4[1-(4.5/6)^2]+(4.25/6)^2\} = 2.54$

CONCLUSIONS

The percolation test aids the sizing of septic tank leach fields by determining the percolation value for the soil, an index of the rate of seepage of water into the soil. The widely used Public Health Service percolation test procedure defines many aspects of the test, though some details are discretionary or broadly defined. Comparative percolation tests were conducted to determine whether factors permitted to vary in the Public Health

FIGURE 4 Steps of proposed method for percolation test: (a) machine-auger except for final foot of depth; (b) hand-auger final foot of depth of test hole; (c) scrape lowest foot of sidewall and remove loose soil; (d) place two inches of pea gravel on bottom of hole; (e) install perforated pipe; (f) backfill between pipe and hole walls with pea gravel; (g) install and connect float valve; (h) presoak test hole for at least 18 hours; (i) float gauge indicates depth of water over gravel; (j) adjust depth of water over gravel at start of test; (k) record time for water level to fall measured distance; (l) compute percolation value.

Service procedure could affect test results. Such factors investigated were:

- Test hole cross-sectional size,
- Method of excavation of test hole,
- Surface preparation of test hole, and
- Protection of interior surface of test hole.

Based on findings of these comparative tests, precautions during testing are recommended to eliminate some causes of variation in test results, and a calculation is developed for adjusting raw data from percolation tests for the particular size of the test hole used. An improved percolation test method is proposed.

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Onsite Disposal of Restroom and Recreational Vehicle Wastes

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ABSTRACT

Septic tank systems are used at 50 percent of roadside rest areas in the United States for onsite disposal of wastewater generated from restrooms and from recreational vehicle waste holding tank dump stations. Survey results are presented from 28 California roadside rest areas of the use of rest areas, and of the volume and strength of wastewater generated at restrooms and dump stations. Traffic densities in peak months averaged 24 percent higher than the annual mean, while peak holiday weekend densities averaged 86 percent higher for facilities serving one direction of traffic. A mean of 12 percent of mainline traffic used the rest areas, and of the traffic using rest areas that provided dump stations, 2 percent were recreational vehicles that actually dumped. Restrooms generated 5.5 gal of waste per vehicle, and dump stations generated 12 gal of wastewater plus 9 gal of washdown water per dump. Restroom wastewater is comparable in strength to domestic wastewater, but dump station wastewater (diluted by washdown water) produces about 20 times the quantity of sludge as the same volume of domestic wastewater. Depending on the proportion of dump station waste and the frequency of pumping the septic tank, rest area septic tanks should be sized to provide 1.5 to 30 days detention of diluted dump station wastewater, compared to 1.5 days for a domestic septic tank. Septic tank-leach field system design procedures consider the risk of overload for a particular design, or permit design to a selected acceptably low risk of overload.

Restroom toilets so predominate among roadside rest area amenities that a rest area may have to be closed if its waste disposal system fails. Being distant from city sewers, most rest areas must dispose of the wastewater they generate onsite. One-half of the roadside rest areas surveyed in the United States used septic tank systems for wastewater disposal (1). The design of onsite wastewater disposal systems for roadside rest areas are addressed in this paper, with emphasis on septic tank systems.

EFFLUENT QUALITY REQUIREMENT

Section 301b of the 1972 Amendments to the Federal Water Pollution Control Act (P.L. 92-500) requires a level of effluent quality of point waste discharges equivalent to secondary treatment (2) (i.e., that level generally specified for municipal discharges). Section 402 of this Act requires monitoring of the quality of effluent discharges greater than 50,000 gal per day. For lesser discharges (as from most roadside rest areas) the question of whether effluent quality monitoring is required is left to the discretion of the permit-issuing authority. Several states surveyed by FHWA required monitoring of minor discharges to document proper operation and maintenance of the facilities, rather than to prove compliance with effluent limitations. Under those circumstances, septic tanks (STs) for treatment of roadside rest area waste would comply with the requirements of P.L. 92-500 (3), though states are empowered to determine the adequacy of proposals for waste treatment at individual sites. Also, some states ban recreational vehicle (RV) waste-holding tank preservatives containing a nonbiodegradable active ingredient, such as zinc (4).

LOADINGS ON ROADSIDE REST AREA WASTE DISPOSAL FACILITIES

Of 95 roadside rest areas (RRAs) operating in California during this 1978-1980 study, the 28 that were investigated are identified in Figure 1 (5). All RRAs surveyed provided toilet facilities; 17 also provided trailer sanitation stations (TSSs) for dumping RV waste holding tanks. Traffic density was measured on the mainline near each RRA (as annual mean density, annual 7-day peak density, and annual 3-day peak density), and also on ramps serving the RRA and TSSs. Water use and waste volumes were measured, and waste was sampled for chemical analysis (6) at selected RRA restrooms and TSSs identified in Figure 1. Measurements of restroom or TSS water consumption and waste production were correlated against RRA or TSS ramp axle counts, and against counts of vehicles or restroom visitors when taken.

Mainline Traffic Peaking Factors

Traffic density varies according to diurnal, weekly, and seasonal patterns and long-term trends, superposed by random fluctuations. By averaging traffic density over each of a series of equal time intervals, short-term fluctuations tend to be eliminated. The averaged record of traffic density becomes smoothed with less extreme peaks and valleys than the smoothed record, and the degree of smoothing increases with the duration of the averaging interval.

In design of RRA facilities, the duration of the averaging interval for smoothing a record of traffic density has to be matched to the load-smoothing characteristics of the facility being designed. Various averaging intervals have been suggested for design of the leach field in a septic tank-leach field (ST-LF) system. Use of a one-month averaging interval (7,8) is based on information that the typical hydraulic retention time in an LF is two to four weeks (9). For design of RRA water supplies, a 1-day averaging interval (i.e., design for the peak day) has been recommended, with storage for smoothing shorter duration (diurnal) peaks (10). Designing for the mean traffic density over the six peak 3-day weekends is an alternative to designing for the peak month (7,8,10).

Others recommend designing RRA waste disposal facilities to effectively treat diurnal peaks. One such approach is to design for a peak 8-hr flow rate of twice the 24-hr-averaged rate, then to judge effects of hourly flow variations on the design (3,11). Another approach is to design for a peak hourly waste flow rate of 3.2 to 3.6 times the 24-hr-averaged rate, equivalent to an hourly waste volume of 13.5 to 15 percent of the daily volume (12-15).

Many of the preceding recommendations derive from a study conducted in 1975 by the FHWA, U.S. Department of Transportation, entitled, Establishment of Roadside Rest Area Water Supply, Waste Water Carriage, and Solid Waste Disposal Requirements. The study contained results of more than 300 days of observation at each of five RRAs in Colorado, Flor-

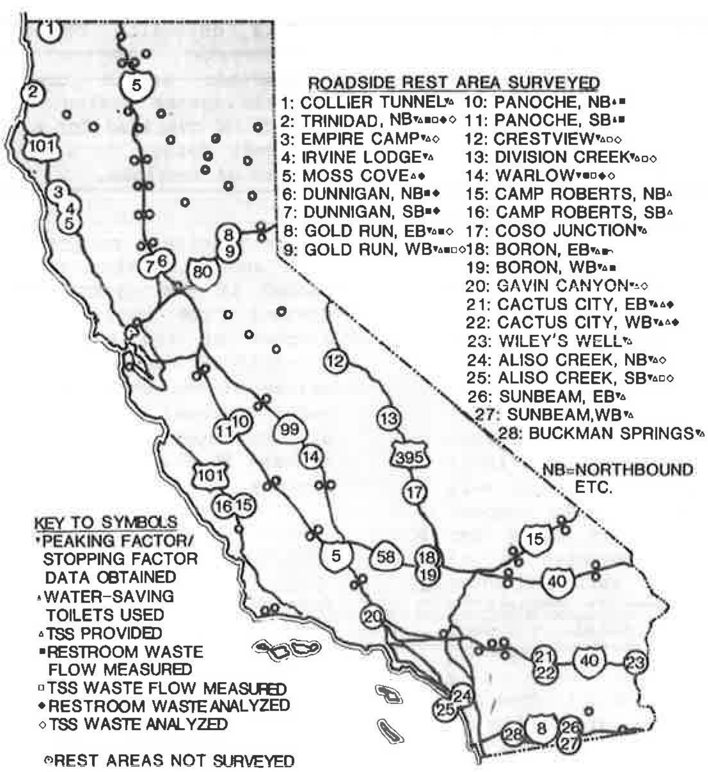

FIGURE 1 California roadside rest areas surveyed in this study.

ida, Iowa, New Hampshire, and Tennessee. This study revealed that 51 to 63 percent of RRA use occurred between 8:00 a.m. and 4:00 p.m., with standard deviations of 7 to 12 percent. RRA use in the peak hour of each day averaged 6 to 11 percent of total use at the five locations, with a standard deviation of about 4 percent. The authors proposed using peak 8-hourly and 1-hourly traffic densities of 2.0 and 3.75 times the mean daily density, equivalent to 67 percent of the daily flow in the peak 8-hr and 16 percent of daily flow in the peak hour.

Design criteria for ST-LF systems for applications such as RRAs are largely based on experience with domestic ST systems. Domestic STs routinely handle typical household diurnal and weekly waste load fluctuations and perhaps even longer-period fluctuations. Only load fluctuations of a longer period than domestic systems typically experience require explicit consideration when projecting from domestic system design bases.

The peaking factor (PF) is the ratio of averaged peak traffic density to average density. Table 1 gives PFs for peak averaging periods from 3 to 30 days, as measured near 12 California RRAs for 2 years. To compute these PFs, average traffic densities and monthly (30-day averaged) peaks were obtained from published records (16,17), and shorter term peaks were recorded around holiday weekends (5). The data in Table 1 indicate both one-way and two-way PFs, respectively, with the higher of the two opposing traffic densities as a multiple of average one-way density (for design of one-way facilities), and the sum of the two opposing densities as a multiple of average two-way density (for design of two-way facilities). Two-way PFs were less than one-way PFs by up to 8 percent because of desynchronization of peaks in opposing directions.

Stopping Factors

The RRA stopping factor (RRA SF) is the ratio of traffic density on the RRA ramp to the average daily traffic (ADT). Similarly, the trailer sanitation station (TSS) SF is the ratio of the density of RVs dumping at the TSS to the density of traffic on the RRA ramp. The mean of 39 RRA SFs measured at 20 California RRAs during 1979-1980 summer holiday weekends was 0.12 with a standard deviation (SD) of 0.06, whereas the mean of 35 TSS SF was 0.021 with an SD of 0.016. Previously cited values of the RRA SF include 0.006 to 0.253 from an FHWA study (18), 0.109 to 0.129 for some Illinois RRAs (19), 0.06 to 0.16 for certain Minnesota RRAs (20), 0.06 to 0.09 for one RRA in each of four states--Colorado, Florida, Iowa, and New Hampshire--with SDs of 0.01 to 0.02 for about 300 sampling days in each case, and 0.06 to 0.08 from guidelines for design of California RRAs (21). No data on TSS SFs were obtained from the literature.

In this study, TSS SFs were based on daylight traffic counts excluding any unusually high count in a 15-min interval. Also, TSS SFs for each site were adjusted individually for the observed fraction of traffic that passed through the TSS ramp without dumping; this fraction varied among sites from 0 to 99 percent, with a mean of 53 percent. The mean number of axles per vehicle was 2.5 for RRA ramps and 2.8 for RVs dumping at TSSs; the standard deviation of the daily mean number of axles per vehicle is thought to have been about 0.2 in each case.

Roadside Rest Area Waste Characteristics

RRA waste is characterized by the volume of waste generated per unit axle count on the ramp serving the facility, and also by the strength of the waste. Both the volume and the strength of RRA waste differ between restroom wastes and TSS wastes. Further, different types of restroom waste and different types of TSS waste each have fairly distinctive characteristics. The volume and strength of restroom waste depends on whether conventional toilets or water-saving toilets are installed. Water-saving toilets generate a lesser volume of waste than conventional toilets, but water-saving toilet waste is stronger.

TSS waste comprises two flow streams: waste dumped from the RV holding tanks, and washdown water from a faucet at the TSS for cleanup after dumping. Some RVs are equipped with two waste holding tanks, one for the strong toilet wastes (black water) and the other for the weaker wastes from the kitchen, bath, and basin (gray water). Other RVs use a single tank for both types of waste (mixed black and gray water tank).

The strength of RRA waste is characterized by several chemical parameters. Most parameters express the concentration of a particular type of pollutant in the waste, in units of milligrams of pollutant per liter of wastewater (mg/L). "Catch-all" chemical analytical procedures are used to define some of these parameters, such as the total concentration of liquefied organic putrescible matter as determined by the soluble chemical oxygen demand (sCOD) test,

TABLE 1 Peaking Factor for Various Averaging Periods

Peak averaging period, days	Peaking Factor (PF) (Peak-to-average traffic density ratio)				Ratio of two-way PF to one-way PF
	One-way traffic		Two-way traffic		
	Mean	Standard deviation	Mean	Standard deviation	
30	1.24	0.09	1.24	0.09	1.0[a]
14	1.47	0.31	1.39	0.29	0.95
7	1.58	0.32	1.50	0.30	0.95
3	1.86	0.50	1.71	0.47	0.92

[a] Assumed value; monthly traffic densities were not separated by direction.

and the concentration of sludge- and scum-producing solids by the suspended solids (SS) test. For other pollutants, chemically definitive analyses are used, as for the nutrients ammonia and phosphate. In the context of RRA waste treatment, the active ingredients of chemical preservatives added to RV waste holding tanks for odor control are significant (e.g., formaldehyde, zinc, and phenol). These preservatives in waste discharged from RV holding tanks interfere with biodegradation of organic matter in treatment processes. Biological waste treatment process units (e.g., septic tanks) may have to be enlarged if the waste contains a preservative.

Figure 2 shows volume and strength characteristics for wastes from the RRA restrooms and TSSs surveyed. Each numbered frame depicts a specified volume or strength characteristic of a certain type of RRA waste. The two left columns of frames in Figure 2 are for restroom wastes, and the three right columns are for TSS wastes. Frames in the upper row show the volume of waste, whereas frames in the center and lower rows show two quality characteristics of the waste, namely sCOD and SS. The horizontal axis of each frame is the RRA number, designated in Figure 1. The following discussion of waste characteristics refers to these Figure 2 data.

Volume of Waste from Restrooms with Conventional Toilets

Restroom waste volume is expressed as gallons per unit axle count on the RRA ramp. For restrooms with conventional toilets, a mean of 2.2 gallons of waste was generated per axle count (gal/axle), with an SD of 0.9 gal/axle (Frame 1, Figure 2). This mean of 2.2 gal/axle multiplied by 2.5 axles/vehicle equals 5.5 gal/vehicle, which compares with previously cited waste production data: 7.6 gal/vehicle according to an FHWA study (18), 5.5 gal/vehicle for a Tennessee RRA (22), and 3.6 to 4.7 gal/vehicle for RRAs in Colorado, Florida, and New Hampshire with SDs of 1.0 to 1.4 gal/vehicle for 200 to 300 sampling days at each station. During the three peak summer months, mean waste production increased from 4.4 to 6.1 gal/vehicle with standard deviations of 1.1 to 2.3 gal/vehicle for samples of about 60 sampling days.

Volume of Waste from Restrooms with Water-Saving Toilets

Water-saving toilets had been installed in restrooms at two RRAs that were newly opened at the time of the waste survey. For these restrooms, Frame 2 of Figure 2 shows a mean and SD of waste volume of 0.24 and 0.03 gal/axle, respectively. These toilets had been adjusted to deliver about 1 gal/flush for effective cleansing, although adjustment of water-saving toilets to deliver as little as 1 quart per flush is possible; conventional toilets typically use about 5 gal/flush. The measured mean waste production per restroom visitor was 0.95 gal with an SD of 0.42 gal, based on hourly waste volumes and visitor counts. Hutter (23) reported (for water-saving toilets of the same Microphor manufacturer) a waste production of 1.5 gal/visitor at RRAs in New Mexico, and 1.3 gal/visitor at a Colorado RRA.

Volume of TSS Waste

TSS waste volume is expressed per unit axle count on the TSS ramp, excluding spurious axle counts not associated with dumping events as previously described. Volumes of RV waste, TSS washdown water, and waste plus washdown water per TSS axle are plotted in Frames 3 to 5 of Figure 2. Each plotted point represents the mean volume from about ten dumping events at that TSS. The mean and SD of the volume of RV waste were 4.4 and 0.7 gal/axle, or 7.5 and 1.3 gal/axle for RV waste plus washdown water. The mean consumption of washdown water was 3.2 gal/axle, but consumption was only 1.2 gal/axle at one site where a spring-loaded washdown faucet was installed, with a fine (1/4-in.) nozzle on the washdown hose.

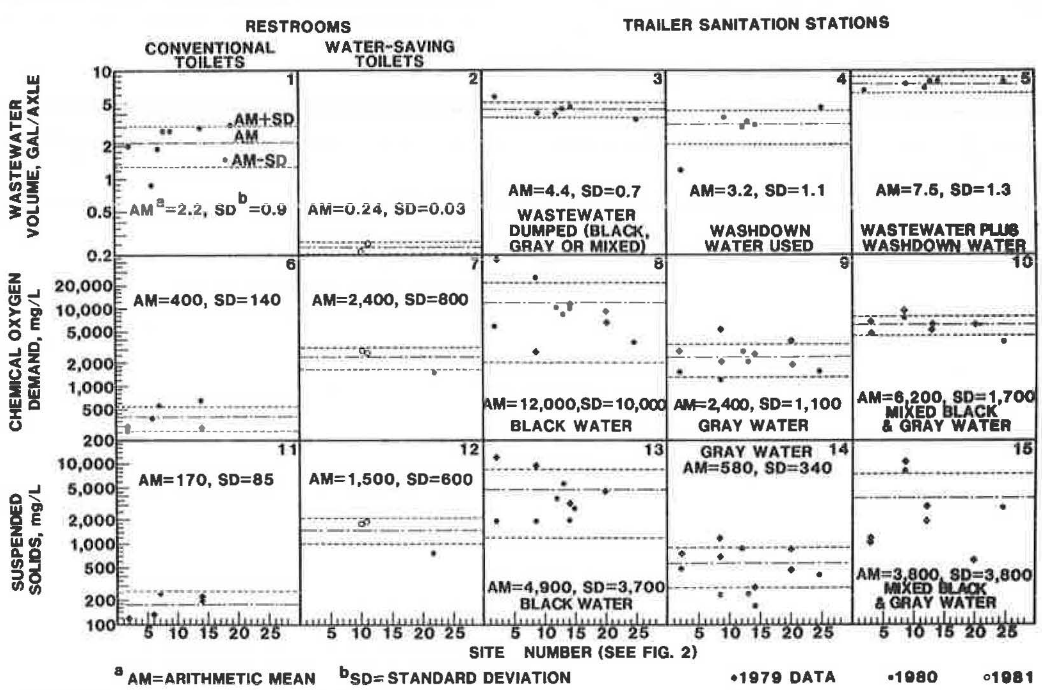

FIGURE 2 Volume and strength characteristics of roadside rest area wastewaters.

Strength of Waste from Restrooms with Conventional Toilets

Restrooms with conventional toilets produced waste with a mean sCOD of 400 mg/L and an SD sCOD of 140 mg/L (Frame 6, Figure 2); the mean and SD of waste SS were 170 and 85 mg/L, respectively (Frame 7). Values from 5 to 12 times these values were determined on several samples of waste from restrooms with conventional toilets, but these high values were rejected as probably resulting from nonrepresentative sampling of the waste; access to the waste flows was generally difficult. The mean observed COD of 400 mg/L and SS of 170 mg/L compare with values of 360 and 180 mg/L, respectively, suggested for RRA restroom waste by Hughes et al. (3). Typical levels of COD and SS in domestic wastewater of 500 and 200 mg/L, respectively (24) are also similar.

Strength of Waste from Restrooms with Water-Saving Toilets

Water-saving toilets produced waste having a mean and SD sCOD of 2,400 and 800 mg/L, and a mean and SD SS of 1,500 and 600 mg/L (Frames 7 and 12, Figure 2). Water-saving toilets produced about one-eighth the volume of waste from conventional toilets, but water-saving toilet waste was about 8 times stronger.

COD and SS of TSS Waste

sCOD and SS data for RV black water, for RV gray water, and for RV mixed black and gray water appear in Frames 8 to 10 and 13 to 15 of Figure 2. RV black water was green in color due to the mix of urine with blue preservative colorant; it had a mean COD of 12,000 mg/L (Frame 8) and a mean SS of 4,900 mg/L (Frame 13). Both the COD and the SS of RV black water were about 30 times higher than for restroom waste, and 24 times higher than for domestic waste. Each RV waste sample was composited from subsamples collected from about 8 RVs, undiluted by washdown water. However, after dumping a waste holding tank at a TSS, RV waste is diluted by washdown water from a faucet at the TSS. Frames 3 and 5 of Figure 2 show that a mean of 4.4 gal/axle of RV waste is diluted to a mean of 7.5 gal/axle of waste plus washdown water. Then for RV waste diluted by washdown water, the mean COD and SS may be estimated as 12,000 x 4.4/7.5 = 7,000 mg/L, and 4,900 x 4.4/7.5 = 2,900 mg/L, respectively; these values measure the strength of waste entering treatment.

Preservative Content of TSS Waste

TSS waste holding tank samples were also analyzed for the preservatives formaldehyde, zinc, and phenol. These analyses indicated that RV odor control chemicals are frequently used in RV black water tanks, but infrequently in RV gray water tanks and RV mixed black and gray water tanks. Ten samples of RV black water contained a mean of 300 mg/L of formaldehyde, with an SD of 350 mg/L and a maximum of 960 mg/L. For comparison, the typical calculated concentration of formaldehyde in a waste holding tank to which the manufacturer's recommended dosage of formaldehyde-based preservative has been added is 1,350 mg/L (6). The mean formaldehyde content of black water after dilution by washdown water is calculated to be 300 x 4.4/7.5 = 180 mg/L. Zinc was present in RV black water at a mean concentration of 75 mg/L in 1978, but declined to below 1 mg/L in 1979 after zinc-based preservatives were banned in California (4). Phenol was present at below 3 mg/L in RV black water. A fourth preservative, quaternary ammonium, was not determined in TSS waste, but it was listed as the active ingredient by a few preservative manufacturers.

Effect of Preservative on ST-LF Effluent Quality

As a part of research conducted on treatment of RRA wastes, pilot ST-LF units were operated with measured dosages of formaldehyde spiked into the sewage feed to simulate TSS waste feed (6). Pilot units were evaluated for removal of sCOD, formaldehyde, and other chemical parameters over a range of waste flow rates. At one of the flow rates used, the daily volume of sewage treated equaled the liquid capacity of the ST (1.0 days ST detention), and each day, each square foot of the LF received 2.9 gal of ST effluent (2.9 gal/sf-day LF loading). Under these conditions, 65 percent of sCOD and 94 percent of formaldehyde were removed when the waste was spiked with 90 mg/L of formaldehyde. But with an increasing formaldehyde dosage, the effluent deteriorated. At a formaldehyde dosage of 360 mg/L, removals of sCOD and formaldehyde declined to 43 and 85 percent. Formaldehyde was lost by degradation and volatilization.

Effect of Preservative on Septage Accumulation Rate

Besides causing the effluent to deteriorate, preservatives also reduce the rate of degradation of septage (sludge and scum) in an ST, resulting in an increased net rate of accumulation of septage. For the pilot ST-LF units, over a 9-month period of operation, an ST receiving formaldehyde-free sewage accumulated 1.27 gal of septage for each pound of SS in the waste feed. This 1.27 gal compares to 1.76 gal of septage per pound of SS for an ST receiving sewage dosed with a mean of 200 mg/L of formaldehyde. That is, spiking the waste feed to an ST with 200 mg/L of formaldehyde increased the rate of accumulation of septage by a factor of 1.76/1.27 = 1.39.

The high SS content of TSS waste also increases the rate of accumulation of septage, compounding the effect of preservative. RV black water diluted by washdown water contained 2,900 mg/L SS, compared to 200 mg/L in domestic waste (24). Consequently, the net rate of accumulation of sludge and scum from RV black water is higher than that from the same volume of domestic waste by a factor 1.39 x 2,900/200 = 20.

To make a TSS ST as safe against overfilling with septage as a domestic ST, the TSS ST volume per unit flow needs to be 20 times greater than for an ST for domestic waste. If such a large factor is considered excessive, it may be reduced from 20 to some lower value provided septage is pumped from the ST before it flows into the LF. If a TSS ST were constructed with a liquid capacity four times greater than for a domestic ST treating the same waste flow, it is calculated that the TSS ST would fill with septage in about 12 months. In 3 months, a TSS ST would accumulate a volume of sludge equal to the liquid capacity of a domestic ST treating the same waste flow.

A parameter, designated k, expresses the ratio of the sludge accumulation in a TSS ST to the capacity of a domestic ST treating the same waste flow, given the TSS ST pumpout interval. From the preceding, k=20 for a TSS ST pumped according to the same schedule as a domestic ST, or k=4 for a maximum TSS ST pumping interval of 12 months, or k=1 for a maximum TSS ST pumping interval of 3 months. But if two-thirds of the TSS ST capacity is needed for settlement and only one-third is allocated to storage of sludge then the TSS ST should be pumped at one-third of the above maximum intervals. Reference is made to a frequently pumped ST, pumped perhaps three times a year at design load (k=4), or an occasionally pumped ST (k=20).

ROADSIDE REST AREA SEPTIC TANK-LEACH FIELD SYSTEM DESIGN BASIS

The following procedures apply to the design of small (<15,000 gal per day) facilities to which conditions listed in Table 2 pertain.

TABLE 2 Parameters Determining RRA Waste Flow and ST Volume

Symbol, parameter and unit	Mean	Standard deviation
Q design waste flow, gal/day	a	a
V septic tank liquid capacity, gal	a	a
T annual peak 3-day mainline traffic density, vehicles/day	b 1.3ADT	150
S RRA stopping factor	0.12	0.06
A axles per vehicle entering RRA	2.5	0.2
G restroom waste volume, gal/RRA axle	2.2	0.9
s TSS stopping factor	0.021	0.016
a axles per vehicle dumping at TSS	2.8	0.2
g TSS waste volume, gal/TSS axle	7.5	1.3
k Corrector for high strength and preservative content of TSS waste	20	-

[a] dependent variables;

[b] ADT = average daily mainline traffic.

Design Waste Flow

With notation as listed in Table 2, the flow rate of RRA waste is:

$$Q = TS(AG + sag) \quad (1)$$

where

TS = the peak number of vehicles entering the RRA in question;
AG = the volume of restroom waste per vehicle entering the RRA; and
sag = the volume of TSS waste per vehicle entering the RRA. (For an RRA that is not provided with a TSS, sag = zero.)

Septic Tank Capacity

For domestic-type waste (such as restroom waste) the required volume of an ST is 1.5 times the daily waste flow (25). For TSS waste, the high strength and toxicity were shown to cause up to 20 times as much septage to be generated per unit volume of waste as from domestic waste. To make a TSS ST as safe against overfilling with septage as a domestic ST, the TSS ST volume per unit flow needs to be k $\leq$ 20 times greater than for an ST for domestic waste. That is, the capacity of an RRA ST is

$$V = 1.5\ TS(AG + ksag) \quad (2)$$

Equation 2 makes clear the same ST capacity (1.5 days detention) for waste from restrooms with conventional toilets as for domestic waste; these two wastes contain 170 and 200 mg/L of SS, respectively. No data were obtained on ST capacity requirements for RRAs with restrooms with water-saving toilets. However, because the product of waste volume and waste strength is about the same between water-saving toilets and conventional toilets, it may be reasonable to provide the same ST volume in either case, if sludge storage ultimately limits ST load capacity. For TSS wastes, Equation 2 illustrates an ST capacity of k ($\leq$ 20) times that for domestic waste (up to 30 days detention).

Waste Flow and ST Volume per Mainline Vehicle

The RRA stopping factor is independent of ADT, so restroom and TSS waste flows and ST volume requirements that are expressed per unit of RRA traffic density may also be expressed as multiples of ADT. The mean waste flow per unit ADT from an RRA without a TSS is 0.86 gal, or 0.93 gal from an RRA with a TSS, calculated by substituting into Equation 1 the mean parameters listed in Table 2. Similarly, by Equation 2, the mean required ST volume per unit ADT is 1.3 gal for an RRA without a TSS, or 3.4 gal for an RRA with a TSS.

Probabilistic Design

Waste flows and ST volumes computed above are mean values, so that use of these values for design of a facility involves a 50 percent risk of underdesign. A lower (acceptable) risk of underdesign, p(t), may be specified, for example, p(t)=5 percent. Then design values of waste flow and ST volume can be calculated such that the risk that a field value will exceed the design value is only p(t). For this purpose, design values of waste flow and ST capacity are set higher than their respective means by amounts equal to t standard deviations, where t is the normal variate corresponding to a single-tailed exceedance probability of p(t).

Based on the SDs of the dependent parameters listed in Table 2, the SD of waste flow and of ST volume can be calculated, each expressed in gallons per vehicle of mainline ADT. For waste flow from an RRA without a TSS, these SD values are 0.54 gal/ADT for monthly peaks or 0.86 gal/ADT for weekend peaks, while for an RRA with a TSS, they are 0.57 or 0.91 gal/ADT, respectively. The respective SDs of the volume of an ST at an RRA without a TSS are 0.80 or 1.30 gal/ADT, while with a TSS they are 1.01 or 1.65 gal/ADT for an ST pumped frequently (k=4), or 2.29 or 3.65 gal/ADT for an ST pumped occasionally (k=20).

Graphical Solution of Waste Flow and ST Volume

Figure 3 is a nomograph to aid calculation of the waste flow and the required ST volume for an RRA. Figure 3 provides for (a) projection of the growth of ADT with time, given the annual growth rate and design period; and (b) selection of RRA waste flow and ST volume per unit ADT, given the acceptable risk of underdesign, p(t), and whether a TSS is provided at the RRA.

To project the growth of traffic with time, Figure 3 assumes continuous growth according to

$$ADT = ADT_0 \exp(ni/100) \quad (3)$$

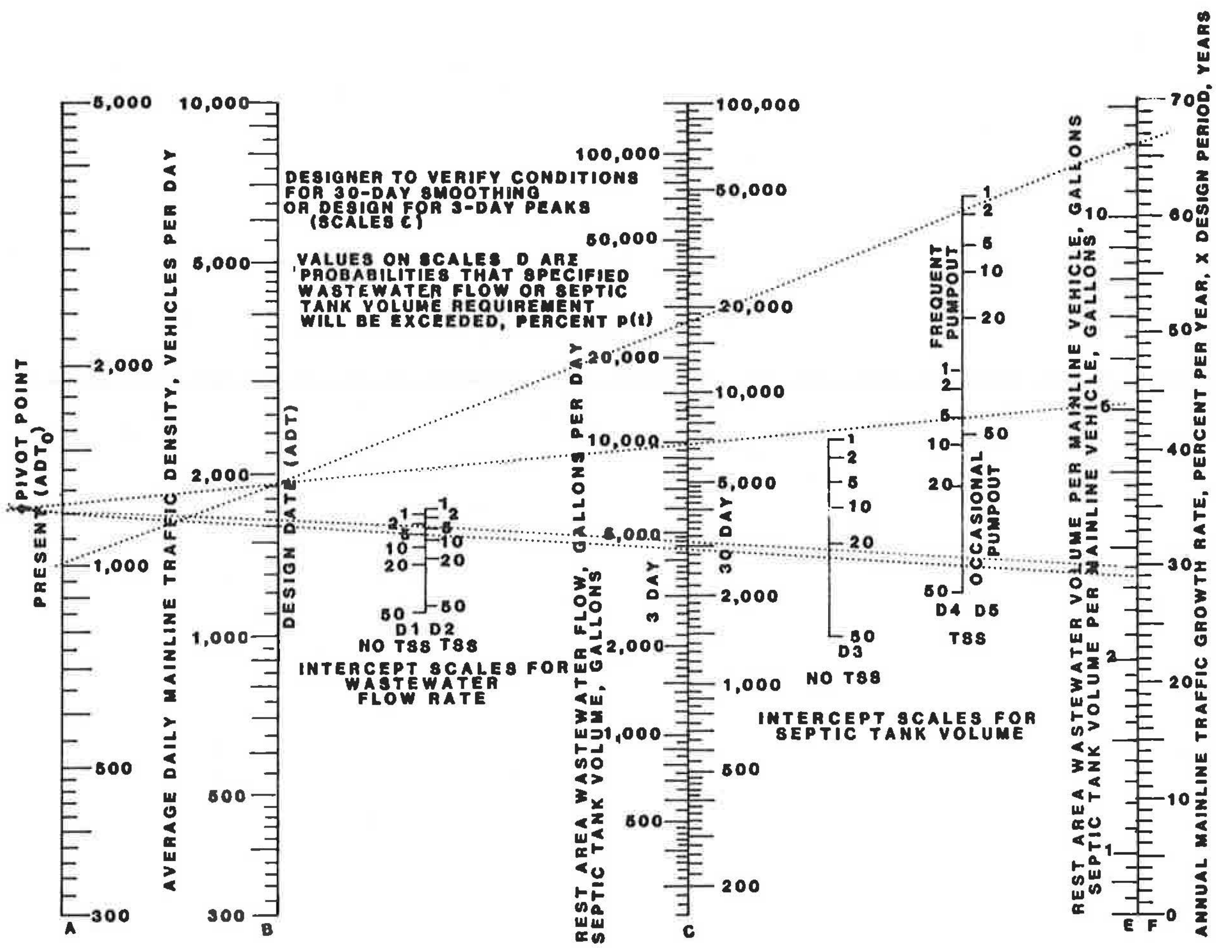

FIGURE 3 Nomograph for estimation of design wastewater flow and septic tank volume for roadside rest area.

where

ADT = average daily traffic density at end of design period;
ADT_O = initial average daily traffic density;
n = design period, in years; and
i = annual growth rate, in percent.

Example lines on Figure 3 illustrate the procedure for obtaining the design waste flow and ST volume. For the purpose of the example, a design period of n = 11 yr and a traffic growth rate (for ADT) of i = 6 percent/yr are assumed. Then ni = 11 x 6 = 66 is entered in Scale F of Figure 3. This Scale F point is joined to ADT_O = 1,000 in Scale A to read ADT = 1,930 in Scale B, the traffic density at the end of the design period.

Assume that:

1. This RRA is to serve one direction of traffic flow,
2. A TSS will be provided,
3. Restrooms will be provided with water-saving toilets,
4. The system hydraulics will provide only for 3-day smoothing, and
5. Septage will be pumped from the ST several times a year (i.e., frequently).

The restroom waste flow and ST capacity are first calculated as for restrooms with conventional toilets, and waste flow is later adjusted for water-saving toilets. A p(t)=5 percent risk of underdesign is acceptable; by joining from the pivot point in Figure 3 through points corresponding to p(t)=5 percent in Scales D1 and D2, RRA waste flows with and without a TSS are read on Scale E as 2.7 and 2.8 gal per ADT, respectively. That is, the TSS contributes 2.8 - 2.7 = 0.1 gal/ADT, and the restroom would contribute 2.7 gal/ADT if it were equipped with conventional toilets. Similarly, by joining from the pivot point through Scale D4, the capacity of the ST to serve restrooms and TSS is read on Scale E as 5.1 gal per ADT.

Water-saving toilets reduce the flow of restroom waste. Given reason to expect continued diligent and skilled maintenance of water-saving toilets, the part of the design waste flow originating from RRA restrooms might be reduced to as little as one-eighth of that calculated for conventional toilets (or for example, one-half with ordinary maintenance). Assume one-fourth for this example. Then the combined flow of waste from restrooms and TSS is 2.7/4 + 0.1 = 0.78 gal/ADT; this multiplies by the 1,430 ADT to obtain the total waste flow as 1,100 gal per day.

By entering the unit ST volume of 7.3 gal/ADT into Scale E and joining to the 1,430 ADT in Scale B, the required ST volume of 7,300 gal may be read from Scale C for 3-day smoothing, regardless of whether conventional or water-saving restroom toilets are installed.

Dimensioning of Septic Tanks

Several guidelines constrain the dimensioning of an ST:

1. Liquid capacity ≥ 1,500 gal;
2. Two compartments, the first about twice as long as the second;
3. Total length ≥ 12 ft;
4. Breadth ≥ 4 ft;
5. Liquid depth ≥ 2.5 ft and ≤ 5 ft;
6. Total depth = 1.2 x liquid depth; and
7. Breadth x 2 ≤ total length ≤ breadth x 3 (1).

Figure 4 shows the design of an ST of given capacity subject to these constraints, showing example lines for a 7,300 gal ST. After entering the 7,300-gal capacity in Scale G of Figure 4, the approximate length of the ST is read directly opposite on Scale H (26 ft), and a similar and convenient ST length is entered in Scale I (25 ft). Then a line is drawn through the 7,300-gal capacity on Scale G through the 25-ft length on Scale I to intercept the turning scale (Scale L), and return through a suitable combination of ST depth and breadth in Scales J and I, 4 ft and 10 ft, respectively. The first cell of the two-celled ST is 17 ft long (approximately 2/3 x 25 ft) x 10 ft wide x 4 ft liquid depth, and the second cell is 8 x 10 x 4 ft. The total depth of each cell is 1.2 x liquid depth = 1.2 x 4 or approximately 5 ft.

Sizing the LF

The LF is designed to (a) store one day's waste flow in the interstices within the loosely packed stone in the LF trenches below the LF pipe invert; and (b) provide a sufficient area of seepage face to accept the incoming waste flow, area being measured on the vertical face of trench walls below the LF pipe invert (1). Seepage rate (waste flow divided by the area of the seepage face) must be limited to avoid hydraulic overload on the LF, depending on the soil percolation value according to

$$S \leq 5P^{-0.5} \tag{4}$$

where S is LF seepage rate, gal/ft^2-day, and P is percolation value of soil, min/in., by the method of Van Kirk et al. (26), and Grottkau et al. (see paper elsewhere in this Record).

Having fixed the volume of stone in the trench such that interstices within the stone will store one day's waste flow, and the vertical area of stone-filled trench walls by Equation 4, the trench width is also fixed. It then remains to select a suitable combination of trench length and trench-depth-below-pipe-invert to provide the trench wall area determined from Equation 4.

A network of trenches is arranged to suit the LF site, preferably minimizing the flow path from the LF inlet to the most remote point. Adjacent trenches are centered not closer than 6 ft apart. A dosing siphon is provided if the total length of the trenches exceeds 500 ft. An independently dosed separate LF is provided for each 1,000 ft (or less) of LF trench length (1).

A nomographic procedure for design of an LF is presented in Figure 5, together with lines showing a worked example for the case: waste flow = 1,100 gal/day; soil percolation value = 12 min/in.; and LF gravel unit weight = 95 lb/ft^3. As shown by Scales O and P in Figure 5, the unit weight for gravel of 95 lb/cf corresponds to a gravel porosity of 0.425,

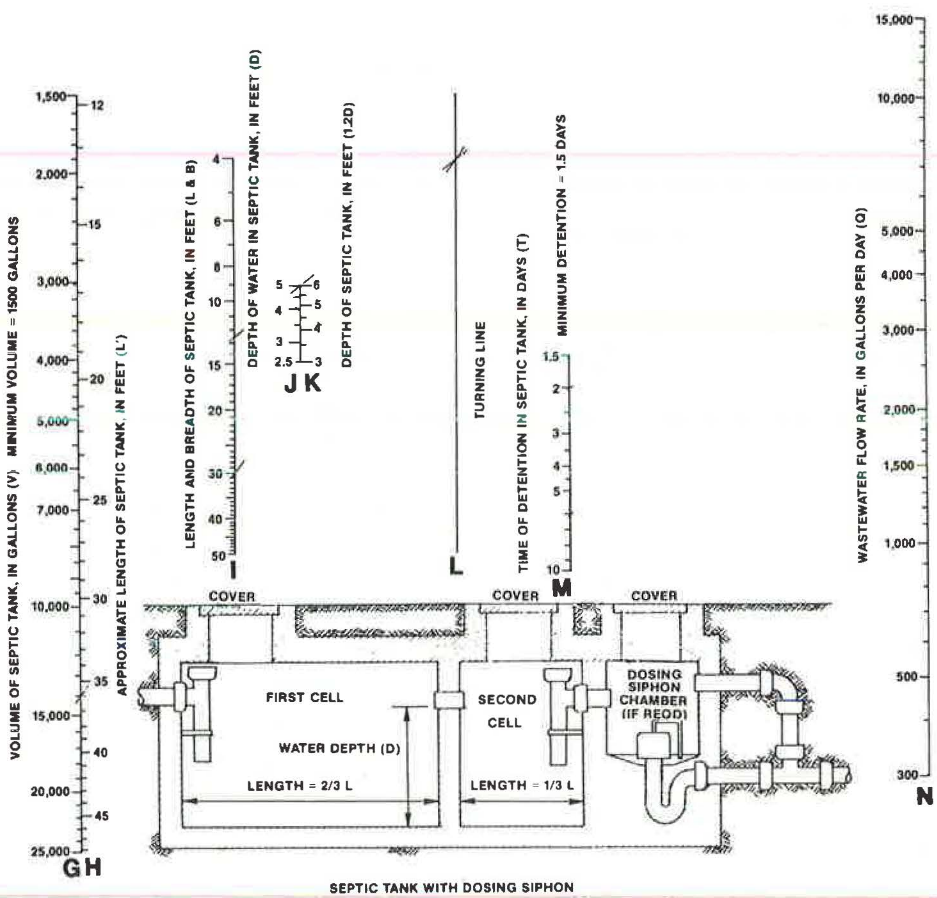

FIGURE 4 Nomograph for computing dimensions of septic tank.

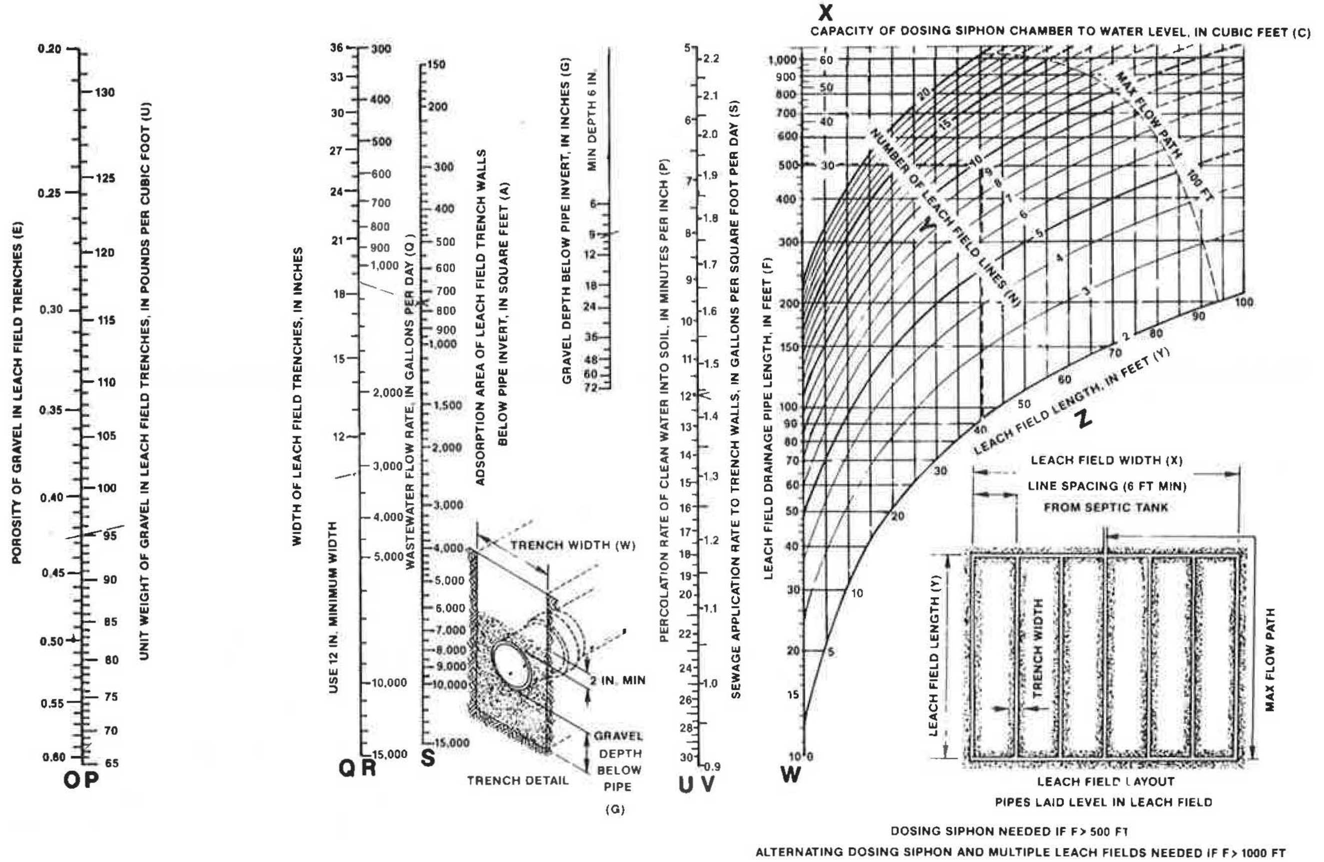

FIGURE 5 Nomograph for computing dimensions of leach field.

based on an assumed specific gravity of 2.65 for the gravel particles. Also, opposite the soil percolation rate of 12 min/in. in Scale U, Scale V shows the allowable rate of application of sewage to the trench walls as 1.44 gal/ft^2-day. The trench width is read on Scale Q by joining the 95-lb/ft^3 gravel density on Scale P to the soil percolation rate of 12 min/in. on Scale U; the resulting width of slightly less than 12 in. is set to 12 in. for easy construction.

Next the soil percolation value of 12 min/in. on Scale U is joined to the waste flow of 1,100 gal/day on Scale R to obtain the Scale S intercept of the required total trench sidewall area below the LF pipe invert, that is, 760 ft^2. This example assumes that the required clearance of the trench floor above groundwater level limits the depth of gravel below the LF pipe invert to 9 in. Then the 760 ft^2 wall area on Scale S is joined through the 9 in. trench-depth-below-pipe-invert on Scale T, show- ing the required length of LF trench on Scale W as 510 ft.

Because this 510-ft LF trench length exceeds 500 ft, a dosing siphon is needed. Opposite the 510-ft trench length on Scale W, the capacity of the dosing siphon is shown as 31 ft^3 on Scale X. Had the trench been longer than 1,000 ft, an alternating dosing siphon would have been necessary to cycle the waste flow between two or more LFs, each LF with 1,000 ft or less of trench.

Finally, dimensions for the network of trenches in the LF are selected. From the 510-ft trench length on Scale W, read horizontally to the particular Scale Y curve that corresponds to the desired number of parallel LF lines, then vertically down to the length of these LF lines on Scale Z. In the example, ten lines are selected on Scale Y, and 40-ft long LF lines are read from Scale Z. The width of the LF is the number of lines, less one, times the line spacing = (10-1) x 6 = 54 ft, for trenches spaced at 6 ft. This completes the ST-LF functional design.

SUMMARY

Twenty-eight of the 95 roadside rest areas in California were studied to obtain data relevant to design of waste disposal facilities for roadside rest areas. More than 20 of the rest areas investigated provided trailer sanitation stations for dumping recreational vehicle waste holding tanks. Data obtained from the rest areas investigated included:

1. Peaking factors (i.e., the ratio of annual peak mainline traffic density to mean annual density);
2. Stopping factors (i.e., the fraction of mainline traffic using roadside rest area facilities);
3. Wastewater production per vehicle entering a rest area; and
4. Wastewater quality characteristics.

The mean ratio of the peak traffic density to average density ranged from 1.24 for the peak month to 1.86 for the peak 3-day holiday weekend in the case of one-way traffic, or up to 1.71 for the peak weekend for two-way traffic. Two-way peaks are lower due to desynchronization of peaks in opposing directions.

A mean of 12 percent of mainline traffic entered roadside rest areas. At those rest areas where a trailer sanitation station was provided, a mean of 2.1 percent of traffic entering the rest areas were recreational vehicles that dumped one or more waste holding tanks at the trailer sanitation station.

A mean of 5.5 gal of restroom wastewater was produced per vehicle entering a rest area where conventional toilets were provided. Two locations with newly installed water-saving toilets produced about one-eighth of this waste volume.

Where trailer sanitation stations were provided, each recreation vehicle dumped a mean of 12 gal of wastewater. In addition, a mean of 9 gal of washdown water from a faucet at the trailer sanitation station were consumed for cleanup after each dump. Roadside rest area restroom wastewater is comparable in strength to domestic wastewater. Trailer sanitation station black water diluted by washdown water contributes 20 times the volume of sludge and scum to a septic tank as does the same number of gallons of domestic wastewater. Consequently, a septic tank for a trailer sanitation station must usually be larger than a septic tank for the same flow of domestic wastewater.

A nomographic procedure estimates loadings on roadside rest area wastewater disposal facilities in probabilistic terms, so that the risk of overloading the facilities may be considered as a design variable. Nomographs are provided for the design of septic tanks and their associated leach fields for waste flows up to 15,000 gal per day.

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Ice Jams at Highways and Bridges–Causes and Remedial Measures

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ABSTRACT

Ice jams cause substantial damage to highways and bridges yearly; however, the highway engineer often is not familiar with what causes them, how to remove or prevent them, or how to lessen their damage-causing potential. Presented in this paper are the basic types of ice and how to recognize them; how they might cause ice jams to form, where they might occur, how to remove them, how to prevent them, and how to compute the height they might reach. With this information the engineer should be aided in his design or reconstruction of highway facilities so that they will not be affected by, or cause, ice jams.

Property owners as well as the general public want a logical answer as well as action toward the elimination of, or at least the reduction of damage from, ice jams.

Highway facilities are usually the most immediate visible means of causing ice jams and often take the brunt of the damage; therefore, the highway agency must be as well prepared as any other federal or state agency to understand ice jams and must be able to take action to combat or prevent them from occurring. The purpose of this paper is to review the three aspects of ice and ice jams highway engineers must understand before they can adequately and successfully deal with ice jam problems. First, the different types of ice must be defined and understood. Different kinds and different forms of ice can create different kinds of problems. Second, the most likely places for ice to jam must be closely identified. Third, several methods of ice jam removal must be considered--some are more effective than others but these are also more expensive. In addition, two theoretical methods for predicting ice jam levels will be presented.

TYPES OF ICE

The U.S. Army Corps of Engineers' Engineering Circular "Ice Engineering" (1) lists approximately 22 different types of ice. The highway engineer is mainly interested in four types:

1. Frazil ice,
2. Ice floes,
3. Ridged ice, and
4. Rotten ice.

One of the most common and most difficult types to combat is frazil ice. Frazil is that ice most likely to be observed in open stretches of river during the early stages of freezing. It occurs or is produced in supercooled water just a few tenths of a degree below freezing and appears like small pieces of flowing slush. Any stream moving at about 2 feet per second (fps) or faster is a potential frazil producer as is a lake or other large body of water that might experience surface turbulence. The problem with frazil is its adhesiveness. As individual pieces collect and increase in size, it becomes more buoyant; yet, as the velocity slows, the potential for a solid ice cover increases. At the upstream edge where an ice cover ends, frazil often begins to adhere to the underside of the cover, causing what is called a hanging dam. As the water tries to pass lower under this dam, more frazil adheres to it and sometimes can, with enough frazil, create a complete ice dam in a river. The potential and often real problem should be obvious. Water will back up, causing higher water surfaces upstream, while lower flows downstream may cause ice grounding or other types of problems. Increased river bed or bank scouring can occur too, as the river tries to force its way through the ever decreasing opening under a hanging dam.

Frazil can also cause problems at hydro dams, water intakes, or at any submerged structure it might adhere to. Intakes or trash racks often become clogged solid with ice--usually frazil--and this buildup can even cause structural failures due to the added dead load. It is difficult to detect a frazil hanging dam due to its solid ice cover, but there are two possible methods. By the coring or sounding method, solid ice is first encountered, then the slush or frazil, and finally, flowing water. Another method of detection is if the ice cover is relatively thin--then the buoyant frazil will push up underneath, causing the sheet to hummock. This can be hard to detect with a snow cover, however.

An extreme example of frazil ice occurred in January 1981 on the Connecticut River near where Vermont, New Hampshire, and the Canadian province of Quebec intersect. Ice jamming caused flooding in the New Hampshire and Vermont towns as well as the raising of ice levels to within one foot of the only highway bridge over the river in the vicinity. The concern was not so much for the bridge itself but for the New Hampshire town's water line that was on the bridge. If the bridge were lost, the water supply would be lost also.

Coring through the ice on the river about a mile downstream revealed a layer of solid ice about 4 feet thick, and under that was another 8 to 10 feet of slush or frazil. Actual free flowing water was only about 2 to 3 feet deep along the bottom of the river. A hanging dam created by frazil ice adhering to the bottom of the solid ice cover was backing up water and thus raising the levels further upstream. The frazil was being generated about 5 miles upstream by turbulent supercooled water flowing over a dam spillway.

An ice jam is usually visualized as large cakes of ice or ice floes, all wedged together into a rough but solid unit. The Corps of Engineers defines a floe as a free floating piece of ice greater than 3 feet in extent (1) (Figure 1). Large floes by themselves are not normally a problem although the force of a floe hitting light objects such as small boat piers can cause extensive damage. The presence of large numbers of ice floes is an indication that, at ice jamming-susceptible locations, a jam is a distinct possibility.

FIGURE 1 Ice floes.

Ridged ice may signify the beginning of a jam and is often mistaken for one. The terms "ridged" or "rubble" ice refer to ice floes piled or scattered haphazardly, one on top of the other, forming ridges, walls, or rough humps, although water continues to flow underneath it (Figure 2).

FIGURE 2 Ridged ice.

Rotten ice is ice in an advanced stage of disintegration. It usually contains pockets of water, may show cracks or potholes, and generally appears unstable (Figure 3). This ice may also be separated from the shore line.

FIGURE 3 Rotten ice.

Where Ice Jams Occur

An ice jam is the accumulation of frazil, ice floes, or ridged ice, wedged together thickly enough to restrict the flow of water such that a head differential is caused. Highway engineers are responsible for prevention of this or keeping the situation as minimal as possible. One aid to doing this is knowing, or being able to predict, the locations of ice jams. There are four sections of any river where jams are most likely to form, with the fourth really being a combination of the first three.

The first is a section of river where the slope or gradient decreases (Figure 4). The flatter gradient section will have a slower velocity, will freeze sooner, and will have a thicker ice cover. When ice begins to break, this area will be the last to do so and as a result, ice floes moving downstream will catch at the upper end of the ice cover. The upstream end of a lake or reservoir or the mouth of a river are all excellent places for this to happen.

Another possible jam site is at a stream constriction. The constriction can be natural, such as a canyon or other narrowing, or manmade, such as a

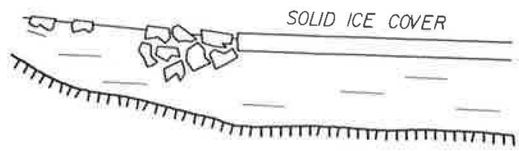

FIGURE 4 Change in gradient.

bridge (Figure 5). A short bridge, with abutments in the channel, or with piers not aligned with the flow, is an excellent ice jammer.

The third location is a shallow reach. Here, ice can either freeze to the bottom or ice floes can ground out as they move through the reach (Figure 6). Sections of rapids or wide, gravel bar areas of rivers are examples of such possibilities.

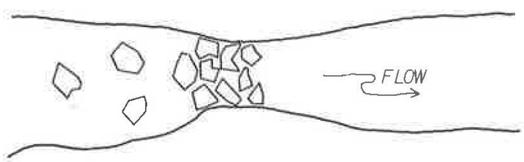

FIGURE 5 Constriction.

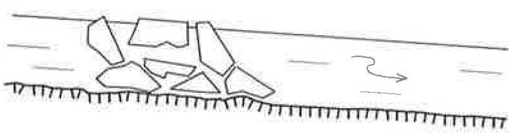

FIGURE 6 Shallow reach.

The fourth location, which may fit into any one of the first three, is at sharp bends. Here, ice builds up on the inside, where the velocity is less, creating a channel narrowing or constriction (Figure 7). Another possibility is that ice grounds on the inside due to the shallowing effect, also with constriction occurring.

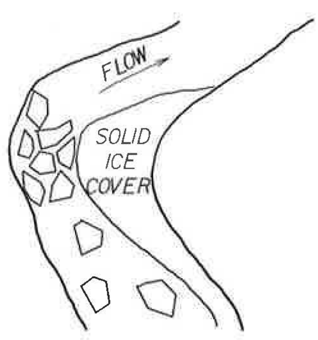

FIGURE 7 Jamming on a bend.

When the jam begins, regardless of the initial cause, water starts backing up, flattening the energy gradient, slowing the upstream velocity, and causing the jam to move even farther upstream. When this happens, flooding occurs rapidly, usually faster than during a non-ice jam flood because a regular flood increases in level based on the supply of water that is related most directly to rainfall--an important variable. During an ice jam flood, the flow rate usually does not change substantially, and the ice is in constant supply. The channel size, however, decreases (as does the capacity to convey water) and, as it decreases, the water surface rises.

Removal of Ice Jams

It is at this point that the highway agency becomes involved. An ice jam has occurred and is causing flooding of a highway or is damaging property and it needs to be removed. It should first be realized that the best time to remove an ice jam is when the water is high. There are two reasons for this: (a) water pressure is high and will help to push the ice away; and (b) the depth and flow rate will be great enough to carry the ice. If the jam is not moved at this time, then there is a chance, especially during mid-winter, that the flow will drop and so will the ice, settling on the bottom and creating a solid dam. Water may later flow over it and freeze, or else a new ice cover will be created upstream, and when that breaks, this jam will catch even more. The Corps of Engineers suggests four techniques to use in breaking a jam: mechanical removal, dusting, blasting, and using icebreaker ships. Because icebreakers are not normally used for breaking up river ice jams near bridges, nor in conjunction with highway flooding, this technique will not be discussed here.

Mechanical removal is probably the easiest and most effective, but it is also the most costly due to equipment rental costs. Bulldozers, backhoes, or draglines are used to remove the ice and create an open channel for the water. Usually the ice can be deposited on the shore but caution should be emphasized. This method was used on one river in Vermont and the owner of the property on which the ice was dumped sued the removing agency because the ice took so long to melt. It not only denied him early spring use of a portion of his field, but the material that was left when the ice did finally melt was not conducive to fertile agricultural soils.

Dusting is a relatively new method not often used, but it can be very effective. Any dark material, usually a black granite or similar type, is spread on the ice in ground-up dust-size pieces to absorb solar radiation and thus speed up the thawing process. This method is normally used before a jam begins because the rough surfaces of a wedged jam create shadows that hide much of the dust. For the same reason, dusting cannot be used if there is a thick snow cover on the ice; nor can it be used in mid-winter because the sun is not high enough in the sky. Dusting is normally done by either a helicopter or a crop duster type of aircraft. Another problem with dusting can be environmental concerns with a material such as ash or granite eventually dropping into the stream. Dusting can also be expensive.

The most popular method is blasting. Blasting is done to loosen the ice and break it into smaller pieces so that it will begin flowing as cakes again. The first consideration before blasting should be whether there is enough flow to float the ice, and then whether there is an open stretch of water downstream where the ice can float without causing problems.

An important consideration in removing a jam is not so much which method to use, but whether any method is really necessary. And if so, will it work? Quite often ice jams will break up by themselves--much of the time without causing substantial damage. Therefore, any knowledge of the performance of past jams at the site and what happened when they did break up would be useful.

There are other methods of combatting ice jams besides removal, and these consist of preventing the jam from causing any real damage. These methods include the construction of levees, both temporary and permanent to contain the higher water elevations within a certain area. By controlling the rate of water releases from a dam, a river can be raised or lowered to either carry the jam or to prevent additional ice from forming. Channels can be cleaned, straightened, narrowed, or relocated, to aid in the prevention, or lessening, of the effects of a jam. Ice booms or dams can be constructed to create the jam in a less harmful area, and finally the area can be evacuated, either temporarily or permanently.

Prediction of Ice Levels

There are two theoretical methods that may be useful in predicting how high ice jams may raise water levels. The first has been developed by the Corps of Engineers Cold Regions Research and Engineering Laboratory. This is a method used to compute flow profiles in a river covered with ice (1). In simple terms, it is the standard step method with a roof of ice over the channel. A roughness coefficient for the underside of the ice must be known or assumed, and this is combined with the channel roughness to give a composite roughness factor (Figure 8). The Cold Regions Laboratory reports that values of n for ice range from less than 0.01 to 0.045. At the downstream edge of a new and rough jam, the roughness value may be as high as 0.07, though this more likely reflects a drag coefficient. From this point on, the normal standard step methods of computing a flow profile are used. This method has been incorporated into the Hydrologic Engineering Center's Computer Program HEC-2, "Water Surface Profiles" as an optional computational sequence.

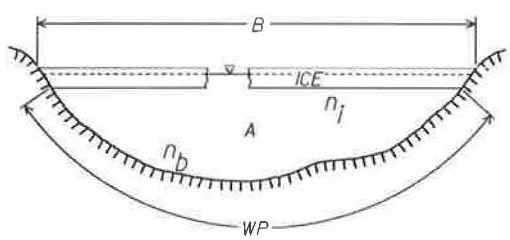

FIGURE 8 An ice-covered stream.

Why is the standard step method that reflects open channel flow used? It appears that an ice-covered stream should have a pressure flow. Actually, an ice cover usually floats and is not solid. There are expansion and contraction cracks, as well as cracks from changes in the water level so the flow really is not under pressure.

Another method, developed by an engineering firm engaged in flood insurance studies (2), may offer possibilities for use by highway engineers in predicting flood levels at bridges or along stretches of highway. The method consists of determining a stage-frequency relationship for ice jam events based on known or calculated high water-high ice levels at the location under study. Then a free flowing stage-frequency relationship is calculated and by the laws of probability of a free-flowing and ice jam event occurring simultaneously, a combined stage-frequency relationship is calculated.

If a site creates ice jams severe enough to raise concerns or complaints, there should be enough people in the area able to give one or more ice or water marks with the years they occurred. With several marks, the winter stage-frequency curve may be plotted by using one of the generally accepted methods of graphing probability distributions.

A better method may be to calculate the stage of a static ice jam based on an equation developed by Michel (3).

$$Y \geq 4.6\ (n\ Q_{max})^{0.46}/B^{0.23} \tag{1}$$

where

Y = average water depth, feet;
B = width of prismatic channel, feet;
Q_{max} = maximum channel discharge, ft³ per second (cfs); and
n = equivalent Manning's roughness coefficient.

$$\left[n_1^{3/2} + n_2^{3/2}/2\right]^{2/3} \tag{2}$$

where

n_1 = roughness coefficient of the ice cover; and
n_2 = roughness coefficient of the channel bed and banks.

Q_{max} is the discharge at various frequencies based on winter peak discharges, usually December 1 to April 1.

The distribution of winter peaks is normally calculated by the Log Pearson Type III method. By using Michel's equation and inserting the highest observed stage over a known period and solving for Q_{max}, a verification with the ice jam discharge-frequency analysis can be made. (Examples of ice jam discharge versus frequency and ice jam stage versus frequency are shown in Figures 9 and 10.)

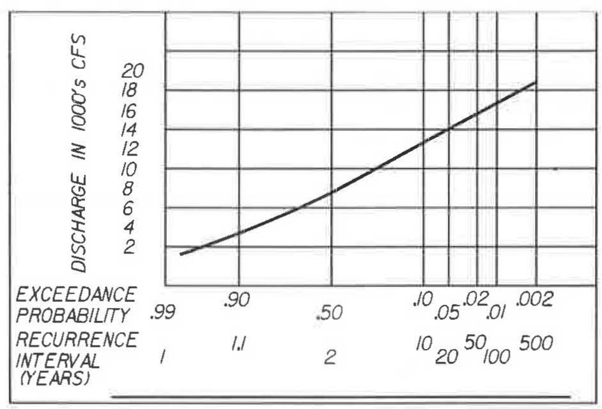

FIGURE 9 Ice jam discharge versus frequency.

The free flowing stage-frequency relationship is simpler to determine. First, the discharge frequency distribution is found by Log Pearson Type III using a record of annual peak discharges. Stage is then calculated by any backwater or flow profile method.

The combined relationship is determined from the laws of probability, which state the probability of the union of two independent events, such as an ice jam (Y_I) and a free flowing event (Y_F), is:

$$P(Y_I \cup Y_F) = P(Y_I) + P(Y_F) - P(Y_I) \cdot P(Y_F) \tag{3}$$

The results of this are shown in Figure 11.

The highway engineer now has a stage-frequency curve considering ice jams at the site under study.

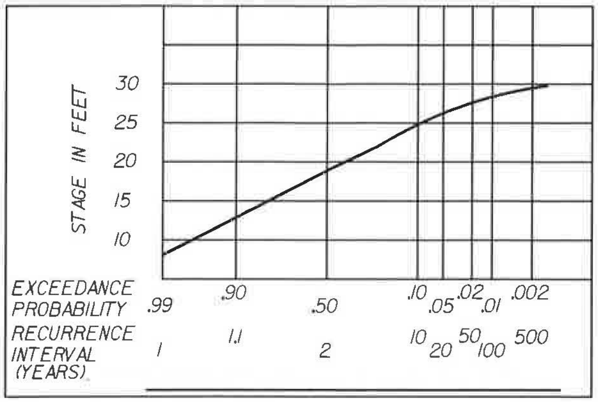

FIGURE 10 Ice jam stage versus frequency.

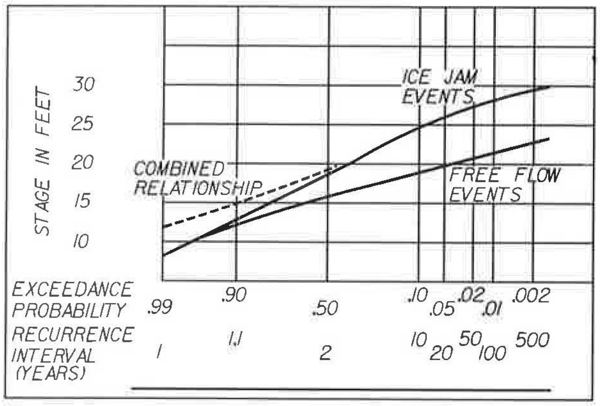

FIGURE 11 Combined stage versus frequency.

With this he can decide heights of bridges or highways necessary to prevent problems from future ice jams.

In order to use this method a record of both annual and winter peak discharges (or the ability to generate one) is required (2).

The Use of Ice-Jam Information

Now that this material is available to the highway engineer or hydraulic specialist, how can it be applied in the areas where ice jams most often affect highways and bridges?

Jamming at bridges is the most common and most critical area of occurrence. Here, ice jams can endanger the bridge as well as cause upstream flooding (Figure 12). Often the bridge is not the cause of the jamming, therefore altering the structure will not reduce the upstream flooding. The danger to the bridge can be reduced, however, by lengthening the span or by raising the superstructure. If the jamming initially begins somewhere else along the stream, the ice levels will remain the same at the bridge so lengthening the span will not help. Raising the superstructure, therefore, usually appears to be the best solution. The use of the latter method would be recommended to predict elevations of ice.

Sometimes the bridge may be the cause of the jamming and the subsequent upstream high water. If this is the case, then the Corps of Engineers' method using different channel sizes may be a better method to use in determining the ideal bridge size. Either streamlining piers or eliminating them completely may help in reducing jamming. A relocation

FIGURE 12 Ice jams and bridges.

to a crossing site less characteristic of the common causes of ice jams may also be a possibility.

In designing new bridges, the design should be checked to determine if it will create one or more of the potential jamming sites mentioned earlier. Ice jamming at locations away from bridges but where the roadway is flooded is also relatively common (Figure 13). Unless the roadway has been reconstructed with side slopes projecting out into the stream creating a constriction, the roadway will not be the cause of the jamming. As with bridges, the best way to solve this problem is to raise the roadway grade. The use of the flow profile method with an ice cover is recommended in determining ice elevations. It must be remembered, however, that this method assumes a solid or static ice cover. It does not take into account pushing or humping, which may increase the actual ice levels observed. An added increase in height over that calculated may need to be made to the new roadway elevation.

FIGURE 13 Ice jams and highways.

Raising of the roadway elevation may redirect ice and water flows, causing flooding elsewhere, or higher elevations upstream. If a portion of the floodplain is cut off by this elevation increase, then another solution may have to be found.

Culverts projecting from roadway fills may snag ice floes, either causing a jam to occur, or, as a minimum, twisting or otherwise damaging the culvert end. Care must be taken in locating culvert outlets directly into streams where ice may be a problem. If outlets cannot be avoided, they should at least be angled downstream so as not to present a sharp obstacle for ice. If ice elevations can be determined, they may be able to be placed above the projected ice elevations.

SUMMARY

Ice jams have created, and will continue to create, problems for the highway engineer. Flooding of roadways and the damaging or destroying of bridges are the most common effects of ice jams, but increased bank erosion and upstream flooding caused by water backup from jams at bridges are also frequent occurrences.

In order to effectively combat ice jams, the highway engineer must first know where they might occur. Ice jams occur generally where: (a) river slope or gradient flattens; (b) stream constricts, either naturally or artificially; (c) stream depth lessens, allowing ice to bottom out; and (d) river makes a sharp bend. These sections may occur naturally, or be man-made. To minimize the chance of ice jamming, they should be avoided.

The engineer must also be able to recognize the different types of ice and what their appearance indicates. Ice forms in different ways, and therefore creates different types of jams, as follows:

1. Frazil ice or slush ice usually adheres to the bottom of solid ice covers and creates a hanging dam. These are difficult to detect and can form quickly.
2. Ice floes are easy to see and are the most common type of ice. If blocked, they continue to pile up against one another and can exert high pressures against structures.
3. Ridged ice looks like a jam, but may not create a problem. Usually water is flowing freely under the ridge with very little increase in water elevation.
4. Rotten ice is in the last stage of ice. It is decaying or disintegrating.

Once a jam forms, it may have to be removed. Although no method is completely effective, removal can help for a while. Some methods for this follow:

1. Mechanical removal is probably the most effective, but it is expensive.
2. Dusting is a relatively new, complicated, expensive method. It does work provided climatic conditions cooperate.
3. Blasting is the most common method, but it is not overly effective.
4. Ice breakers can only be used on large bodies of water where navigation needs are foremost.

Finally, the engineer must also be able to predict how high water elevations may reach during different recurrence interval ice-related events. With this information, it may be possible to raise highways and bridges above the levels of the appropriate ice events.

Methods for predicting ice levels, however, are approximate and usable only in certain cases. The derivation of a frequency-stage curve requires winter peak flows and historical ice level data. Using a stream profile or step method assumes a solid or static ice cover, which, for an ice jam event, is not usually the case.

With this knowledge, the highway engineer should be better prepared to understand ice jams, to help lessen their destructive effects when they do occur, and to design facilities that will not be affected by, or cause, ice jams.

ACKNOWLEDGMENTS

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Iowa Design Manual for Low Water Stream Crossings

RONALD L. ROSSMILLER

ABSTRACT

Most counties have bridges that are no longer adequate and, therefore, are faced with a large capital expenditure if the same type replacement structure is proposed. Because a low water stream crossing (LWSC) may be an attractive low-cost alternative to replacing a costly bridge, a manual has been developed to design LWSCs for use in Iowa. The purpose of the manual is to provide consistent guidelines for county engineers and consultants designing these crossings. An LWSC is defined as an unvented ford, a vented ford (one having some number of pipes), a low water bridge, or other structure that is designed so that its hydraulic capacity will be insufficient one or more times during a year of normal rainfall. The use of unvented fords is discouraged in Iowa, and locations where vented fords are permissible have been narrowly defined. Because local social, economic, and political conditions vary from county to county, no hard and fast rules have been set down as to where LWSCs can be used; nevertheless, once the decision to use an LWSC has been made, the manual contains a simple design procedure for these crossings. This procedure includes the following phases: hydrology, hydraulics, roadway geometrics, and material selection. Discharges are estimated from equations that include drainage area, return period, and flow duration. Three methods are included to select the material used to protect the crossing from washing out, the first two of which are based on geomorphic relationships developed from Iowa stream gauging station records.

Most counties have bridges that are no longer adequate and, therefore, are faced with a large capital expenditure if the same size replacement structure is proposed. A low water stream crossing (LWSC) may be an attractive low cost alternative to replacing a costly bridge. The ideal situation would be to close the road but this alternative is not always available. However, if loss of access for a short time is not a problem, the site may be a candidate for an LWSC. In Iowa locations where LWSCs would be permitted have been narrowly defined.

One example would be on a primitive road serving only as a field access for local farmers. During good weather conditions, a well-designed vented ford would provide adequate facilities for any traffic using the road. During periods of significant rainfall, because the primitive or unpaved road is not passable except by farm equipment or four-wheel drive vehicles, the closing of the flooded LWSC is not a problem to the traveling public.

However, not all obsolete bridges are on primitive roads serving only as a field access. Other potential locations for LWSCs that may tolerate a short loss of access are those that have no:

- Residences with sole access over the LWSC,
- Critical school bus route,
- Recreation use, or
- Critical mail route.

If these uses do exist, the road may still be a potential candidate for an LWSC if an alternate route is available.

A survey of LWSC use in the United States by Carstens (1) indicated that 61 percent of the respondents used LWSCs only on unpaved roads. Because paved highways have geometric design and traffic control conducive to higher speeds, drivers' expectations are not consistent with the vertical profile encountered at LWSCs. Also, because unpaved roads are limited to low traffic volumes, the use of LWSCs on these roads would involve a lower exposure to traffic.

DEFINITION AND PURPOSE

An LWSC is a stream crossing that will be flooded periodically and closed to traffic. Carstens (1) had defined an LWSC as "a ford, vented ford (one having some number of culvert pipes), low water bridge, or other structure that is designed so that its hydraulic capacity will be insufficient one or more times during a year of normal rainfall."

The purpose of the Iowa manual (2) is to provide design guidelines for LWSCs, after it has been determined that an LWSC is applicable at a certain location. Because conditions vary from county to county, rigid criteria for determining the applicability of an LWSC to a given site are not established nor is a "cook-book" procedure for designing an LWSC presented.

COMPONENTS

An LWSC consists of several components: core material(s); foreslope surface; roadway surface; pipes (if it is a vented ford); and cutoff walls or riprap for protection against stream erosion. The core can consist of earth, sand, gravel, riprap, concrete, or a combination of these materials. Erosion protection for the foreslopes can consist of turf, riprap, soil cement, gabions, or concrete. The roadway surface can be composed of similar materials with the provision that a suitable riding surface be provided. The cost and availability of these materials vary from county to county; therefore, the exact composition of the core and surfacing will depend on local conditions. Pipes can be circular, oval, or arch and made of concrete, corrugated metal, or polyvinylchloride (PVC).

Protection against stream erosion can be provided by either cutoff walls or by armoring the stream bed. Cutoff walls can be constructed of either concrete or steel. The armoring could be riprap or gabions. Again, whether steel, concrete, or rock is used will depend on local cost and availability of materials and machinery such as pile drivers.

DESIGN CONCEPTS AND CRITERIA

The following criteria and design steps are unique to Iowa conditions and concepts as to what constitutes a well-designed LWSC. Much of this may be applicable to other states as well but each item should be construed as only a guideline because each site is unique and each county has its own unique set of conditions.

General Criteria

1. Based on the study by Carstens (1), with the adoption of the recommended regulatory sign and support resolution, the road will be closed when water is flowing across it. Because of this, for vented fords the headwater elevation for the selected overtopping frequency and estimated discharge must be at, or slightly below, the low point in the roadway.
2. This overtopping discharge is based on the concept that the crossing will be closed a certain percent of the time. Because each site is unique and the decision on overtopping duration must be based on the existing physical, social, economic, and political factors present for that site and county, only general guidelines are given for the allowable overtopping frequency.
3. The assumption is made that the existing channel cross section is not altered; that is, its width is not increased so that more pipes can be laid in the widened channel. However, the channel banks could be cut down to allow for proper approach grades.
4. The minimum depth of cover over the pipes in a vented ford is 1 foot.
5. Road grades, vertical curve lengths, and rideability reflect the low speeds allowed on these roads.
6. Flows overtopping the crossing should be controlled to minimize erosion so that damage is low and repair is easy. This can be done by keeping the difference between the upstream and downstream water surfaces to a minimum. One way to achieve this is to keep the difference between the low point in the roadway and the stream bed to a minimum.
7. Because alternative types of materials can be used in the construction of an LWSC, the availability and cost of these materials in different counties could lead to different decisions between these counties.
8. Based on the study by Carstens (1), suitable signing reduces the liability.
9. The type of material used to protect the LWSC from erosion could be influenced by the size and location of the county's maintenance force and the number of LWSCs in the county. Some crossings may need to be inspected after a flood event for needed maintenance. This maintenance could range from sediment and debris removal to major repairs. The time lapse between the flood event and the road being reopened could be excessive if the number of LWSCs requiring significant maintenance is large and the maintenance force is small and located some distance away. How long a period of time is excessive is dependent on the site and the county's social and political climate.

Steps in Design

The general steps involved in the design of an LWSC are discussed briefly in the following paragraphs. The location in Iowa is needed to determine in which hydrologic region the LWSC is located. The watershed size is measured in square miles. These two items are used to estimate discharges and to select crossing materials.

Most LWSCs will be vented fords. Because of the safety problems of driving through water, unvented fords could be closed much of the time and should be used only on those intermittent streams that are dry for the percent of time compatible with the uses of the road.

The allowable overtopping duration is a function of the several items discussed earlier. Because each site is unique, the decision on the duration of overtopping must be based on the existing physical, social, economic, and political factors for that site and county. After this decision is made, the overtopping discharge then can be estimated using equations developed by the U.S. Geological Survey (USGS) for Iowa.

Using the overtopping discharge and the criteria listed in the previous section, the number and size of pipes as well as the headwater depth can be determined from Herr and Bossy (3), commonly known as HEC-5 or Bulletin 5. The pipe can be circular, oval, or arch and made of concrete, corrugated metal, or PVC. Each of these pipe shapes and materials is analyzed using HEC-5 under both inlet and outlet control.

The crossing grades and elevations are a function of the overtopping discharge headwater depth and the physical characteristics of the existing channel and roadway. For vented fords, the low point in the roadway should be in the range of 2 to 6 ft above the stream bed, depending on the size of pipes, depth of cover over the pipes, roadway and surfacing material used, and depth of channel.

Two criteria must be met: (a) the headwater depth for the number and size of pipes selected must be at or slightly below the low point in the roadway and (b) the grades and length of the crest and sag vertical curves must meet the stopping sight distance criterion. The possibility exists that in order to meet criterion b, the low point in the roadway has to be raised above the elevation needed for either the calculated headwater depth or minimum cover criteria. In this case, the possibility exists that the number and size of pipes could be reduced.

Material selection for the crossing foreslopes and roadway surface is a function of the channel velocity and tractive force. High flows (Q_{10} to Q_{50}) will usually govern except for large differences between headwater and tailwater depth when the velocity of the overtopping discharge ($Q_{50\%}$ to $Q_{1\%}$) plunging down the downstream foreslope could be the governing case. These materials can range from turf to concrete.

Other considerations include provisions to protect against stream erosion and seepage. This could consist of steel or concrete cutoff walls or riprap blankets.

DESIGN OF A VENTED FORD

Step 1. Region and Drainage Area

Figure 1 shows the locations of the three hydrologic regions in Iowa. For smaller watersheds, the drainage area can be determined from a 7.5- or 15-minute quadrangle map. For watersheds larger than 5 square miles, Bulletin No. 7 by Larimer (4) can be used to determine the drainage area in Iowa.

Step 2. Flow-Duration Estimates

A flow-duration curve indicates the percent of time within a certain period in which given rates of flow

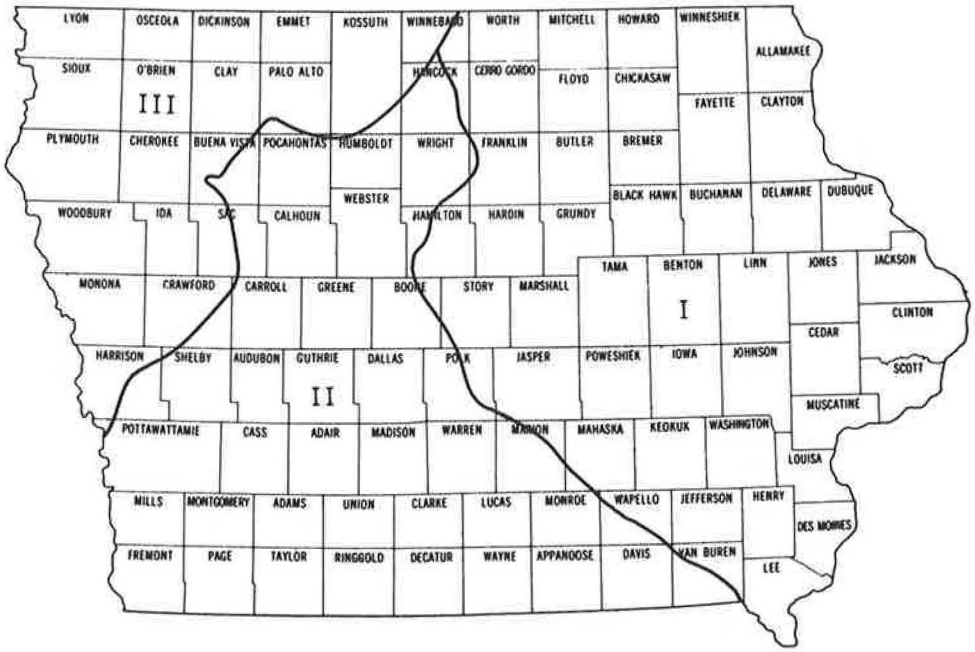

FIGURE 1 Hydrologic regions for duration of discharge equations.

were equaled or exceeded. Flow-duration data for daily flows collected at all the gauging stations in Iowa can be found in Lara (5). These data are used to prepare flow-duration curves at the gauging stations. More frequently, flow-duration information is needed at stream crossings where no recorded data are available. The following procedure can be used to estimate flow-duration information for ungauged sites:

1. Using the map in Figure 1, identify the hydrologic region where the project site is located.
2. Determine the size of the drainage area at the site in square miles.
3. Select a value of e, based on site and county conditions, and the corresponding regression coefficients from Table 1, then solve the following equation.

$$Q_e = aA^b \tag{1}$$

where

Q = discharge in ft^3 per second (cfs),
e = exceedance probability in percent,
A = drainage area in square miles, and
a and b = regression coefficients. (Values of a and b are listed in Table 1 for each hydrologic region shown in Figure 1.)

TABLE 1 Regional Regression Coefficients for Estimating Duration of Flows Having the Indicated Exceedance Probability

Exceedance Probability e, %	Region I a	Region I b	Region II a	Region II b	Region III a	Region III b
50	0.17	1.05	0.06	1.09	0.015	1.24
25	0.52	1.01	0.24	1.06	0.04	1.25
10	1.37	0.98	0.91	1.00	0.15	1.19
5	2.58	0.96	2.26	0.95	0.33	1.15
2	6.78	0.90	6.78	0.90	1.23	1.06
1	13.50	0.85	13.50	0.85	3.56	0.96

Equation 1 and Table 1 are the results of regression analyses performed on the data contained in Lara (5).

Using this equation with Table 1 yields the following results for a 6-square mile watershed in Dallas County, Iowa.

$$Q_{25\%} = 0.24(6)^{1.06} = 1.6 \text{ cfs} \tag{2}$$

$$Q_{2\%} = 6.78(6)^{0.90} = 34.0 \text{ cfs} \tag{3}$$

These discharges are interpreted as follows. If the LWSC is designed for $Q_{25\%}$, the crossing will be closed an average of 3 months each year. If the LWSC is designed for $Q_{2\%}$, the crossing will be closed an average of 7 days each year. Similar equations for other states could be derived using the same methodology employed by the USGS in Iowa.

Step 3. Stage-Discharge Curves

A stage-discharge curve for a channel section is developed by assuming increasing values of depth, determining the discharges by multiplying the cross-sectional area of flow at each depth by the average velocity of flow obtained from Manning's equation at each depth, then plotting depth versus discharge with depth as the ordinate.

The channel cross section and slope (low water surface profile) at the site are measured in the field. Field observations also are made to allow estimation of the roughness coefficient. Calculations for area and wetted perimeter are made by plotting the channel cross section as a series of straight lines, then using simple geometric shapes.

Step 4. Number and Size of Pipes

Determining the number and size of pipes for a particular site is a trial and error process. Several items must be kept in mind:

1. The total width of pipes, including the spaces between them, must be less than the width of the existing channel;

2. The headwater depth controls the low point in the roadway;

3. The pipes can operate under either inlet or outlet control;

4. Pipe lengths are short, but differences in friction losses due to pipe material can still be significant;

5. A large difference between the low point in the roadway and the downstream water surface increases the erosion potential on the downstream foreslope; and

6. A large difference between the low point in the roadway and the stream bed increases the volume of material needed in the crossing and thus increases its cost.

The trial and error process begins by determining headwater depths for the estimated overtopping discharge and assumed combinations of pipe material, number, and size operating under inlet control. The results are reviewed in light of the preceding items and the several combinations reduced to the few best alternatives. These alternatives are checked for outlet control, using the stage-discharge curve developed in the previous step, and the final type, size, and number of pipes selected. These headwater depths for both inlet and outlet control are determined from charts contained in Herr and Bossy (3).

ROADWAY GEOMETRICS

Crossing Profile

General Concepts

Low water stream crossings are designed for occasional overtopping with floodwater and, consequently, have an inherent vertical dip characteristic. This sudden dip in the vertical alignment is not consistent with drivers' expectations of a public highway profile. Proper signing is essential to alert the driver to a condition that cannot be traversed at the higher speeds associated with tangent alignments and flat grades.

The variables of concern in the design of the stream crossing road profile are the tangent grades, the length of sag vertical curve, and crest vertical curve lengths at the stream edges.

Selecting Tangent Grades

The selection of tangent grade lines will be dependent on the height of the stream banks and the slope of the terrain adjacent to the stream banks, as well as the amount of cut allowed into the stream bank. If minimal grading is desired, steep grades will result. However, steep grades significantly increase the stopping distance. In general, a grade of 12 percent could provide a surface suitable for driving when wet and muddy, but only at very low speeds.

The use of flat grades that cause a cutback into the stream bank can result in a maintenance problem. When high water causes overtopping of the crossing, the flood water spreads onto these flat approach grades wider than the normal stream width, and subsequently deposits debris and mud on the crossing roadway.

Selecting the Length of Vertical Curves

A number of criteria are recognized in the design of a crossing profile. Stopping sight distance is the usual criterion for selecting the length of crest vertical curves, whereas headlight sight distance, driver comfort, and appearance may be used for sag vertical curve length determination.

Because of the reduced speed conditions and the inherent short space for crest vertical curves at the stream banks, the normal stopping sight distance criterion for selecting a length of vertical curve is the controlling factor, rather than comfortable ride. Stopping sight distance is applicable on the approaches, especially if obstructions in the horizontal alignment occur, which would restrict the view of the crossing.

Table 2 has been prepared based on the 1965 AASHO stopping sight distance formula. The coefficient of friction was assumed to be 0.20 due to slick conditions on unpaved roads and the grade was assumed to be 10 percent. These distances were then used in LWSC vertical curve calculations.

TABLE 2 Stopping Sight Distances for LWSCs

Vehicle, mph	Perception and Brake Reaction Distance, ft	Braking Distance, ft	Stopping Distance, ft
5	18.4	8.3	27
10	36.8	33.3	70
15	55.1	75.0	130
20	73.5	133.3	210
25	91.8	208.3	300
30	110.3	300.0	410

Crest Vertical Curves

Minimum crest vertical curve lengths were determined using a height of eye of 3.5 ft and a height of object of 6 in. For a given algebraic difference in grades, A, and a vertical curve length, L, selected to fit the terrain, designers generally use the reciprocal of the rate of change of grade, or $K = L/A$, as a measure of curvature in determining speeds for a given crest vertical curve design.

A common procedure for determining minimum length of crest vertical curves is to plot A and L for various speeds. Figure 2 is a design chart for selecting a length of LWSC crest vertical curve, or conversely, having selected a suitable length of vertical curve to fit the terrain, Figure 2 may be used to determine the speed for that design. The minimum vertical curve lengths in Figure 2 are based on a value of three times the speed in feet per second.

Sag Vertical Curves

In the design of a sag vertical curve for normal street and highway design practice, the concept of headlight sight distance determines the length of vertical curve. A suitable length of sag vertical curve allows the roadway ahead to be illuminated so that a vehicle could stop in accordance with the stopping sight distance criteria. For safety reasons, the light beam distance is set equal to the safe stopping distance.

Figure 3 shows the sag vertical curve design chart. It may be used to select the length of sag vertical curve for a specific set of grades and speed condition, or having selected a trial sag vertical curve, the speed associated with that design may be determined. The minimum values in Figure 3 are based on three times the speed in feet per second.

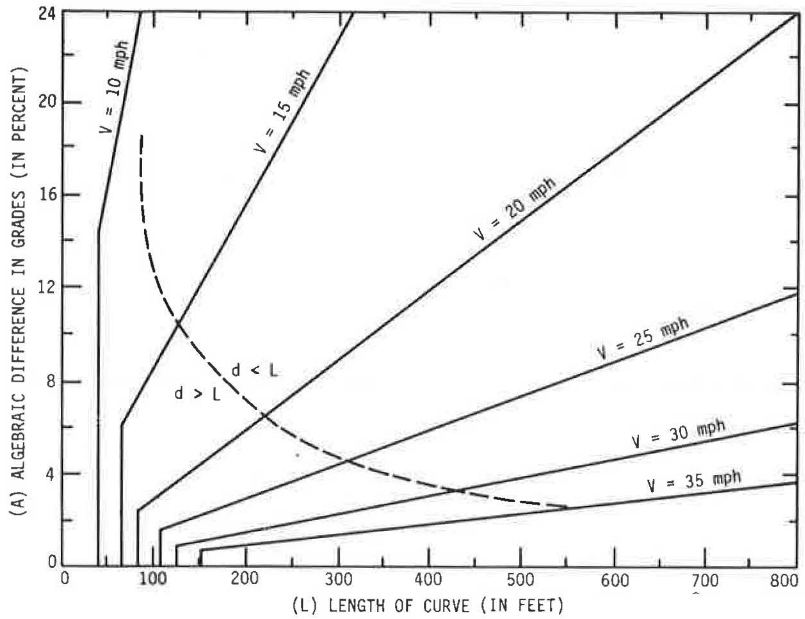

FIGURE 2 Minimum length of crest vertical curve for LWSCs.

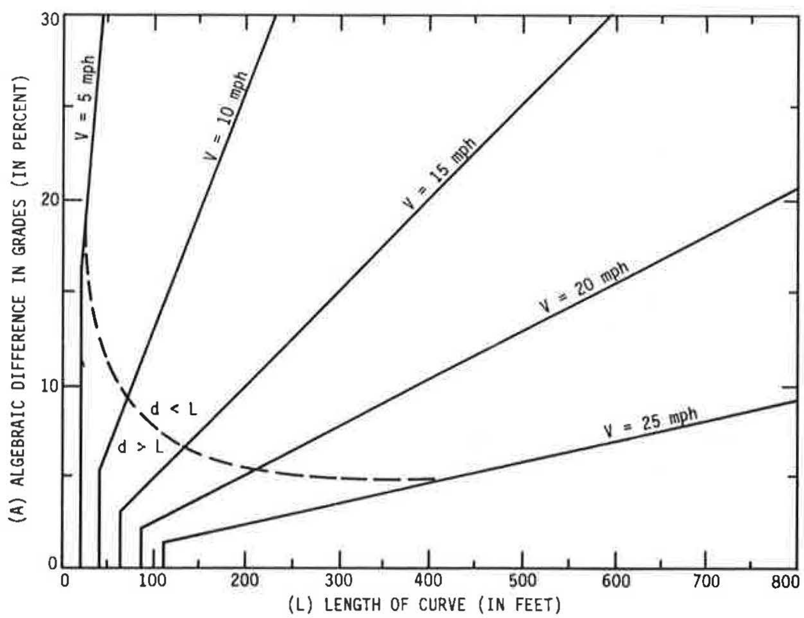

FIGURE 3 Minimum length of SAG vertical curve for LWSCs.

Cross Section

The function of the cross section is to accommodate vehicles on the roadway and to allow periodic higher stream flows to cross the roadway. Passenger vehicles are in the range of 6.0 to 6.5 ft wide, whereas pick-up trucks are in the range of 8 ft wide. Farm vehicles of much wider dimensions commonly use these types of roads and may legally do so. One of the advantages of an LWSC over a bridge, on a farm field access road, for example, is the unrestricted farm vehicle width that can be accommodated. Old bridges with guard rails on the approaches present problems for wide farm vehicles. Farm vehicles in common use have transport widths of 18 to 20 ft; some vehicles may reach 28 ft in transport width.

For design purposes, a 16-ft top width would be minimal, with a 20-ft or greater top width desirable. The roadway should be crowned to cause water to run off and reduce ponding on the roadway. As periodic overtopping of the roadway occurs, a crown of 0.02 ft per foot from the upstream side to the downstream side will tend to be more self-cleaning than a crown symmetrical about the centerline. Also, the pavement should have transverse grooves for traction.

Low water stream crossings have been constructed with vertical sides as well as with battered side slopes. Also, the pipes may protrude or be flush with the foreslopes of the cross section. The major disadvantage of a vertical foreslope is the debris-erosion problem. A 2:1 foreslope with smoothly trimmed pipes may be self-cleaning on the upstream side. Such a configuration provides a more hydraulically efficient design. The use of curtain walls on both the upstream and the downstream edges is common to reduce erosion and undercutting.

Traffic Control Signs

An LWSC has two unique characteristics not associated with a traditional bridge that may create a potential for accidents and subsequent liability claims. The vertical profile at the crossing is usually restricted to low speeds and the pavement surface is subject to periodic flooding. It is imperative that adequate warning of these conditions be transmitted to the user. The recommendations contained herein are based on the research by Carstens (1) and are shown in Figure 4.

The intent of the regulatory sign DO NOT ENTER WHEN FLOODED is to preclude travel across the LWSC when the roadway is covered with water. Such a regulatory sign requires a resolution by the Board of Supervisors. The adoption of this sign in effect precludes the use of an unvented ford.

SELECTION OF CROSSING MATERIALS

The surfacing material of any ford can be determined by using one of the three following methods that estimate a tractive force and velocity. Then these values can be compared with critical values for various materials. The first two methods rely on geomorphic relationships developed from flow gauging stations in Iowa. The first method presumes that the designer only has a knowledge of the size of the drainage area upstream of the proposed crossing site. Figures 5 and 6 are then used to relate watershed size to tractive force and velocity.

The recommended value that grass is capable of resisting is a velocity of 3 ft per second. Table 3 gives values of tractive force that different sizes of riprap are capable of resisting. Using Table 3, the engineer can select a riprap size that will be capable of resisting the τ_t values obtained from Figure 5. The tractive forces given correspond to the critical tractive force (τ_c), which the various sizes of riprap are capable of resisting. For values of velocity and tractive force greater than the values given previously, the engineer can use soil cement, gabions, fabriform, and portland cement concrete as construction materials. Considerations involved in the use of these materials are explained in the Iowa manual (2).

The second design method presumes that the engineer has detailed information about the channel's cross-sectional geometry in addition to knowing the watershed size. Using Figure 7, the designer can estimate a channel slope and depth of flow. The flow velocity can then be determined from Manning's

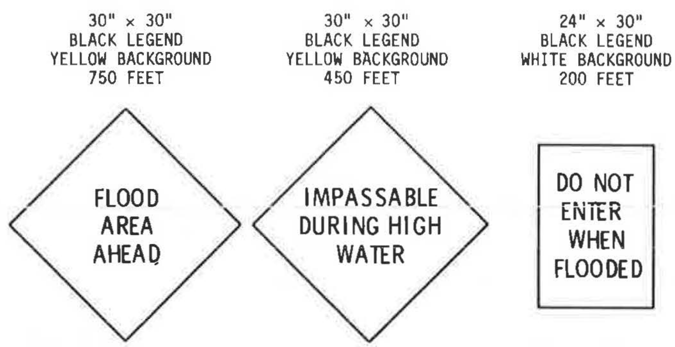

FIGURE 4 Signs recommended for installation at low water installations.

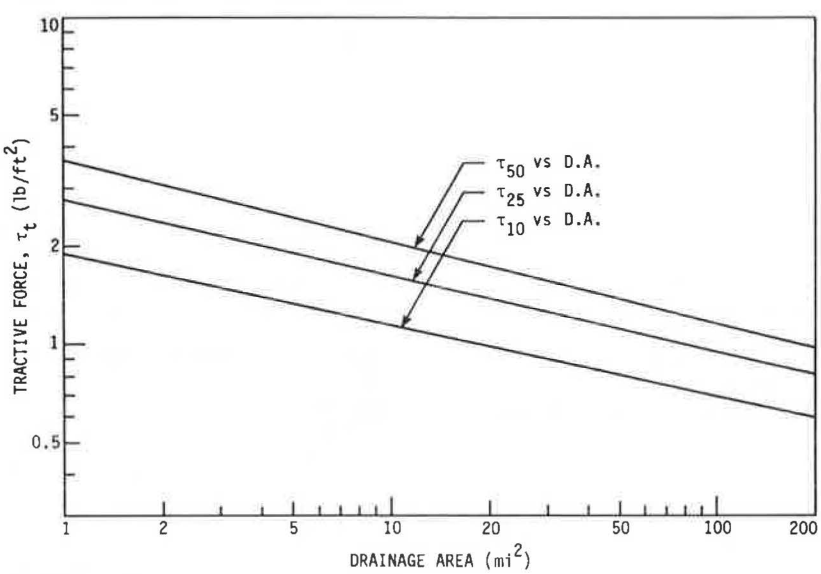

FIGURE 5 Tractive force (τ_t) versus drainage area.

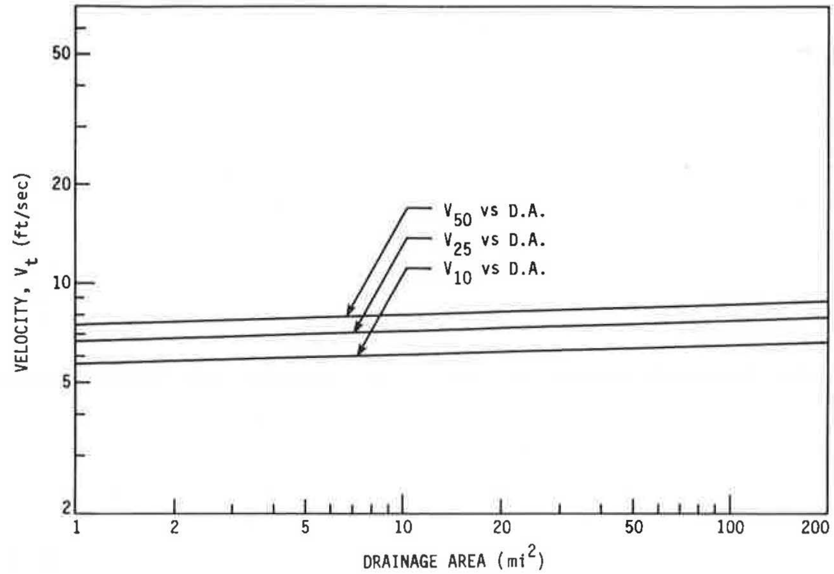

FIGURE 6 Velocity (V_t) versus drainage area, Region I.

TABLE 3 Critical Tractive Force Values for Different Sizes of Riprap

Material, in.	Critical Tractive Force, lb/ft^2
Riprap D_{50} = 6	2.0
Riprap D_{50} = 15	5.0
Riprap D_{50} = 27	7.3
Riprap D_{50} = 30	10.0

equation. The tractive force is calculated by using Equation 4.

$$\tau_t = 62.4Sd_t \quad (4)$$

where

τ_t = the tractive force in pounds per square foot for some return period, t;
S = the channel slope in feet per foot; and
d_t = the flow depth in feet for some return period, t.

The third method uses only physical data collected at the site: drainage area, channel cross section, channel slope, and valley cross section. The flow velocity and tractive force are determined as described in Method 2.

Using the values of V_t and τ_t calculated, suitable riprap can be selected by using Table 3 or other materials can be selected by considering the properties described in the Iowa manual (2). The designer can use one return period or, alternatively, can select values for all three return periods and determine the variation in construction material, if any, that results and use this information in the decision-making process.

SUMMARY

Most counties in the United States are faced with rising costs, stagnant or decreasing budgets, and an

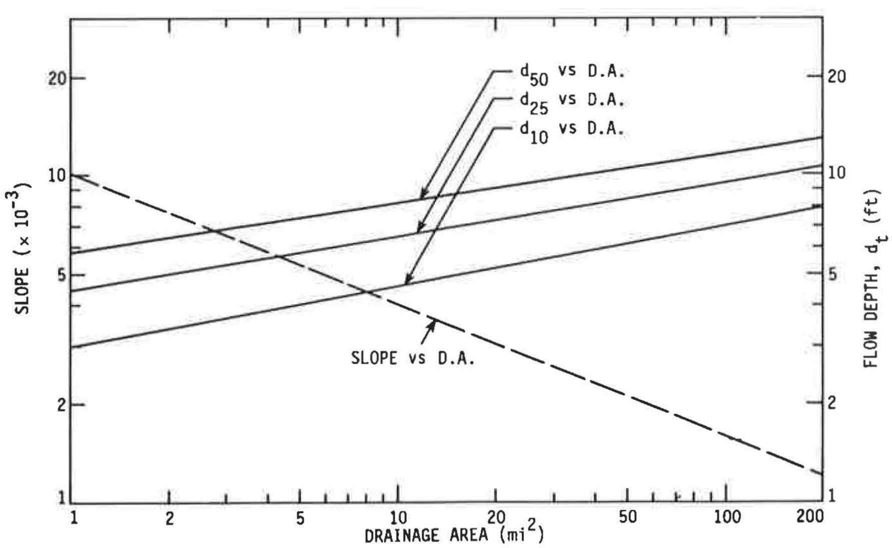

FIGURE 7 Slope and flow depth (d_t) versus drainage area, Region I.

increasing number of structurally and functionally obsolete bridges. Available funds must be stretched and new ways found to keep roads open. LWSCs are one method of replacing old bridges at a lower cost. However, because these low water crossings have an inherent dip in the road profile at the stream and because they are designed to be underwater several times a year, they present a possible hazard and must be properly designed and signed.

These design and signing aspects have been studied and the results presented in a design manual for LWSCs in Iowa. The types of crossings and locations where they may be used have been narrowly defined. In addition to the signing recommendations, the manual includes the hydrology, hydraulics, roadway geometrics, and material selection phases of the design process: estimates of flow for several overtopping durations are obtained from an equation developed by the USGS; the number and size of culverts for a vented ford are determined from a manual published by the Federal Highway Administration; considerations involved in the selection of road grades plus crest and sag vertical curve lengths are discussed; and three methods for designing protective materials to prevent erosion of the crossing are presented. The use of these guidelines and procedures should result in a well-designed and signed low water stream crossing.

ACKNOWLEDGMENTS

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The Effectiveness of Stormwater Detention

BEN URBONAS and L. SCOTT TUCKER

ABSTRACT

The effectiveness of stormwater detention is discussed in terms of quantity, water quality, and institutional constraints; and research needs are identified. The results of a study by the Urban Drainage and Flood Control District in Denver, Colorado, are presented to assess the effectiveness of random on-site detention in controlling flow rates along major drainageways. The study consisted of modeling an actual 7.85-mile^2 watershed in the Denver area under the 2-, 10-, and 100-yr rainstorm scenarios. The study suggests for the Denver region that random on-site detention has the potential of being reasonably effective in controlling the 10- and 100-yr flows along major drainageways. It also suggests that random on-site detention may not be effective in controlling frequently occurring flows such as runoff from 2-yr or smaller storms. The authors also discuss the design accuracy of stormwater systems and that institutional structure is needed to ensure the design, construction, and the continued operation of detention facilities. They conclude that such a structure is a must if detention is to be an effective part of the total stormwater management program.

The approach to drainage until the early 1970s relied on swales, curb and gutter, inlets, storm sewers, and channels to carry away flow as quickly as possible. In recent years this approach has been modified by the introduction of detention storage to hold back runoff and to release it downstream at controlled rates. The concept apparently has considerable appeal because it has been widely embraced throughout the United States, Canada, and many other countries throughout the world.

Although the concept of detention storage has been widely accepted, the questions regarding its effectiveness in managing stormwater runoff persist. It is relatively easy to study the hydrologic effectiveness of individual detention sites. It is another matter to study and quantify the effectiveness of a system of detention ponds, particularly if they occur randomly as to time of construction and in their location.

The investigation of the effectiveness of detention in managing or controlling urban runoff cannot be limited to hydraulic or hydrologic functions alone. Detention ponds, once built, become a part of the overall stormwater management system. They can play a vital role in controlling downstream flooding and have to be accepted into the infrastructure of the metropolitan areas they serve. Thus, the institutional arrangements and systems that can ensure adequate design, proper construction, and perpetually continuing maintenance need also to be considered and evaluated when the effectiveness of any stormwater detention system is assessed.

Even more recently (i.e., within approximately the last 5 years), stormwater detention began assuming an ever increasing role in controlling the water quality of urban runoff. Although attempts to use detention for this purpose date back at least 10 years, data from field installations have become available only recently. These new data now provide a glimpse of the potential effectiveness of detention storage in enhancing urban runoff water quality.

In August 1982 the Engineering Foundation and the Urban Water Resources Research Council of the American Society of Civil Engineers (ASCE) cosponsored a week-long conference on stormwater detention facilities planning, design, operation, and maintenance. Hydrology, water quality, and institutional issues were thoroughly discussed in the context of effectiveness of stormwater detention. The authors, who co-chaired this conference, used some of the information presented there, as well as their own work, to assess the effectiveness of stormwater detention and to identify topics that require further research and development. The purpose of this paper is to discuss the effectiveness of stormwater detention in terms of quantity, quality, and institutional constraints.

RECENT INVESTIGATIONS--QUANTITY

In November 1974 McCuen published an article (1) reporting the results of his modeling effort using 17 subwatersheds and two systems of detention storage. In one system, he modeled 12 ponds and, in another, he modeled 17 ponds. He used 10 storm events at the Gray Haven Watershed (2) to calibrate a "linked-process hydrograph simulation model" before adding the detention ponds to the system. The modeled watershed consisted of 23.3 acres of which 52 percent was impervious. Although the design of individual detention facilities was not described in the article, McCuen reported that the 17 subwatershed scenario had a total of 22,000 ft^3 of storage. On the basis of his modeling results, he suggested:

> 1) that the "individual-size" approach to stormwater detention may actually create flooding problems rather than reduce the hydrologic impact of urbanization; and 2) that a regional approach to urban stormwater management may be more effective than the "individual-site" approach.

In June 1976 Hardt and Burges published a report (3) on their investigation of detention effects from a hypothetical 2,000-acre watershed. Their investigation, using a Soil Conservation Service (SCS) runoff model and a kinematic channel routing technique, was limited to three subwatersheds; nevertheless, it was one of the earlier attempts to examine the effects of detention systems. Their findings are summarized in the following quote from their report:

> Restricting the outflow from a retention facility to a level less than the undeveloped rate could achieve a composite peak flow rate that would equal the pre-urbanization flow but would run for a much greater duration at that rate. The increased flow duration would have potentially undesirable effects on the channel system.

Linsley and Crawford (4) suggested the use of continuous simulation models in urban hydrology. Although this suggestion has considerable merit, it suffers from the fact that continuous record of rainfall is often not available. When it is available, the cost of such modeling can be very expensive, and the majority of design practitioners are not prepared to use continuous long-term modeling in the design of stormwater detention facilities. Walesh (5,6) suggested a technique to reduce a continuous hyetograph record to a reasonable number of discrete hyetographs that represent desired recurrence frequency storms. These representative recorded hyetographs can then be used to design stormwater management facilities, including detention. The reason for suggesting continuous simulation or the use of representative recorded hyetographs stems from the questioning of the validity of using a design storm (7-9).

This design storm controversy has not been resolved; however, the authors believe that there are definite applications, particularly water quality-oriented, where continuous simulation or quasi-continuous simulation should be used whenever rainfall data are available. On the other hand, the authors believe that the design of basic storm sewer systems, channels, and detention ponds can be accomplished with reasonable accuracy by using properly developed design storms. Urbonas (10), based on hydrologic studies in Denver, Colorado, expressed the following opinion:

> It is possible to develop design storms that reasonably duplicate the peak flows from small urban basins at various recurrence intervals. However, this requires substantial rainfall-runoff data to permit calibration of computer models, long term simulation of runoff using recorded rainstorms, and statistical analysis of simulated peaks and volumes.

Such design storms need to be developed for each locale using representative rainfall-runoff data. When developed, they can be used with confidence that the designs for the region will be reasonably accurate and responsive to the stormwater management needs of the region.

RECENT INVESTIGATIONS--QUALITY

Although the use of stormwater detention to enhance urban runoff water quality has been discussed for the last 10 years, only during the last 3 years has reliable data on stormwater detention effectiveness begun to emerge. Initial investigations were limited to efficiencies of sediment entrapment, which were correlated to the fall velocity of sediment particles in still water (11-13). These studies, however, did not identify the differing efficiencies of various pollutant entrapments.

In 1981 Whipple and Hunter (14) reported settleability measurements using a stilling glass tube. Measurements were made for hydrocarbons, suspended solids, 5-day biodegradable oxygen demand (BOD_5), total phosphates, lead, copper, zinc, and nickel for five urban storm runoff samples in New Jersey. They reported that stormwater retention can be effective in removing significant portions of particulate pollutants from runoff if sufficient retention time is provided. They also reported that the settleability varied widely between specific pollutants and even between storms for the same pollutants. They concluded that considerable research is still needed.

Rinella and McKenzie (15) have developed a methodology relating suspended chemical concentrations in stormwater to suspended-sediment particle-size classes based on settling velocities in quiescent native water. These relationships may help to characterize the removal rates of pollutants by sedimentation. The procedure is quite involved and requires one person 6 to 14 hours to separate suspended sediments into particle size classes. Nevertheless, it has the potential of becoming a basis for design of settling treatment ponds for urban runoff pollutants.

Randall et al. (16) also reported on studies of pollutant settleability in runoff for three shopping centers in Virginia. They found that after 32 hours of settling time, an average of 90 percent of the total suspended solids, 46 percent of chemical oxygen demand (COD), 34 percent of total organic carbon (TOC), 56 percent of total phosphorus, 33 percent of total nitrogen, 44 percent of zinc, 86 percent of lead, and 64 percent of BOD_5 had been removed from the water column. However, the levels of many of these constituents in the water column still remained higher than would be acceptable for maintenance of many stream standards.

As a result of the Environmental Protection Agency's National Urban Runoff Program, studies such as reported by Rinella and McKenzie (15), Whipple and Hunter (14), and Randall et al. (16) are beginning to develop some of the information needed for design of detention ponds for water quality enhancement. However, much more field data are needed to verify design technology before it can be confidently stated how effective a design will be in removing pollutants from urban runoff. In another paper, Randall (17) reported on the effectiveness of three ponds based on field observations. The results varied considerably between the sites. As a rule, the two ponds that had a permanent water pool outperformed the dry pond. For the dry pond, the concentrations of nitrogen constituents were greater in the outflow than in the inflow. Results such as these reveal that all of the basic water quality processes that occur in detention ponds are still not understood. Additional research will be required to identify and to quantify them and to develop design techniques that can reliably predict the performance of detention ponds used for water quality enhancement.

HYDRAULIC EFFECTIVENESS OF RANDOM DETENTION

The Urban Drainage and Flood Control District has an interest in stormwater detention in the Denver metropolitan area because it may affect the peak flows along major drainageways. For the purposes of this paper, a major drainageway is defined as one having at least a one-fourth $mile^2$ area tributary to it. In 1969 the District contracted with the U.S. Geological Survey (USGS) to collect simultaneous rainfall and runoff data, which were used to develop regionalized rainfall and runoff estimating procedures. These procedures were then the basis for calibrating a storm water management model for a rapidly urbanizing watershed in the metropolitan area, which was used to study the effects of random detention on the peak flow rates along major drainageways.

A study conducted by the District used an actual Denver area watershed as a study basin. The study watershed had an area of 7.85 $mile^2$, a watershed length of 6.4 miles with an average watershed slope of 0.015. Its shape and drainage pattern is shown in Figure 1, and it was estimated that 1.9 percent of its area was impervious before land development began. After full development, the watershed area is projected at 38 percent impervious.

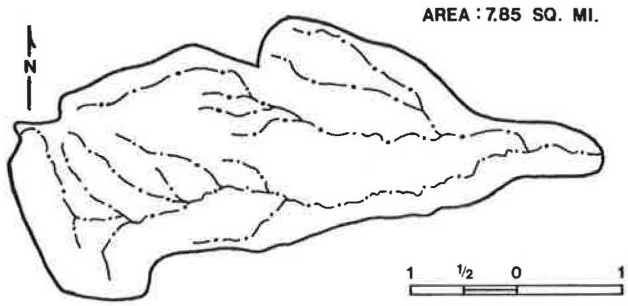

FIGURE 1 Study basin.

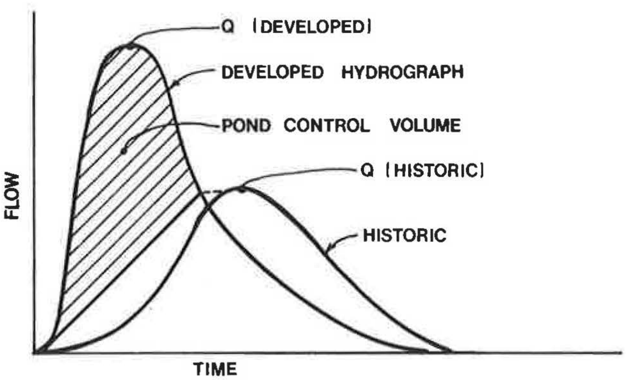

FIGURE 2 Determination of detention pond volume.

Runoff was modeled using 2-hr design storms for the 2-, 10-, and 100-yr recurrence frequencies. These design storms were developed for the Denver area by using the rainfall-runoff data collected by USGS. Modeling was done using stationary storms and mobile storms that traversed the watershed at 6 mph up-

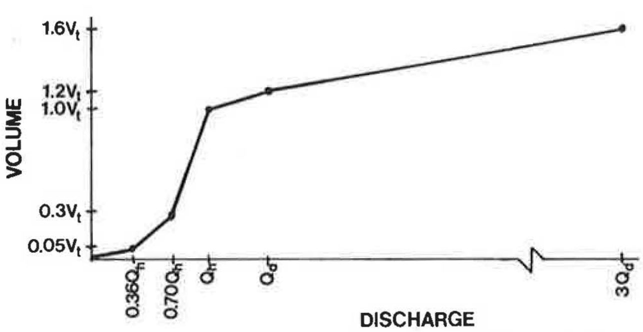

FIGURE 3 Volume versus discharge: 2-, 10-, or 100-year design.

stream and downstream. In addition, runoff was modeled by using three recorded rainstorms under the stationary and moving storm scenarios. Although the runoff results reported in this paper are for the stationary design storm scenarios, the effects of stormwater detention on each storm scenario are similar. Namely, if a reduction in peak flow is calculated with detention for the stationary storm scenario, then a similar reduction is also observed for the same moving storm scenario when compared with the undetained condition.

The results of the District's study have greatest validity for the Denver metropolitan area and other areas of United States having similar meteorological and hydrologic conditions. Because the modeling was for a 7.85-mile^2 watershed, conclusions of this study should not be extrapolated beyond 10 mile^2 watersheds. This appears to be a severe limitation; however, many of the observed rainstorms in the semi-arid climates have a rather limited footprint where the intense rainfall occurs. Thus, it is possible that for many intense rainstorms in semi-arid climates, controlling runoff from 10-mile^2 or lesser watersheds may be very beneficial for flood control purposes. The intent of the District's study was to gain an understanding of the generalized trends of stormwater detention effectiveness, and the results presented herein need to be viewed from that perspective.

The study watershed was subdivided into 56 subcatchments and 52 channel segments. After calibration, runoff was modeled using the various storm scenarios for the undeveloped and the urbanized land use conditions. The model was then modified to include 28 randomly located detention ponds. The ponds intercepted 91 percent of the total area with runoff from 9 percent of the area being undetained. Each pond was sized on the basis of the hydrographs from the before and after development conditions. The control volume was estimated using a process illustrated in Figure 2, where the control volume was assumed to be equal to the shaded portion of the runoff hydrograph.

The hydraulic characteristics of each pond's outlet was designed assuming that the outlet functioned as an orifice until the design control volume was filled, at which point the pond's overflow functioned as a weir. On the basis of observed trends in several individual designs, an outlet discharge versus storage volume relationship was developed in a nondimensional form. This facilitated the design and evaluation of a large number of detention control conditions. Figures 3 and 4 show the design characteristics used for the 28 ponds in the model. In Figure 3, Q_h is the peak flow from an undeveloped subbasin, Q_d is the peak flow from a developed subbasin, and V_T is the design control volume of the pond. In Figure 4, Q_h and Q_d represent

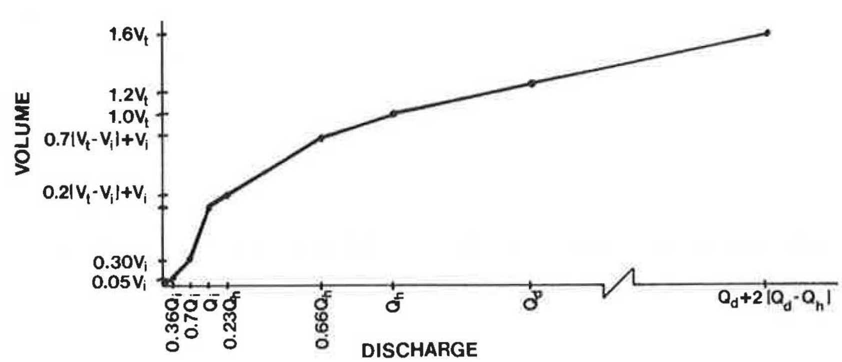

FIGURE 4 Volume versus discharge: 10- and 100-year combination design.

the undeveloped and developed 100-yr storm peak flows, V_T represents the 100-yr control volume, and Q_i and V_i represent the undeveloped peak flow and the required control volume to the 10-yr storm.

Many of the results of the District's random detention study can be found in Glidden (18). Following herein is a series of five figures (i.e., Figures 5-9) that summarize the generalized trends projected by the random detention modeling study. Each figure relates the size of the watershed to the nondimensional peak flow of that size of watershed. The nondimensionalized peak flow was obtained by dividing the actual peak flow by the peak flow from the undeveloped watershed. As an example, a value of one on the ordinate represents no change from the undeveloped condition and a value of two represents an increase in peak flows by a factor of two from the undeveloped condition.

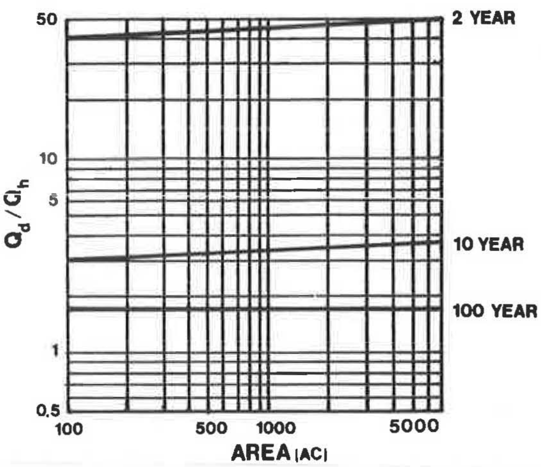

FIGURE 5 Urban runoff trends—developed without detention.

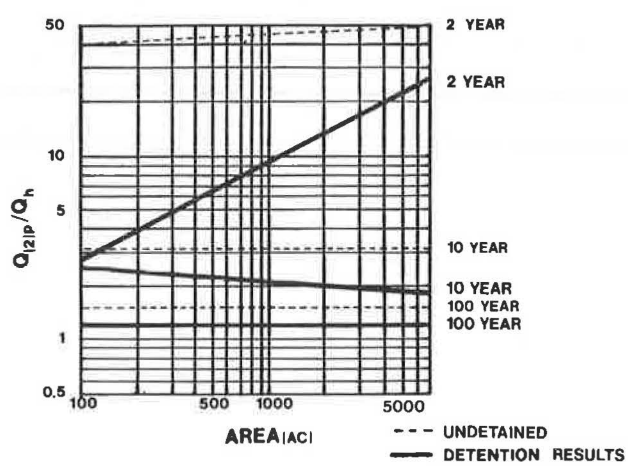

FIGURE 6 Two-year design effectiveness.

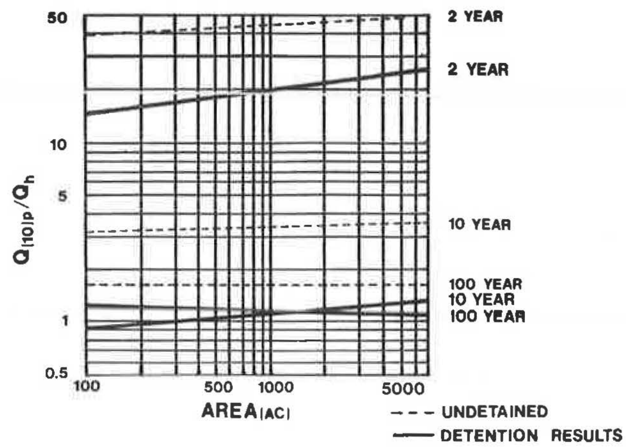

FIGURE 7 Ten-year design effectiveness.

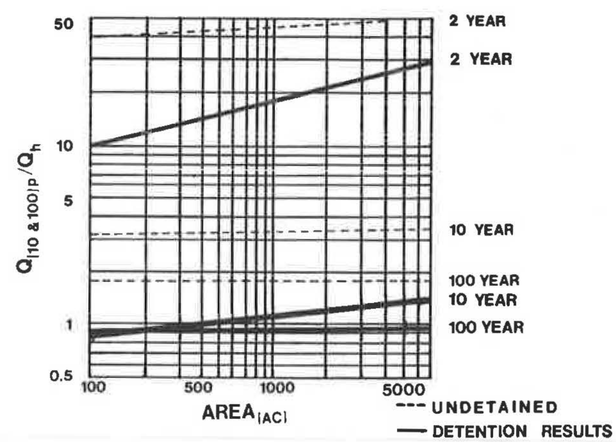

FIGURE 8 One hundred-year design effectiveness.

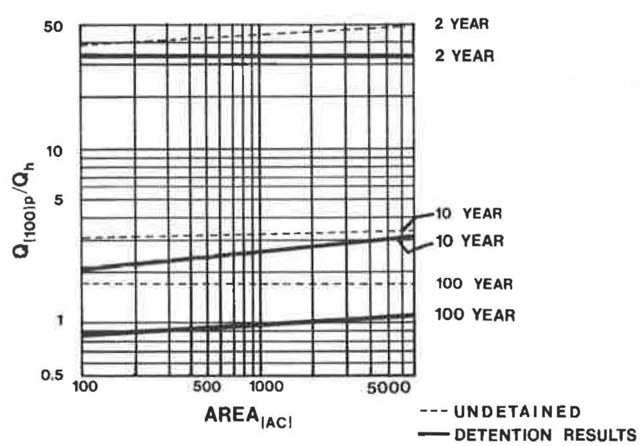

FIGURE 9 Ten- and 100-year combination effectiveness.

The subscript d in Figure 5 refers to the flow conditions when the basin is developed, and the subscripts 2p, 10p, 100p, and 10 and 100p refer to the flow conditions under different detention policy scenarios. Figure 5 shows the estimated trends in peak flows along the major drainageways without on-site detention, and Figures 6 through 9 show the trends when different on-site detention designs are used. It is important to recognize when studying these figures that the trends they suggest are applicable only to semi-arid meteorological zones similar to the Denver region.

A study of Figures 6 through 9 reveals the following trends:

1. The 2-yr random detention pond design was effective only on an individual pond site basis in controlling the 2-yr storm runoff. As the number of

ponds increased with the increasing tributary area, the 2-yr design rapidly diminished in effectiveness. This is because the 2-yr storm volume increased manyfold after development and, although the peaks were controlled at the detention individual sites, the resulting flat peaked outlet hydrographs added directly as the flow progressed downstream. In contrast, before development the individual tributary hydrographs had small volumes and were out of phase with each other. The 2-yr design reduced somewhat the 10-yr and the 100-yr storm runoff peaks when compared with the undetained condition.

2. The 10-yr random detention pond designs were relatively effective in limiting runoff peaks along the major drainageways from the 10-yr storms and was somewhat effective in controlling the 100-yr storm, but was virtually ineffective in controlling the 2-yr design storm.

3. The 100-yr design was effective in controlling the 100-yr storm but was virtually ineffective in controlling the 2- and 100-yr storms.

4. The combination 10- and 100-yr pond design was effective in controlling the 10- and 100-yr storm runoff, but was ineffective in controlling the 2-yr storm runoff. The two frequency designs appeared to be more effective in controlling their two design storms than the individual 10- and 100-yr frequency designs were in controlling their respective recurrence storm runoff.

The results of the District's study appear to verify some of the conclusions of other investigators (3). The one surprise, although predictable, was that the 2-yr design was not very effective in controlling peak flows along the major drainageways from the smaller storms. It may be that McCuen's (1) study, because it used recorded data, was limited to such smaller storms. It does not mean that the 2-yr design is ineffective for individual sites and may be more effective than the study results indicate if the spatial distributions of the smaller storms are considered. Additional work is needed to quantify realistic spatial storm patterns before the 2-yr detention design effectiveness in controlling peaks along major drainageways can be assessed.

DESIGN ACCURACY AND EFFECTIVENESS

The topic of design accuracy was indirectly mentioned in the earlier discussion of the design storm concept. The possible citations concerning urban design storms are numerous and have been tabulated by the Design Storm Task Committee of the Urban Water Resources Research Council into an Annotated Bibliography (19) that can be obtained on request from ASCE. The mere fact that design storms or their substitutes are used as input in the sizing of detention basins leaves a lot of room for argument concerning their design accuracy and their effectiveness. Although the questioning has merit and should not stop if technology is to move forward, it should not paralyze a designer into an endless analysis process. In the authors' opinion, it is important that the designers recognize the limitations in the accuracy of the rainfall input, yet move forward to design what are considered reasonably sized facilities in line with current state of the art.

Unlike many other fields of engineering, the statistics of hydrologic data have very wide bounds of design confidence. As an example, a 1980 USGS document (20) provides regression equations and techniques for estimating flood peaks, volumes, and hydrographs on small streams in South Dakota. The maximum estimated ranges in the standard error of estimate are +152 and -60 percent for the flood peaks and +136 and -58 percent for the runoff volumes. Such uncertainties, as an example, in structural analysis would be considered intolerable and would be dealt with through the use of very large safety factors. On the other hand, drainage and flood control engineers work with similar kinds of uncertainty all the time whether they know it or not. Thus, whenever accuracy or effectiveness is discussed, the randomness of the physical phenomenon involved should be kept in mind as well as the fact that the data base that was used in developing all of the surface runoff calculating techniques oftentimes had very broad bands of data scatter.

Institutional Constraints

In their discussion, Jones and Jones (21) point out that many communities mandated misuse of detention ponding with resultant waste of land and economic resources. They encourage communities to avoid arbitrary specification of single recurrence probability in their ordinances. Instead, communities need to reexamine their selected design basis and attempt to arrive at a design basis that is demonstrably cost-effective. Too often, either the extreme rare event or the small frequent event are the basis for local requirement, which, when applied uniformly and without regard to the effects downstream, can lead to either local drainage and erosion problems or to flooding problems. Jones and Jones stated further:

> It follows that design of detention pond outlet works often should have a multi-probability basis: (a) for frequent low flow conditions; (b) for the detention design discharge condition; and (c) for the extreme runoff (emergency spillway) condition.

The District's study revealed that even though the smaller storms may be the pond design criteria, the increased runoff volume resulting from urbanization virtually precludes design of on-site ponds that can effectively control peak flows along downstream drainageways. This mandates that downstream drainage facilities cannot arbitrarily be sized to accommodate flow from historic or undeveloped watershed only on the basis of on-site detention policy. It is incumbent on communities to also examine the detention requirements for each site, when detention is required, to ensure that pond releases will not create hazards or damages to downstream properties.

Requiring on-site detention is not an assurance that the drainage needs of the community and those of the new development are satisfied. Communities and developers need to recognize that detention, when used, is only one element of a total formalized (or natural) drainage system and that it cannot be treated haphazardly. Thus, institutional arrangements in communities are equally as important as sound design practices. In other words, communities need an institutional structure that not only ensures sound design, but also ensures that the required detention ponds fit the system and are not used merely to pacify local regulatory requirements.

Beyond this, an institutional structure is needed to ensure that detention ponds are properly constructed and maintained for as long as they are a part of the community's drainage system. Assessing the potential hydraulic effectiveness of a detention ordinance can be compared to weighing candy with only one-half of a balance scale. Even though the product looks attractive, it is impossible to know the quantity. If there is an emerging theme among the stormwater management professionals, it is that more often than not such institutional structures are not in place, are inadequate, or are underfunded. Thus, the true effectiveness of detention systems cannot be assessed without knowledge of how policy requirements translate into physical facilities and how these facilities will continue to function over the many years they are expected to operate.

RESEARCH NEEDS

During the 1982 Stormwater Detention Facilities Conference, workshops were held to identify research needs regarding the quantity, quality, and institutional aspects of stormwater detention. Summaries of these workshops are included in the conference proceedings (22), which contain probably the most comprehensive listing of research needs ever compiled on the topic of stormwater detention. It is not really possible to add to those lists; however, some of the research needs considered particularly relevant to the topic of effectiveness are highlighted here.

In the area of hydrology and hydraulic effectiveness, there still remains a need to improve runoff estimating techniques. Any additional research on this topic has to be field data-based. There are sufficient models of every sort at this time; what is still lacking is good quality long-term data for rainfall and simultaneous runoff. In addition, very little is understood at this time by hydrologists about meteorological processes and spatial patterns of rainfall. It is not enough to collect point rainfall data. Hydrologists need to learn more about weather movements and the causes of different types of storms. This will require the setting up of dense raingauge networks before sufficient data can be collected to quantify spatial patterns of rainfall. Such information, once developed, may permit the confidence limits in urban surface runoff hydrology to be narrowed.

Additional research work is also needed in the area of random on-site detention effectiveness. The District's work was very limited and site specific. Considerable additional work is needed before we can be confident of the effectiveness trends by various random systems at on-site detention. Also a corollary effort is needed to determine if there is merit to variable on-site detention requirements. That is, should all detention in the watershed be sized for the same requirement, or is it more cost-effective to require different volumes and release rates depending on the location and development patterns in the watershed? Lakatos and Kropp (23), on the basis of their modeling work, have suggested just such an approach in Pennsylvania.

In the area of water quality, considerable research, using field data, is needed to develop reliable water quality enhancement design procedures. In addition, there is an immense lack of understanding of the basic physical, chemical, biochemical, and biological processes taking place in ponds used for water quality enhancement of urban runoff. In the authors' opinion, these processes need to be identified and understood before real progress can be made in developing sound design procedures.

Finally, work is needed in the institutional-related areas. Institutional elements contributing to successful programs need to be clearly identified so that professionals in other communities have models to follow. As part of the institutional issues, the needs and cost of an on-site detention maintenance program need to be quantified. Such information is vital if communities are to make sound decisions concerning detention requirements. For example, what are the elements of a pond that facilitate easy, low cost maintenance and what are the elements that do just the opposite?

CONCLUSIONS

The effectiveness of on-site detention ponds was addressed from the quality, quantity, and institutional aspects. Recent investigations have begun to indicate that detention ponds can be effective in improving the water quality of urban runoff. Generally, one-half to one- and one-half days of retention time is required in the pond to show a significant improvement. Also, it appears that ponds with a permanent waterpool are more effective than dry ponds. However, much more data and experience are needed to draw firm conclusions over the long term.

The model study of random on-site detention in one Denver area watershed has indicated the following:

1. When ponds are designed to control the peak flow from a single recurrence event, the effectiveness of the system in controlling flow rates along major drainageways is limited only to events of the same design recurrence frequency.
2. Ponds designed to control peak flows of two separate recurrence frequencies appear to be effective in controlling flow rates along major drainageways for a range of flows and also appear to be more effective in controlling the two individual design storms.
3. Designs intended to control frequent events (e.g., 2 years) are effective immediately downstream of each pond. They appear to be less and less effective in controlling the flows along the major drainageway as more and more ponds contribute to the system. A much better understanding of spatial distribution of rainstorms will be needed to fully substantiate this conclusion.

Finally, any assessment of the effectiveness of random on-site detention needs to consider the institutional structure that ensures adequate design, proper construction, and long-term operation and maintenance. Otherwise, an assessment of the effectiveness of any individual community's detention system is an exercise in futility.

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Semi-Arid Storm Hyetograph Properties in Wyoming

VICTOR R. HASFURTHER and PATRICK T. TYRRELL

ABSTRACT

Design storm patterns for use in predicting floods by simulating precipitation events in ungauged drainage basins in Wyoming are presented. The design patterns were developed from observed rainfall and are separated into two categories: thunderstorms (events less than 4 hr in duration) and general storms (events lasting 4 or more hr). Comparisons of predicted runoff using the new design storms and design storms recommended by the Soil Conservation Service (SCS) and the Bureau of Reclamation (BUREC) were made using the following models on a 0.83-mile^2 watershed: (a) HEC-1 (the Hydrologic Engineering Center); (b) HYMO (Problem-Oriented Computer Language for Hydrologic Modeling); (c) HYDRO (the SCS Triangular Hydrograph); and (d) the U.S. Geological Survey (USGS) distributed-routing digital rainfall-runoff models. The new design storms typically produce greater runoff peaks when simulating thunderstorm events, and, in most cases, smaller peaks when simulating runoff from general storms, than those predicted with the established procedures. Instructions describing the use and limitations of the new storm pattern construction method are included.

The design of hydraulic structures for use in ungauged drainage basins requires some estimate of flood flows and their frequency of occurrence. Because no historical streamflow data exist for these drainages, floods are generally estimated either by regional frequency analysis or, with the help of digital computers, by parametric rainfall-runoff event simulation.

Computer models dealing with rainfall-runoff event simulation are commonly used today by engineers and hydrologists. These models are used to predict flood hydrographs given an input rainfall volume, distributed over time in some manner, and certain geomorphic, soil, geologic, vegetative, or other basin parameters.

Studies exist in the literature that document the effects of time distribution of rainfall on runoff hydrographs. The reader is referred to works by Wei and Larson (1), Yen and Chow (2), and Shanholtz and Dickerson (3) as examples. Because this relationship between the time distribution of rainfall and hydrograph characteristics exists, the separate study of storm rainfall is essential for accurate flood prediction notwithstanding other variables that also influence the runoff process. In addition, methods of constructing design storms are available and in wide use, but they are general in nature and assume storms occur with the same temporal distribution across much of the country. Because of the drastic climatic differences between the areas encompassed by existing procedures, it was believed that the design curves of these methods are not likely to be representative of the actual time distribution of storms in semi-arid regions. It was therefore decided to develop a new design storm construction procedure applicable to semi-arid areas based on observed storm rainfall in Wyoming.

REVIEW OF PREVIOUS WORK

Relatively few precipitation studies conducted to date deal with the temporal distribution of rainfall in the manner used by hydrologists and engineers in parametric flood prediction.

The Soil Conservation Service (SCS) method (4) presents three temporal rainfall distribution curves for runoff prediction. The Type I and Type IA curves are used for studies in Alaska, Hawaii, and the coastal side of the Sierra Nevada and Cascade mountain ranges. The Type II curve is applied in the remaining part of the United States, Puerto Rico, and the Virgin Islands. These curves are based on generalized rainfall depth-duration curves obtained from published data of the U.S. Weather Bureau [National Oceanic and Atmospheric Administration (NOAA)]. All design storms developed using this method, regardless of duration, are based on the 24-hr volume for a given frequency and location.

The Bureau of Reclamation (BUREC) method (5) is developed in two parts, one for the United States east of the 105-degree meridian and the other for areas west of the 105-degree meridian. The procedure requires arranging hourly rainfall increments in a specified sequence depending on the duration and type of storm (thunderstorm or general storm). Maximum 6-hr point rainfall values are used in designing general storms, and maximum 1-hr point rainfall values are used in designing thunderstorms.

The U.S. Weather Bureau procedure (6) uses depth-duration-frequency (DDF) curves in design storm construction. In this method, rainfall intensities are obtained from the DDF curves for a given frequency and duration at a certain locality. These intensities are then rearranged arbitrarily to form a storm pattern.

Kerr et al. (7) present a method of hyetograph construction for Pennsylvania. Cumulative dimensionless rainfall versus time graphs used by the method are derived from historical rainfall data. The curves allow the user substantial flexibility because, rather than define a single storm sequence, they bracket a range of possible storm patterns. Selection of the time distribution of a design storm can be made by the user, providing the limits of the bracketing curves and the minimum and maximum intensities given are observed.

Huff (8) presents a procedure derived from heavy storms observed in Illinois. His distribution patterns are based on the time quartile in which the majority of rain occurs for a given storm. For each quartile storm type, frequency values are given so that the user knows the return period of his design storm.

A method described in Keifer and Chu (9) uses intensity-duration-frequency curves for hyetograph design at a given location. In general, the proposed storm pattern is fit to exponential growth and decay curves with the most intense part of the storm defined by a parameter termed the "advanceness ratio." This method was developed in Chicago for urban sewer design but can easily be used in other areas of the country where adequate rainfall records are available.

Frederick et al. (10) developed annual maximum precipitation events for different durations. The largest precipitation amounts for the selected durations that coincide with a given duration event are selected. The events are stratified according to magnitude, and ratios of shorter to longer duration precipitation totals are formed. Accumulated probabilities of this ratio are suggested as a tool to estimate precipitation increments necessary in the synthesis of precipitation mass curves. By analyzing the relative timing of the shorter duration event within the longer duration event, a characteristic time distribution can be developed.

METHODOLOGY

Accumulation of Rainfall Data

The study of time distribution of rainfall requires historic data recorded as continuously as possible. Because continuously recorded rainfall data were not available in the quantities needed for this study, discrete data were used. Hourly measurements from NOAA publications (1948-1979) (11) provided the data base for the study of general storms whereas the 5-min incremental precipitation data available in Rankl and Barker (12) were used in thunderstorm analysis. The precipitation stations used from both sources are described in Table 1.

The definition of a storm had to be established before usable information could be obtained from the data. In this paper, the criteria used for defining a storm are as follows:

1. General storm--preceded and followed by at least 2 hr of zero rainfall, at least 4 hr in duration, and at least 0.5 in. in volume.
2. Thunderstorm--preceded and followed by at least 1 hr of zero rainfall, at least 20 minutes and at most 4 hr in duration, and at least 0.5 in. in volume.

These criteria are arbitrary but consistent with similar criteria recommended by Huff (8), Ward (13), and Croft and Marston (14). Minimum duration requirements were used to ensure that the time distribution of any storm was described by at least four data points. A total of 531 general storms and 72 thunderstorms were examined.

The period of record represented by the data at most stations covers the years 1969-1979, though the lack of definable storms at some stations required data from as early as 1948. Because the development of design storms inherently assumes future rainfall events will occur with the same distribution as past events, the use of data from stations with variable periods of record is acceptable, assuming consistency of past records.

Description of Study Areas

The state of Wyoming was divided into its major surface water drainage basins for this study. This was done to determine if differences in storm rain-

TABLE 1 Precipitation Stations Providing Data for Study

Reference Number	Location Name or Number	Major Drainage Basin	Source	Recording Interval
1	Casper WSO AP	North Platte	NOAA[a]	1-Hour
2	Cheyenne WSFO AP	North Platte	NOAA	1-Hour
3	Douglas Aviation	North Platte	NOAA	1-Hour
4	Encampment	North Platte	NOAA	1-Hour
5	Jelm	North Platte	NOAA	1-Hour
6	Laramie 2 WSW	North Platte	NOAA	1-Hour
7	Medicine Bow	North Platte	NOAA	1-Hour
8	Oregon Trail Crossing	North Platte	NOAA	1-Hour
9	Pathfinder Dam	North Platte	NOAA	1-Hour
10	Phillips	North Platte	NOAA	1-Hour
11	Pine Bluffs	North Platte	NOAA	1-Hour
12	Rawlins FAA AP	North Platte	NOAA	1-Hour
13	Saratoga 4 N	North Platte	NOAA	1-Hour
14	Seminoe Dam	North Platte	NOAA	1-Hour
15	Shirley Basin Station	North Platte	NOAA	1-Hour
16	Torrington 1 S	North Platte	NOAA	1-Hour
17	Wheatland 4 N	North Platte	NOAA	1-Hour
18	Buffalo	Powder	NOAA	1-Hour
19	Douglas 17 NE	Powder	NOAA	1-Hour
20	Dull Center	Powder	NOAA	1-Hour
21	Gillette 18 SW	Powder	NOAA	1-Hour
22	Hat Creek 14 N	Powder	NOAA	1-Hour
23	Lance Creek	Powder	NOAA	1-Hour
24	Moorcroft	Powder	NOAA	1-Hour
25	Mule Creek	Powder	NOAA	1-Hour
26	Newcastle	Powder	NOAA	1-Hour
27	Osage	Powder	NOAA	1-Hour
28	Pine Tree 9 NE	Powder	NOAA	1-Hour
29	Powder River	Powder	NOAA	1-Hour

TABLE 1 (continued)

Reference Number	Location Name or Number	Major Drainage Basin	Source	Recording Interval
30	Recluse	Powder	NOAA	1-Hour
31	Sheridan WSO AP	Powder	NOAA	1-Hour
32	Story	Powder	NOAA	1-Hour
33	Boysen Dam	Big Horn	NOAA	1-Hour
34	Lander WSO AP	Big Horn	NOAA	1-Hour
35	Meteetse 1 ESE	Big Horn	NOAA	1-Hour
36	Powell Field Station	Big Horn	NOAA	1-Hour
37	Riverton	Big Horn	NOAA	1-Hour
38	Tensleep 4 NE	Big Horn	NOAA	1-Hour
39	Thermopolis	Big Horn	NOAA	1-Hour
40	Thermopolis 25 WNW	Big Horn	NOAA	1-Hour
41	Worland	Big Horn	NOAA	1-Hour
42	Big Piney	Green	NOAA	1-Hour
43	Mountain View	Green	NOAA	1-Hour
44	Mud Springs	Green	NOAA	1-Hour
45	Rock Springs FAA AP	Green	NOAA	1-Hour
46	Lake Yellowstone	Yellowstone	NOAA	1-Hour
47	Jackson	Snake	NOAA	1-Hour
48	Moran 5 WNW	Snake	NOAA	1-Hour
49	Evanston 1 E	Bear	NOAA	1-Hour
50	06631150	North Platte	USGS[b]	5-Minutes
51	06634910	North Platte	USGS	5-Minutes
52	06634950	North Platte	USGS	5-Minutes
53	06644840	North Platte	USGS	5-Minutes
54	06648720	North Platte	USGS	5-Minutes
55	06648780	North Platte	USGS	5-Minutes
56	06312910	Powder	USGS	5-Minutes
57	06312920	Powder	USGS	5-Minutes
58	06313050	Powder	USGS	5-Minutes
59	06313180	Powder	USGS	5-Minutes
60	06316480	Powder	USGS	5-Minutes
61	06382200	Powder	USGS	5-Minutes
62	06233360	Big Horn	USGS	5-Minutes
63	06238760	Big Horn	USGS	5-Minutes
64	06238780	Big Horn	USGS	5-Minutes
65	06256670	Big Horn	USGS	5-Minutes
66	06267260	Big Horn	USGS	5-Minutes
67	06267270	Big Horn	USGS	5-Minutes
68	06274190	Big Horn	USGS	5-Minutes

[a] NOAA (11)
[b] Rankl and Barker (12)

fall characteristics exist between basins. Figure 1 shows the state of Wyoming divided into these major drainages along with the precipitation stations used in this study. It should be noted that the precipitation data base (Figure 1) is not well distributed across the state and that most of the precipitation stations are located in valley areas. Data for the thunderstorm analysis are mainly concentrated in the center of the state.

Analysis of Storm Parameters

Determining if differences in storm rainfall characteristics exist between basins requires statistical analysis of certain storm parameters. Definitions of parameters used in describing storm rainfall follow:

1. Storm duration--the amount of elapsed time, in hours, from the beginning to the end of a storm.
2. Storm volume--the total amount of rainfall measured during a storm, in inches.
3. Rain intensity--the average rainfall rate during a storm, in inches per hour, calculated by dividing a storm's volume by its duration.
4. Percent time to peak intensity--the amount of time, expressed as a percent of total storm duration, from the beginning of a storm to the period of most intense rainfall.
5. Pattern index--the area beneath a dimensionless cumulative rainfall versus time curve, expressed as a decimal or as a percent.

Pattern index and percent time to peak intensity were the parameters used for determining whether differences in the time distribution of rainfall exist between basins. This determination was made by using a one-way analysis of variance (ANOVA) technique for samples of unequal size. The procedure, described in Miller and Freund (15), tests for differences in the population means for the populations

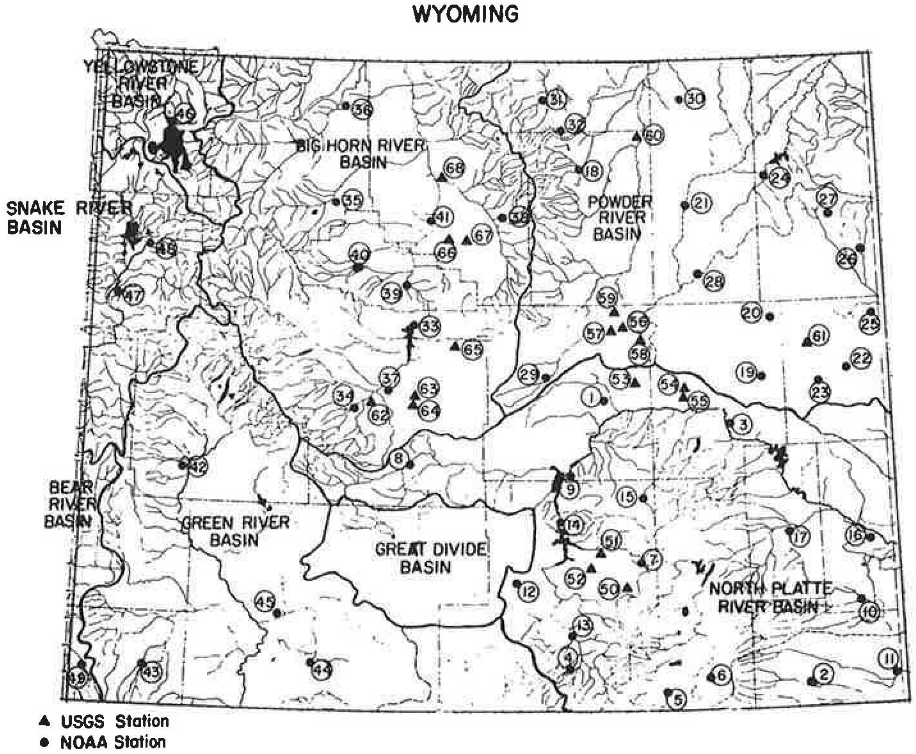

FIGURE 1 Map of Wyoming indicating the major surface water drainages. Station numbers refer to Table 1.

from which the samples were taken. Such tests indicate whether significant differences in parameter values exist between all the major drainages. If differences existed, the state would have to be divided accordingly before design storms could be constructed. If no differences existed, the state as a whole could be analyzed with the resulting design storms applicable statewide. The other parameters were used for describing the rainfall characteristics of each major drainage and for the state as a whole.

Construction of Design Curves

All the observed dimensionless mass rainfall curves are superimposed on one graph to create a family of probable storm patterns. Such an approach to design storm development is described in Kerr et al. (7). The most attractive feature of this method is its flexibility, which allows the user a choice of three given design hyetographs, as well as the freedom to construct a hyetograph, within limits. Such flexibility is desirable when, for example, a person is designing a structure based on peak flow-rate in one instance and on runoff volume in another. The use of several curves can allow maximization of either peak discharge or runoff volume for a given storm volume. A single design curve does not have this ability.

Figure 2 is a set of design curves. All of the storms used in the development of this set of curves are nondimensionalized and plotted on one graph of percent rainfall versus percent time. The bold vertical lines at each 10 percent time increment represent the range of all storm data used. In the center of the plot is the mean curve. The curve is fit through the points representing the average cumulative percent rainfall at each 10 percent time increment. It should be noted that the mean curve does not describe the average observed storm;

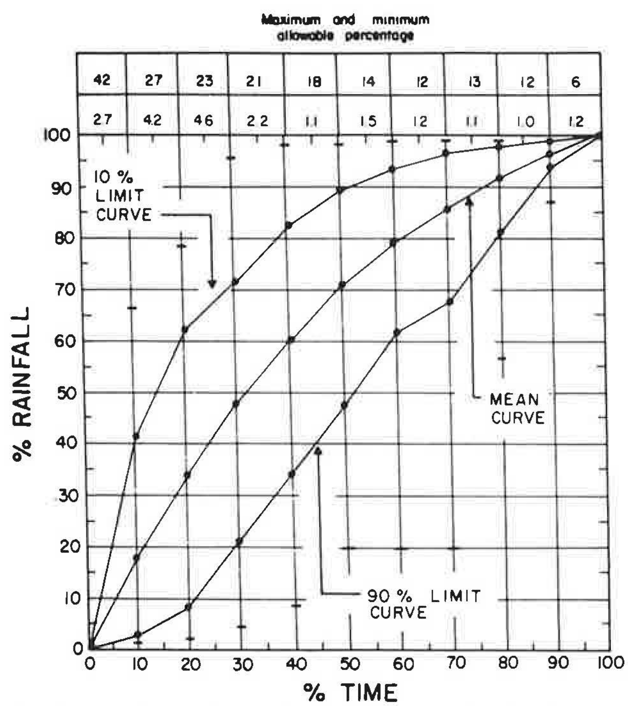

FIGURE 2 Dimensionless design mass curves for thunderstorms in Wyoming.

rather, it shows average accumulated rainfall with time based on all storms used.

Also drawn on the plot are 10- and 90-percent limit curves. The 10-percent limit curve represents, at a given percentage of storm duration, that value above which 10 percent of the storms had accumulated more precipitation. Similarly, 10 percent of the storms had each accumulated less than the value described by the 90 percent limit line at a given percentage of storm duration. It is incorrect to assume that 10 percent of the storms were totally above the 10 percent limit line or totally below the 90 percent limit line. The use of 10 percent as the cutoff when defining the upper and lower limit lines is arbitrary but reasonable. Using a smaller cutoff percentage resulting in a broader set of enveloping limit curves would be too general to accurately predict probable storm patterns. A larger cutoff value would result in a narrower envelope and a loss in flexibility of the method.

Under the assumption that future rainfall events will have the same time distribution as past events, these limit curves are the boundaries of a region of probable storm sequences. The user of the curves has the freedom to use either limit curve or the mean curve when choosing a design storm. The user may choose his own storm sequence as long as it is between the limit curves at all times and adheres to the maximum and minimum percentage guidelines in the first line of Figure 2. These percentage guidelines are constructed in a manner similar to the limit curves in that for each 10 percent time interval they represent intensities exceeded by only 10 percent of the storms (minimum percentage of storm volume for that 10 percent increment of time) as well as intensities exceeded by more than 90 percent of the storms (minimum percentage of storm volume for that 10 percent increment of time). In using these percentage guidelines, the designer cannot create a storm with a percentage greater than the value defined by the maximum or less than that defined by the minimum for the appropriate 10 percent time increment of storm duration.

Designing storms in this manner makes the utmost use of historical rainfall patterns while allowing the user flexibility in choosing the time distribution that will provide the critical peak discharge or runoff volume for his purpose.

Comparison of Storm Design Methods

The creation of new storm patterns for use in a particular region is logically accompanied by a comparison of the results of using the new method with results obtained using established design storm techniques. Such a comparison will prove the need for the new region-specific design curves if the existing general methods do not produce similar runoff characteristics when applied to a given event.

The different storm designs are compared by inputting them to four different rainfall-runoff simulation models and examining the runoff hydrographs produced. Thunderstorm and general storm runoff are simulated with each model. For each model and storm type, the infiltration parameters are held constant so that any differences noted in outflow hydrograph characteristics can be attributed to differences in the input hyetographs. The models used are described in Table 2. In addition to the design storm construction method presented in this paper, techniques given by SCS (4) and BUREC (5) are used for comparison. These last two methods have been described in the review of previous work.

DESIGN STORM RESULTS

Statistical Analysis

Examination of the linear regression and ANOVA tests performed on the rainfall data leads to the following conclusions:

1. A difference in the time distribution of thunderstorm rainfall compared to general storm rainfall exists for the entire state of Wyoming.
2. The time distribution of both thunderstorms and general storms is not dependent on the drainage basin in which the storms occur. However, the data in Figure 1 indicate that the data base used was not well distributed across the state.
3. No relationship exists between time distribution characteristics and duration of general storms or thunderstorms.

Inferred by Conclusions 1 and 2 is the need for only one set of general storm design curves and one set of thunderstorm design curves for use statewide. Conclusion 3 infers that design storms of varying duration, that is 1-, 2-, or 3-hr thunderstorms or 6-, 12-, or 24-hr general storms, can all be handled with the same set of design curves. Table 3 lists the results of selected important linear regression and ANOVA tests used in drawing these conclusions. The rest of the statistical analysis results can be found in Tyrrell (22).

Probably the most outstanding characteristic of the storms analyzed is their individual diversity.

TABLE 2 Description of Digital Computer Models Used in Design Storm Comparisons

Model	Citation	Method of Estimating Infiltration	Method of Constructing Outflow Hydrograph
SCS Triangular Hydrograph	Design of Small Dams (5)	Uses a "minimum infiltration rate" and runoff curve number based on soil type.	Relates incremental excess precipitation to incremental runoff with a hydrograph that is triangular in shape.
HEC-1	U.S. Army Corps of Engineers (16)	Uses an exponentially decaying function that depends on rainfall intensity and antecedent losses.	Derives outflow hydrograph from either (1) unitgraph input by either, or (2) Clark (17) synthetic unitgraph.
HYMO	Williams and Hann (18). U.S. Department of Agriculture.	Similar to SCS method above; uses curve number and minimum infiltration rate.	Uses dimensionless unitgraph (described by exponential expressions relating flowrate to time) and a "dimensionless shape parameter."
USGS	David R. Dawdy, John C. Shaake, Jr., and William M. Alley (19). U.S. Geological Survey.	Uses the Philip (20) variation of the Green-Ampt (21) equation. Method includes soil-moisture accounting between storms.	Performs finite difference solution of kinematic wave equation for each channel and overland flow segment in drainage basin.

TABLE 3 Results of Selected Statistical Analysis of Rainfall Characteristics

Linear Regression:

Dependent Variable	vs	Independent VAriable	Correlation Coefficient(R)	Conclusion
Pattern Index for all storms.		Duration of all storms.	.167	No significant relationship.
[a]Duration of all general storms-North Platte drainage.		Percent time to Peak Intensity-general storms-North Platte drainage.	.055	No significant relationship.
[a]Duration of all thunderstorms-North Platte drainage.		Percent time to Peak Intensity-thunderstorms-North Platte drainage.	.170	No significant relationship.

Analysis of Variance:

Null Hypothesis (H_o)	F Statistic Data	$F_{.05}$	$F_{.10}$	Conclusion
Pattern Index values for general storms are equal for all five major drainages.	1.22	2.44	1.99	Do not reject H_o; conclude no difference in Pattern Index due to drainage basin location.
Pattern Index values for thunderstorms are equal for three major drainages.	.79	3.14	2.38	Do not reject H_o; conclude no difference in Pattern Index due to drainage basin location.
[a]Pattern Index values are equal for thunderstorms and general storms-North Platte River drainage.	24.65	3.91	2.74	Reject H_o; conclude some difference in Pattern Index due to type of storm.

[a]Results from the North Platte drainage data analysis are presented as an example. Results from the other basins are similar.

This same finding is corroborated in the paper by Kerr et al. (7) for storms in Pennsylvania. It is precisely because of this diversity that the use of an enveloping set of curves is preferred to the use of a single storm pattern when attempting to predict runoff.

Presentation and Use of Design Curves

Figures 2 and 3 show the design curves for thunderstorms and general storms, respectively, constructed according to the procedures previously outlined.

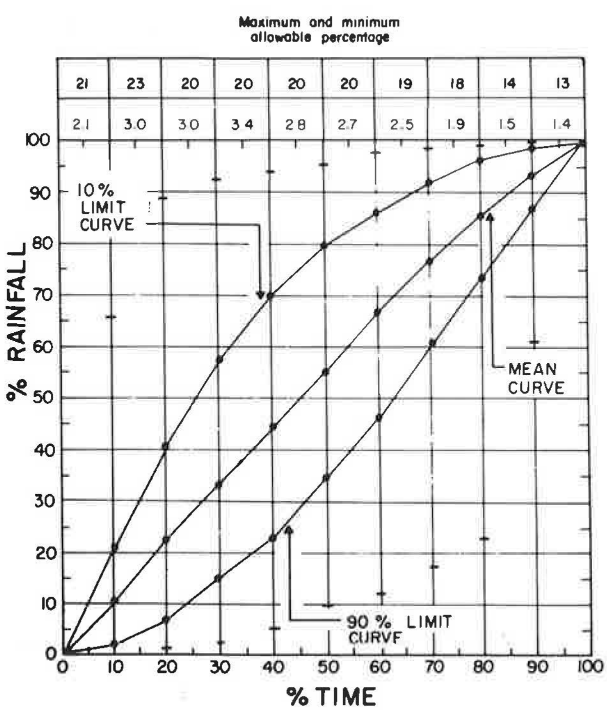

FIGURE 3 Dimensionless design mass curves for general storms in Wyoming.

Figure 2 is to be used when the duration of the design storm of interest is less than 4 hr. Figure 3 is to be used for events 4 hr long or longer.

Following is a list of steps involved in using the design curves:

1. Select the storm type to be simulated at a certain location; for example, the 10-yr, 6-hr event in Buffalo, Wyoming. Consult some source of rainfall frequency data, such as the Rainfall Frequency Atlas by Miller et al. (23), to find the volume of rain expected for this event.
2. Select the appropriate set of design curves. For the preceding example, the general storm curves (Figure 3) are applicable because the duration is longer than 4 hr.
3. Select one curve from the plot, either the 10- or 90-percent limit curve, the mean curve, or some nonstandard curve. When choosing a nonstandard curve, the user must remember to stay on or between the limit curves at all times. Also, the steepness (intensity) of a curve in any 10 percent time interval is dictated by the maximum and minimum allowable percentages shown at the top of the design curves. A nonstandard curve must not include more than the maximum percentage of storm volume indicated (maximum intensity), nor less than the minimum percentage of storm volume indicated (minimum intensity), in any given 10 percent interval of storm time. Examples of nonstandard time distributions are given in succeeding sections of this paper.
4. Using the curve from Step 3, select the percent rainfall values that correspond to the percent time values.
5. Organize the data obtained in Step 4 into the form required by whatever model is being used; that is, rainfall either as actual depth or a percent of storm volume, sequences either cumulative or incremental.
6. Run the model with infiltration and geomorphic soil, geologic, vegetative, or other basin parameters as required.

It is recommended that the user run several simulations with different hyetographs to determine the critical runoff volume or peak discharge. The suite of design curves used probably will include both limit curves, the mean curve, and several curves chosen arbitrarily by the user.

A parameter not included in this study is the areal distribution of rainfall. Therefore, the user of the method presented here is obliged to reduce point rainfall values when working with large drainage basins. Methods of reducing point rainfall with increasing drainage basin area are presented in Design of Small Dams (5) and in the Rainfall Frequency Atlas (23). These reductions are necessary because of the tendency of point rainfall values to overestimate actual areal precipitation on large areas.

Because this new design method depicts "probable" events, rather than extreme events (i.e., ultra-high-intensity bursts or long periods of very intense rain), it should not be used when designing for runoff due to "probable maximum" rainfall. Existing methods for probable maximum design (5) should be consulted for those cases.

RESULTS OF DESIGN STORM COMPARISONS

General Information

The purpose of this section is to compare the use of differing design storms in parametric flood prediction. Computer models used are HEC-1 (Hydrologic Engineering Center), HYMO (Problem-Oriented Computer Language for Hydrologic Modeling), HYDRO (SCS Triangular Hydrograph method), and USGS (U.S. Geological Survey-distributed routing model). The reader is referred to Table 2 for descriptions and references for these models. Design storms recommended by BUREC (5) and SCS (4) are used in the comparison.

The procedure followed in the comparison was to input differing design storms to a model, while leaving all geomorphic, soil, geologic, vegetative, infiltration, and other basin parameters unchanged, and examine differences in the simulated outflow hydrograph peak and volume. Variations thus found are attributable only to variations in the input hyetograph.

Some problems were encountered in the use of existing design storms. For example, the SCS method, rather than using a rainfall volume based on a certain duration for a given frequency, uses the 24-hr amount for designing storms of all durations. This practice results in slightly different storm volumes than those for varying durations found in Miller et al. (23) publication. Despite this anomaly, the SCS hyetograph was used without a volume correction. Thus, a valid method-by-method comparison is ensured. The BUREC method also involves an odd twist basing its storm volumes on fractions and multiples of the 6-hr value for a given frequency. Modern practice has corrected this deficiency by allowing the use of volumes expected for various durations, not a manipulation of the 6-hr amount, while retaining the recommended time sequence. The BUREC method also typically calls for basing designs on runoff from a 3-hr thunderstorm and an 18-hr general storm. Because there exists no 18-hr duration precipitation data, no storms of this length were used in comparison. Also, a 2-hr thunderstorm was deemed most representative of short duration events (thus, the 3-hr event was not used).

Storms selected for the comparisons were 2, 6, and 24 hr in duration. The 2-hr event is considered a thunderstorm; the other two are general storms. A small drainage (0.83 mile^2) in the Powder River Basin was the test basin used for the simulations. Storm volumes (6) for the durations listed earlier (with a 10-yr return period) at this location are: 2-hr = 1.60 in.; 6-hr = 2.00 in.; and 24-hr = 2.75 in. Runoff model parameters used with each model for comparison can be found in Tyrrell (22).

Design Hyetographs

Figures 4 and 5 show the dimensionless design hyetographs used for the thunderstorm and general storms as cumulative rainfall amounts. The WYO distribution sequences (mean, 10- and 90-percent limit) can be found in the curves shown in Figures 2 and 3. Those WYO storms designated A and B correspond to nonstandard curves arbitrarily selected by the authors using Figures 2 and 3. The data in Table 4 indicate the cumulative values for each design hyetograph for the 10-yr, 2-hr thunderstorm.

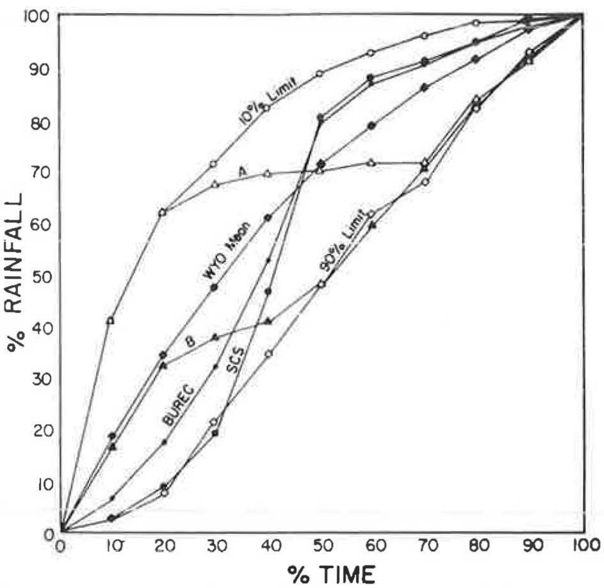

FIGURE 4 Dimensionless design mass curves (thunderstorms) for comparative purposes.

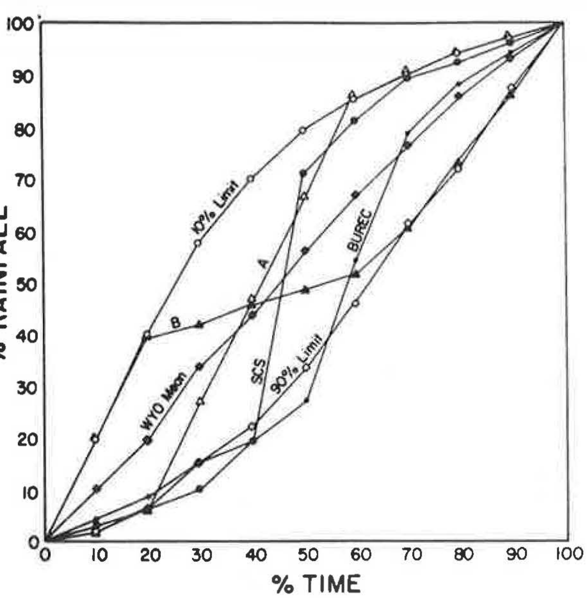

FIGURE 5 Dimensionless design mass curves (general storms) for comparative purposes.

TABLE 4 Comparative Hyetographs for 10 Year, 2-Hour Thunderstorm Cumulative Rainfall (inches)

Time, Minutes	[a]SCS Type II	BUREC	WYO: Mean	10% Limit	90% Limit	A	B
0	----	----	----	----	----	----	----
15	.06	.14	.35	.75	.06	.75	.35
30	.15	.36	.66	1.10	.24	1.02	.58
45	.45	.65	.91	1.30	.50	1.09	.64
60	1.17	1.26	1.14	1.44	.75	1.12	.75
75	1.30	1.39	1.30	1.50	1.01	1.15	1.01
90	1.37	1.49	1.42	1.55	1.25	1.25	1.25
105	1.43	1.55	1.52	1.58	1.44	1.44	1.44
120	1.47	1.60	1.60	1.60	1.60	1.60	1.60

[a]Based on 10 year, 24-hour volume (2.75")

Tables 5, 6, and 7 present the results of the runoff model runs for the 2-, 6-, and 24-hr events, respectively. Generally, results from HEC-1, HYMO, and HYDRO simulations indicate that for longer events, the WYO curves produce less runoff (peak and volume) than the other methods, while for shorter events, the WYO curves produce greater runoff. Results from USGS model runs differed from the other models' results by predicting, for all three storm durations, smaller runoff peaks and volumes due to the WYO design curves when compared with established procedures. Because of these results, it is suggested that current methods, in general, may lead to consistent over-design of hydraulic structures, at least when long duration (general storms) events are stated as part of the design criteria. Also, the ability of any one of the group of WYO curves to produce greater runoff than the others is dependent on the model used.

DISCUSSION OF RESULTS

The most significant difference between the WYO design storm methodology and those developed by SCS and BUREC is the use of totally dimensionless curves. By nondimensionalizing the time axis, the average intensities of designed storms are decreased as the storm durations are increased. For example, if two general storms of the same volume but differing durations, for example, 6 and 12 hr, were distributed over time according to the mean curve of Figure 3, the 12-hr storm would have one-half the intensity of the 6-hr event at any point along the curve. This explains why the WYO curves tend to produce smaller runoff peaks than the other methods for long events and larger peaks for short events. Such a change in intensity with duration may appear inappropriate at first, but analysis of 100 runoff-producing storms recorded by Rankl and Barker (12) indicates that,

TABLE 5 Runoff Characteristics for 10 Year, 2-Hour Thunderstorm

	MODEL:							
	HYDRO		HYMO		HEC-1		USGS	
Design Storm	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)
SCS Type II	47.8	.098	11.7	.036	38	.39	41.1	.162
BUREC	65.3	.137	17.3	.053	36	.38	40.2	.162
WYO-Mean	61.7	.139	12.9	.040	28	.31	16.0	.094
10% Limit	61.8	.123	19.9	.061	42	.45	33.2	.146
90% Limit	76.1	.135	30.7	.100	29	.32	20.6	.107
-A	62.2	.125	17.2	.064	34	.42	22.2	.138
-B	76.1	.135	30.7	.100	27	.32	19.2	.103

TABLE 6 Runoff Characteristics for 10 Year, 6-Hour General Storm

	MODEL:							
	HYDRO		HYMO		HEC-1		USGS	
Design Storm	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)
SCS Type II	85.3	.175	42.7	.143	36	.38	47.1	.184
BUREC	81.6	.251	37.6	.205	20	.23	19.4	.116
WYO-Mean	52.8	.275	18.9	.094	2	.03	6.7	.065
10% Limit	50.5	.208	26.9	.103	11	.14	8.5	.075
90% Limit	83.6	.287	54.8	.261	10	.12	12.4	.085
-A	89.1	.221	49.4	.164	18	.22	16.7	.101
-B	83.6	.226	55.8	.261	10	.16	10.5	.082

TABLE 7 Runoff Characteristics for 10 Year, 24-Hour General Storm

	HYDRO		HYMO		HEC-1		USGS	
Design Storm	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)	Peak (cfs)	Vol. (in.)
SCS Type II	138.6	.346	57.9	.285	30	.34	43.1	.189
BUREC	95.5	.268	45.9	.221	14	.16	14.4	.103
WYO-Mean	0	0	0	0	0	0	1.49	.043
10% Limit	24.3	.107	14.7	.091	0	0	2.22	.051
90% Limit	8.0	.085	6.5	.074	0	0	2.88	.056
-A	50.9	.384	36.6	.319	0	0	5.18	.069
-B	8.1	.075	6.1	.063	0	0	2.82	.056

although there is not a good linear relationship (R = 53 percent), the peak intensity of a storm appears to decrease with increasing storm length, as shown in Figure 6. It appears reasonable, therefore, for the WYO storm design techniques to make long storms generally less intense than short storms.

Lower rainfall intensity, as obtained from the different WYO curves, is the reason zero runoff is predicted in some instances for the 24-hr event. For example, referring to Table 7, no runoff is produced using the WYO mean curve with the HYDRO and HYMO models. Notice that, for general storms, the WYO mean curve is almost a 45-degree line indicating an almost constant intensity storm. For the 14-hr event, this constant intensity (0.11 in./hr) is less than the minimum infiltration loss of 0.15 in./hr. Thus, no runoff occurs. Similarly, the HEC-1 model produces zero runoff in several instances. Because shorter storms do produce runoff, according to HEC-1, the reason for zero predicted runoff in the longer storms obviously also involves low rainfall intensity and associated infiltration losses.

It is interesting to note that choosing a WYO curve for producing peak discharge or volume depends on the computer model to be used. For instance, referring to Table 5, the WYO 90 percent limit curve produces more runoff (peak and volume) than the 10 percent limit curve when HYDRO and HYMO are used. When HEC-1 is used, the 10 percent limit curve yields the greatest runoff peak and volume. The user of these curves is, therefore, warned not to assume that a peak-producing hyetograph for one model will perform similarly with a different simulation scheme. The user should always test several curves for their peak-producing ability when changing models, or when changing storm durations with the same model.

SUMMARY AND CONCLUSIONS

Summary

Parametric flood prediction on ungauged basins in Wyoming requires the use of temporal storm patterns that realistically represent anticipated local rainfall events. Because methods of hyetograph construction currently in use are very general in application, this requirement is not met. Therefore, a design storm methodology based on analysis of time distribution characteristics of 603 observed storms in Wyoming is presented. The WYO method of storm design uses not one but several mass rainfall curves, allowing flexibility of use and maximization of runoff from a given storm volume.

Comparisons were made between the WYO method and design storms recommended by SCS and BUREC using HEC-1, HYMO, HYDRO, and USGS distributed routing rainfall-runoff models.

Conclusions

1. The time distribution of both thunderstorms and general storms in Wyoming is not dependent on the drainage basin in which the storms occur.

2. The most outstanding characteristic of the storms analyzed is their individual diversity. No relationship exists between time distribution characteristics and duration of general storms or thunderstorms. However, a difference in the time distribution of thunderstorm rainfall, compared to general storm rainfall, exists.

3. One set of thunderstorm design curves and one set of general storm design curves can be used to

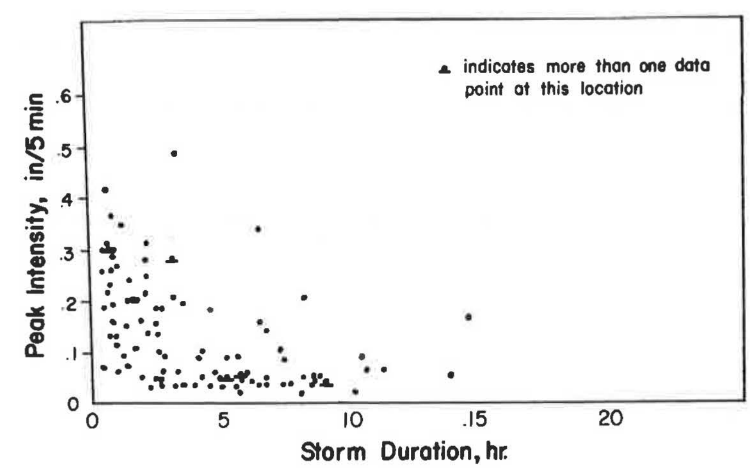

FIGURE 6 Variation in peak intensity with storm duration.

create design hyetographs for the entire state of Wyoming.

4. The WYO design storm methodology should not be used to design for probable maximum type events because the most intense rainfall values have been neglected by the definition of 10- and 90-percent limit curves.

5. Simulation of runoff peak and volume using WYO design curves is sensitive to storm duration and choice of runoff model.

6. WYO curves typically predict greater runoff peaks than SCS or BUREC synthetic hyetographs for short duration events, and less runoff, in most cases, for long duration events, according to HEC-1, HYMO, and HYDRO model results.

7. WYO curves consistently produce less runoff than SCS or BUREC synthetic hyetographs when the USGS distributed routing model is used.

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