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Hydraulic Characteristics of PVC Pipe in Sanitary Sewers (A Report of Field Measurements)

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INTRODUCTION

Since pipe made of poly vinyl chloride (PVC) is relatively new, its hydraulic characteristics that cause fluid friction head loss are not as well known as those of cast iron and concrete pipe. The lack of universally accepted roughness parameters for PVC pipe is attested by the fact that most textbooks dealing with hydraulics and fluid mechanics do not include such values. The hydraulic characteristics of PVC pipe have been measured in laboratory studies for both open channel flow conditions, Neale and Price (1), and for full flow conditions, Jeppson (2). However, no studies are known to exist which have determined these hydraulic characteristics of PVC pipe in the field. Furthermore, it is well known that the formation of scale can build up in steel, wrought, and cast iron pipe, so that after many years of service their hydraulic efficiency is significantly reduced. Whether PVC pipe is effected similarly is also not known.

To assist in answering some of these questions related to the hydraulic performance of PVC pipe, this study has made a number of field measurements of flows in sanitary sewer lines of PVC pipe. These measurements were made during 1975. Hopeful of obtaining additional measurements in other parts of the country the publication of the results has been delayed. These additional measurements have never been obtained, but since there have been a number of requests for the results, this report is being published, even though we wish the conclusions could be verified by additional field data from more geographically diverse areas, and in PVC sewer lines in service for varying periods, but as long of periods as possible. From these somewhat limited measurements, the hydraulic characteristics of these pipes have been determined.

The objectives of the study include the determination of these characteristics of newly installed pipe, but more importantly, these characteristics for pipes which have been in service for a number of years, to ascertain if any significant decrease has occurred in the carrying capacity of these PVC lines. Consequently, sewer lines were sought out which had been in service as long as possible. Because of the logistics of making the measurements within a limited budget, these lines had to be within the Intermountain West, however. They were all selected in the proximity of Denver, Colorado.

As a basis to which the results of the field measurements can be compared, the most applicable results from the aforementioned laboratory studies will be summarized. These pertinent conclusions are:

1. The coefficient for the empirical Hazen-Williams equation for PVC pipes is $C_{Hw} = 150$.

2. The coefficient for the empirical Manning equation is $n = 0.009$.

3. The equivalent sand roughness for use in determining the friction factor for the Darcy-Weisbach equation for PVC pipe is $e = 0.000,007$ ft (0.0021 mm). With this roughness PVC pipe is hydraulically smooth for Reynolds numbers, R_e, less than 3.5×10^5.

It should be noted that even though the Hazen-Williams

$$Q = 1.318\, C_{Hw}\, A\, R^{.63}\, S^{.54} \quad \text{(ES units)} \ldots\ldots \text{(1a)}$$

$$Q = 0.849\, C_{Hw}\, A\, R^{.63}\, S^{.54} \quad \text{(SI units)} \ldots\ldots \text{(1b)}$$

and Manning equation

$$Q = \frac{1.49}{n}\, A\, R^{2/3}\, S^{1/2} \quad \text{(ES units)} \ldots\ldots \text{(2a)}$$

$$Q = \frac{1}{n}\, A\, R^{2/3}\, S^{1/2} \quad \text{(SI units)} \ldots\ldots \text{(2b)}$$

are widely used, they cannot be defended on the same theoretical, basis as can the Darcy-Weisbach equation,

$$h_f = f\, \frac{L}{D}\, \frac{Q^2}{2g\, A^2} \ldots\ldots \text{(3)}$$

In the above equations Q is the volumetric flow rate; A is the cross sectional area of flow; R is the hydraulic radius and equals the area divided by the wetted perimeter, P; S is the slope of the energy line and equals hf/L, in which L is the length over which the head loss h_f occurs; g is the acceleration of gravity; and D is the pipe diameter.

The Hazen-Williams equation was devised principally for use with relatively smooth pipes flowing full and the Manning equation is intended for use in open channels or pipes flowing partly full, and has been developed from data obtained from channels with relatively rough walls. Despite this traditional use of the Hazen-Williams equation for full flow and the Manning equation for open channel flow, the Hazen-Williams equation can provide more accurate predictions of flow rate or head losses over a wide range of flow conditions than the Manning equation in small diameter sewer lines even though such flows are open channel flows.

The Darcy-Weisbach equation in the form of Equation 3 applies for full flowing pipes, but the same basic approach can be used for open channel flow, by replacing D by 4R and letting A be the actual cross-section area of flow. The following Chezy equation results from this modification of the Darcy-Weisbach equation.

$$V = C\sqrt{RS} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4a)$$

or

$$Q = CA\sqrt{RS} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4b)$$

in which the coefficient $C(R_e, e/R)$ is a function of Reynolds number $R_e = 4VR/\nu$ (ν = kinematic viscosity of fluid) and is related to the friction factor f by

$$C = \sqrt{8g/f} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

A task force of the American Society of Civil Engineers (3) has recommended the use of Equation 4 for open channel flow where appropriate. Its use seems particularly appropriate when dealing with PVC pipe under partly full conditions, because its wall material is much smoother than the materials in common open channel. The following implicit equation is used for determining the Chezy coefficient, C.

$$C = -32.1 \log \left(\frac{e/R}{12} + \frac{0.156\, C}{R_e}\right) \ldots\ldots\ldots\ldots (6)$$

FIELD MEASUREMENTS

Twenty-five separate measurements of flow conditions were made at four different localities. Table 1 summarizes the results from these measurements. The pipe lines included in these measurements varied from new installations to pipe lines which had been in service for 5 years. The difference in elevations Δz, in column 4 of Table 1 between the two ends of the pipe considered the test section, was determined by an engineers level using the pipe inverts at consecutive manholes. These, as well as the distances between manholes in column 5 of Table 1, agreed well with data obtained from drawing and design plans in the sanitary district offices.

The velocity data in column 6 of Table 1 were determined by current meters manufactured by Ott and Marsh McBirney (Model 201) which utilizes a small impeller and the electromagnetic principle, respectively. Good correlation exists between the velocities determined by both meters. However, the Marsh McBirney meter was less susceptible to fouling with tissue paper, etc., which was profusely present in the flows measured. Therefore, after duplicating velocity measurements with both meters at the first few sites the Marsh McBirney meter was used exclusively. In addition, measurements based on salt dilution methods, which utilized the increase in conductivity produced by a known added flow of salt water, were investigated. These measurements generally produced good correlation with the current meter measurements and showed consistency between the velocity computed from the time between which the salt solution was first added and a steady state increased conductance was measured downstream and the velocity computed from the flow rate determined from the increase in steady state conductivity. However, on at least one occasion the result from the salt dilution measurement was greatly affected by an extreme fluctuation in the conductivity of the sewer flow. Therefore, this method of flow measurement was not used at more than the first couple of sites.

Depth of flow was established by a point gage micrometer especially mounted for use in a manhole. Inside diameters were taken as the maximums established by subtracting twice the minimum wall thickness from the average outside diameter as established by ASTM standards.

ANALYSIS OF DATA

A primary objective of the field measurements is to determine the applicability of the commonly recommended value of Manning's n = 0.009, as well as the equivalent sand roughness e = 0.000 007 ft (0.000 21 cm) for PVC pipes. Table 2 summarizes the results of computations of Manning's n, Hazen-Williams C_{HW} and the equivalent sand roughness e, from Equations 1-6.

Reynolds numbers used to compute the equivalent sand roughness e were based on water at 60°F (15.6°C) with a kinematic viscosity of ν =

Table 1. Data obtained from field measurements.

Meas. Ident.	ID (inches)	Depth of Flow, Y (inches)	Diff., Δz (ft)	Length Between MH (ft)	Velocity (ft/sec)	Date 1975	Location	Comments
							Arvada, Colo. (Simms & 63rd)	Slime build up on entire wetted surface. Easily removed. Line had been in service 2 years.
1	10.078	2.89	2.49	294	4.30	6/11	MH#1 1st East of Union	
2	"	4.15	2.87	335	3.98	6/11	MH#1 @ Union	
3	"	3.83	2.87	335	4.70	6/11	MH#2 2nd East of Union	
4	"	3.20	3.20	350	3.90	6/11	MH#3 1st East of Simms	
5	"	2.50	3.16	350	5.04	6/13	MH#4 2nd East of Simms	
6	"	3.21	3.22	350	4.60	6/13	MH#5 3rd East of Simms	
7	"	2.38	3.16	350	4.69	6/13	MH#7 @ Simms	
							North Table Mt. (48th & Isabella)	Slime build up at water line only. Easily removed. 5 years in service.
8	7.90	0.50	2.16	180	1.86	7/25	MHB14.2 - B14.3 Isabella	
9	"	0.50	2.16	180	1.00	7/18	MHB14.2 - B14.3 Isabella	
10	"	1.98	1.44	360	2.22	7/18	MHB14.1 - B14.2 Isabella	
11	"	2.15	4.76	366	4.20	7/18	MHB14.3 - B14.4 Isabella	
12	"	2.32	5.85	325	5.04	7/25	MHB14.4 - B14.55 Isabella	
							Flora & 44th	
13	"	1.70	1.44	360	2.26	7/9	MHA3.41 - A3.4.2 Flora	
14	"	1.58	0.22	55	2.07	7/9	MHA3.4.2 - A3.4.3 Flora	
15	"	1.48	0.80	200	1.97	7/11	MHA3.4.3 - A3.4.4 Flora	
16	"	1.49	1.20	300	1.93	7/11	MHA3.4.4 - A3.4.5 Flora	
17	"	1.19	0.72	179	1.67	7/11	MHA3.4.5 - A3.4.6 Flora	
18	"	1.27	0.95	238	1.60	7/11	MHA3.4.6 - A3.4.7 Flora	
19	"	0.98	0.28	70	1.45	7/24	MHA3.4.7 - Flora	
							Arvada (Northridge)	No slime build up. 6 months in service.
20	"	1.90	13.43	328	4.85	7/24	MH 21-22 66th & Jay St.	
21	"	1.58	8.90	330	5.17	7/24	MH 20-19 Jay St.	
22	"	1.43	4.90	330	3.58	7/24	MH 19-18 67th & Jay St.	
23	"	1.36	15.90	355	5.88	7/25	MH 16-15 Ingalls Ct.	
24	"	1.38	12.88	356	5.98	7/25	MH 15-14 Ingalls Ct.	
25	"	1.33	2.39	355	2.22	7/25	MH 14-13 Ingalls Ct.	

Table 2. Roughness parameters computed from the field data in Table 1.

Meas. Ident.	Size (in.)	Area (ft^2)	Hyd. Radius, R (ft)	Flow Rate (ft^3/s)	Manning's "n"	Hazen-Williams C_{Hw}	Reynolds No. $R_e \times 10^{-5}$	Chezy "C"	Equ. Sand Roughness e (ft)	Froude No.
1	10	0.131	0.138	0.565	0.0085	149	2.97	126	0.00009	1.82
2	"	0.215	0.184	0.856	0.0112	115	2.75	100	0.0015	1.38
3	"	0.193	0.172	0.908	0.0091	141	3.24	122	0.0002	1.70
4	"	0.151	0.151	0.591	0.0103	123	2.69	105	0.0008	1.56
5	"	0.107	0.122	0.540	0.0069	182	3.48	152	Hyd. Smooth	2.31
6	"	0.152	0.151	0.698	0.0088	145	3.17	124	0.00014	1.84
7	"	0.100	0.117	0.468	0.0072	175	3.24	144	Hyd. Smooth	2.21
8	8	0.009	0.027	0.017	0.0079	149	1.01	104	0.00014	1.95
9	"	0.009	0.027	0.009	0.0142	80	0.54	56	0.0059	1.05
10	"	0.067	0.097	0.148	0.0089	145	1.20	113	0.00018	1.14
11	"	0.075	0.104	0.315	0.0089	139	2.27	114	0.00024	2.07
12	"	0.083	0.111	0.421	0.0091	134	2.73	113	0.00031	2.38
13	"	0.054	0.085	0.121	0.0080	160	1.22	123	Hyd. Smooth	1.26
14	"	0.048	0.079	0.100	0.0084	153	1.12	116	0.00007	1.20
15	"	0.044	0.075	0.087	0.0085	151	1.07	114	0.00011	1.18
16	"	0.045	0.075	0.086	0.0087	147	1.04	111	0.00016	1.16
17	"	0.032	0.062	0.054	0.0088	145	0.90	106	0.00022	1.13
18	"	0.035	0.065	0.057	0.0096	134	0.87	99	0.00049	1.04
19	"	0.024	0.051	0.035	0.0090	141	0.78	101	0.00030	1.08
20	"	0.063	0.093	0.305	0.0128	92	2.62	79	0.0040	2.55
21	"	0.048	0.079	0.251	0.0087	136	2.80	112	0.00025	3.00
22	"	0.042	0.073	0.150	0.0088	138	1.94	109	0.00027	2.19
23	"	0.039	0.069	0.230	0.0091	128	3.18	106	0.00038	3.70
24	"	0.040	0.070	0.236	0.0082	145	3.23	117	0.00012	3.73
25	"	0.038	0.068	0.086	0.0089	136	1.20	104	0.00036	1.41
				Av.	0.00907	139				
				Stan. Dev. σ	0.0012	23.1				

1.217×10^{-5} ft²/sec (1.131×10^{-6} m²/S = .01131 Stokes).

To assist in interpretation of these field data, the values of C have been converted to the friction factor f by Equation 5 and these f's, along with the Reynolds numbers have been plotted on a Moody diagram (see any fluid mechanics text for a Moody diagram). Also, superimposed on this Moody diagram are lines obtained by plotting the friction factors produced by the Hazen-Williams and Manning formula for selected values of these coefficients and hydraulic radii. These latter lines can be plotted best by algebraically manipulating the Hazen-Williams and Manning formula in the form of the Darcy-Weisbach formula Equation 3, and then identifying the quantity that replaces f. This manipulation of the Hazen-Williams, Equation 1a (using ES units) produces:

$$f = \frac{5.89\, g}{C_{Hw}^{1.852}\, R_e^{0.148}\, \nu^{0.148}\, R^{0.0185}} \quad \ldots\ldots\ldots\ldots (7a)$$

or for water at 60°F (15.6°C), and g = 32.2 ft²/sec.

$$f = \frac{1014.11}{C_{Hw}^{1.852}\, R_e^{0.148}\, R^{0.0185}} \quad \ldots\ldots\ldots\ldots (7b)$$

From the nature of Equations 7, one should note that if C_{Hw} and R_e are held constant, that it will plot as a straight line on log-log paper with f and R_e the variables being plotted. Therefore, lines for constant C_{Hw} and R plot as straight lines on a Moody diagram with a downward slope as those given. The hydraulic radius R has a relatively small influence on the position of the line.

The same manipulation of the Manning's Equation 2a produces,

$$f = \frac{3.622\, gn^2}{R^{1/3}} \quad \ldots\ldots\ldots\ldots (8a)$$

or for $g = 32.3$ ft²/sec,

$$f = \frac{116.64\, n^2}{R^{1/3}} \quad \ldots\ldots\ldots\ldots (8b)$$

Equations 8 do not indicate that f is a function of Reynolds number and, therefore, Equations 8 plot as horizontal lines on the Moody diagram, Figure 1. The influence of the hydraulic radius is much more pronounced in the case of Manning's equation than the Hazen-Williams equation.

Since the majority of the hydraulic radii of the field measured flows are in the range of 0.1, Figure 1 indicates that Manning's n from n = 0.008 to n = 0.001 enclose the bulk of the measurements. The range of Hazen-Williams C_{Hw} that encloses the bulk of the data is from C_{Hw} = 150 to C_{Hw} = 110. Table 2 gives an average value of n = 0.00907 and C_{Hw} = 139.

Figure 1, as well as Table 2, shows considerable variability in values of coefficients that describe the measured flow characteristics. This variability is not surprising, however, upon considering the circumstances of the measurements. The flows in these sewer lines were actually unsteady flows, since the lines had to accommodate the homes and businesses that they served. While considerable caution was exercised to insure that the measurements were made during periods of relatively small time dependent flow changes, and also that the depths and velocity measurements were made simultaneously, the undefined transient nature of these flows obviously can cause considerable scatter in computed coefficients. Furthermore, even if data could or had been collected for a steady flow condition, the flow in the test section would likely not be uniform. While only those sewers were used in which all indications were that good grade control existed during installation, these installations do not duplicate careful laboratory procedures that would control pipe elevations at each point to one-hundreth or even one-thousandths of a foot.

As an illustration of how lack of absolute grade control can effect the computation of the roughness coefficients, assume that in measurement number 20 the grade control allowed the pipe at the manholes where the measurements were made to be 0.2 ft (0.06 m) below the true grade, which in this example, is $\Delta z/L = 13.23/328 = 0.0405$. Furthermore, assume this 0.2 ft (0.06 m) deviation from true grade is the vertex of a parabola with is origin midway between manholes. Then letting z define the position of the pipe bottom, we get the equation,

$$z = 0.043384\, x - x^2/134480$$

If Manning's n for the pipe in measurement 20 is n = 0.009 instead of 0.0128 as Table 2 gives, then the normal depth for the flow rate Q = 0.305 cfs (0.00864 m³/s) is y_o = 0.1325 ft (0.0404 m). Solving the gradually varied flow equation starting midway between the manhole with the normal depth, gives a depth y = 0.1487 ft = 1.8 inches at the manhole where we assumed a 0.2 ft (0.06 m) drop from true grade. This gradually varied flow is based on the solution of the equation,

$$\frac{dy}{dx} = \frac{S_o - S_f}{1 - F_r^2} \quad \ldots\ldots\ldots\ldots (9)$$

in which $S_o = -dz/dx = 0.043384 - x/67240.$, S_f is the slope of the energy line based on Manning's equation with n = 0.009 and F_r^2 is the Froude number squared or Q^2T/gA^3. The 1.8 inches (4.57 cm) is not too different from the measured 1.9 inches (4.83 cm), therefore, even for measurement

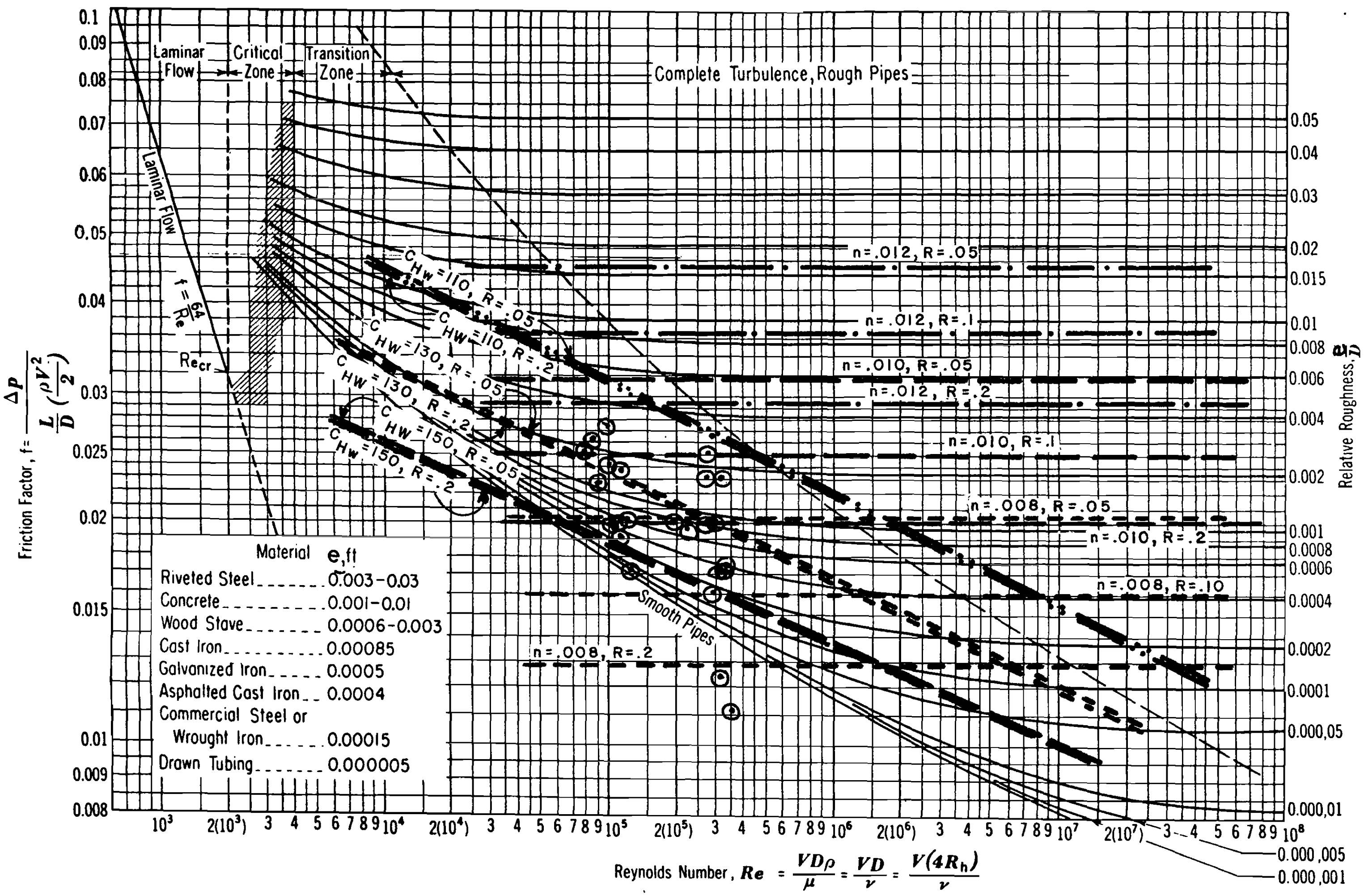

Figure 1. Friction factors for commercial pipe.

20 the actual Manning n could easily be 0.009 instead of 0.0128.

Since a deviation from true grade of only 0.2 ft (0.06 m) represents good grade control, it is not surprising to have the measured results scatter as shown in Figure 1. Measurement number 9 also produces roughness coefficients which appear to be in error. However, it should be noted in the case of this measurement the flow is very close to critical depth, and the depth of flow varies considerably with minor changes near critical depth.

A sensitivity analysis using Manning's equation reveals that the measured parameters have the following influence on Manning's n.

Parameter	% Variation in Parameter	% Variation in n	Ratio of Variations	% Variation in e	Ratio of Variation
y	5	3.64	0.728	19.7	3.94
S	5	2.73	0.546	18.9	3.78
V	5	4.55	0.910	97.6	19.52

These variations are for D = 8.0 inches, y = 2.4 inches, V = 2.0 ft/sec, $R_e = 2 \times 10^5$, and S = 0.004, and would be different at other values.

The sensitivity analysis indicates that the error in the determination of Manning's n will be slightly less than the error in the measurements of y, S, or V. On the other hand, any experimental error in determining y, S, or V will be amplified by from 4 to 20 times as much in the determination of the equivalent sand roughness, e. Consequently, an accurate evaluation of e requires very great precision in measurements of the flow characteristics. Conversely, when using the Chezy equation for flow computations a larger range of acceptable values of e exists for a given accuracy in determining flow rate for a given pipe. In other words a ten fold variation in e from say 0.0001 ft (3.5×10^{-5} m) to 0.001 ft (3.5×10^{-4} m) may result in only a ratio of predicted flow rates of 0.75, whereas a ten fold variation in n causes a ratio of 0.1 in flow rates. This fact illustrates another advantage of using equations based on friction theory for use in flow computations. It should also be noted that out-of-roundness, or ellipticity, has only a minor effect in reducing the cross-sectional areas. For a 5 percent out-of-roundness, the reduction in area is less than 1 percent.

CONCLUSIONS

The results of the 25 field measurements taken from four different localities verify that the value of Manning's n should be taken equal to 0.009 for PVC pipe. This value is in agreement with the value determined in laboratory measurements. No distinguishable difference could be observed by eye between new PVC pipe or such pipe which has been in service for 5 years. The field test also indicates that for common velocities encountered in sanitary sewers ($R_e < 3 \times 10^{-5}$), that PVC pipe might be considered to have an equivalent sand roughness of approximately 0.0005 ft (1.52×10^{-4} m) when using the Chezy equation for flow computations. When using the Hazen-Williams formula, an appropriate coefficient is 140. This value is slightly less than that determined from laboratory tests.

The field tests confirm the fact, which is known from fluid friction theory, and which has been observed in laboratory tests, that as the flow increases (i.e. depth of flow, flow rate, and velocity) the Manning's n coefficient actually decreases. This fact is evident by examining the variation of the Darcy-Weisbach friction factor f or the Chezy coefficient C as a function of Reynolds number, even over the small range of Reynolds number of these measured flows. This decrease in n does not imply that pipe walls become smoother at larger flows. Rather it points out that Manning's n should not be considered independent of flow conditions. It adds additional credence for using formulas such as the Darcy-Weisbach and Chezy equations with coefficients dependent on the relative roughness of the wall material and the Reynolds number of the flow.

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