Field measurements of the hydraulic resistance of sanitary sewers[1]

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Measurements of the in-service hydraulic roughness of some 33 lengths of small suburban sanitary sewers are reported. The test results indicate that the hydraulic roughness used in design should vary with pipe slope, material, and the proportion of the design flow that is dry-weather flow. In particular, the typical design roughness currently recommended seems low for low slope sewers, but high for those with slopes greater than about 1%. The tests revealed that sewers installed below the usual recommended minimum slope were operating as successfully as those of steeper slope, which implies that the current minimum-slope recommendations may be too conservative. An alternate approach to minimum-slope specification is suggested. Several augmented-flow tests were also carried out. These confirmed that the variation in effective roughness with depth is much stronger than is usually assumed. They also showed that there is a significant hysteresis in the roughness variation on the rising and falling limbs of a hydrograph.

Key words: sewers, pipeflow, coefficients, roughness, resistance, hydraulics, wastewater, sanitary.

Cet article traite des mesures de la rugosité hydraulique de quelque 33 longueurs d'égout domestique. Les résultats indiquent que la rugosité hydraulique utilisée dans la conception varie selon la pente du tuyau, le matériau et la proportion du débit de calcul qui est en fait un débit d'étiage. La rugosité de calcul type actuellement recommandée semble faible dans le cas des égouts à faible pente et élevée dans le cas de ceux dont la pente est supérieure à 1%. Les essais effectués ont révélé que les égouts dont la pente est inférieure au niveau minimum recommandé fonctionnaient aussi bien que ceux dont la pente est plus abrupte. Ces résultats semblent indiquer que les recommandations relatives à la pente minimale seraient trop prudentes. Une solution de rechange concernant la pente minimale est proposée. Plusieurs essais avec débit accru ont également été effectués. Ils ont confirmé que la variation entre la rugosité réelle et la profondeur était plus forte que celle normalement présumée. Ils ont en outre démontré qu'il existe une importante hystérésis au niveau de la variation de la rugosité sur les branches montante et descendante de l'hydrographe.

Mots clés: égouts, coefficients, rugosité, résistance, hydraulique, eaux usées, sanitaire.

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Introduction

Standard sanitary sewer design involves selection of a pipe diameter sufficient to carry the design peak flow, without surcharging, on a slope sufficiently steep that a minimum "scouring" velocity exists when the sewer is flowing full or half-full. This scouring velocity is usually taken as 0.6 m/s, a figure that appears to come from recommendations made by the Boston Society of Civil Engineers in 1942 (Committee to Study Limiting Velocities of Flow in Sewers 1942).

Determination of the necessary diameter and minimum slope therefore requires selection of an appropriate flow resistance coefficient for the sewer. Again following recommendations of the Boston Society, a common choice in North America is a Manning n of 0.013. This value and the associated variation of n with flow depth generally quoted are consistent with what was found in laboratory tests on the resistance to flow in part-full pipes by Yarnell and Woodward (1920) and Wilcox (1924). These were therefore likely the basis of the recommendations, though the tests were for clean water flowing through new pipe, not in-service sewers. More recently, Perkins and Gardiner (1982, 1985) reported on tests of sanitary sewers in a quasi-field situation. These tests confirm what simple inspection of an in-service sanitary sewer indicates: slime growth over the wetted perimeter causes a dramatic difference between the in-service flow resistance of sanitary sewers and that of clean water flow in new pipes. These observations suggest that reappraisal of sanitary sewer design resistance coefficients would be appropriate and, indeed, new recommendations have been developed in the United Kingdom based on the work of Perkins and Gardiner.

Nevertheless, relatively few reliable measurements of the flow resistance of truly in-service sanitary sewers seem to have been reported in the literature. Of those known to the writers, none were for the smaller diameter sewers that make up by far the majority of sewers laid, and none were for the newer widely utilized pipe materials such as PVC. Field measurements of the flow resistance of in-service sanitary sewers were therefore undertaken to aid in the development of new guidelines in Alberta. Because the most common materials currently used for new suburban sanitary sewers in Alberta are PVC and concrete, attention was confined to these materials.

Field measurements

All the sewers tested were between 2 and 5 years old. The diameters ranged from 200 to 686 mm, and the slopes from 0.027 to 3.12%. Only sites where close-to-uniform flow could be expected were investigated. Most of the study sites were in the Edmonton area, but, to include sewers with substantial normal (dry-weather) flows, tests were carried out on some sewer lengths in Banff and Red Deer. In all, some 189 resistance measurements were made for normal flows in 22 manhole-to-manhole lengths of PVC sewer and 66 measurements in 9 similar lengths of concrete sewer.

Because most of the tested sewers had low flows under normal circumstances, augmented-flow tests were undertaken at selected sites to determine the resistance under conditions more representative of currently assumed design conditions. For these augmented flows, 121 resistance measurements were

NOTE: Written discussion of this paper is welcomed and will be received by the Editor until April 30, 1990 (address inside front cover).

[1]This paper was presented at the 1988 Annual Conference of the Canadian Society for Civil Engineering, Calgary, Alta.

TABLE 1. Summary of hydraulic roughness measurements for normal (dry-weather) flows in PVC sanitary sewers

Location	Nominal diameter (mm)	Length (m)	Slope (%)	Average Q (L/s)	Average V (m/s)	Average d/D	Boundary shear, τ_0 (Pa)	Average n	Average k_s (mm)	Standard deviation of k_s (mm)	No. of measurements
Thorndale (industrial)											
MH 19-17	300	60.1	0.275	2.29	0.313	0.129	0.65	0.016	3.8	0.8	6
MH 17-15 May		58.3	0.393	1.88	0.320	0.108	0.79	0.016	4.7	0.5	6
August				0.82	0.251	0.069	0.52	0.016	3.8	0.7	3
MH 15-12		109.5	0.356	1.38	0.274	0.105	0.70	0.017	6.2	2	6
MH 19-15		118.3	0.333	1.03	0.219	0.090	0.56	0.020	9.7	3	6
Stony Plain											
MH 103-102 1984	250	49.5	0.180	0.82	0.215	0.128	0.35	0.015	2.6	0.5	13
1985				1.28	0.254	0.158	0.43	0.014	2.2	0.1	3
1985				4.33	0.393	0.273		0.013	1.2	0.1	3
MH 102-101 1984		93.9	0.212	0.712	0.208	0.120	0.39	0.016	3.5	0.5	7
1985				1.21	0.184	0.189	0.60	0.023	19	8	3
MH 103-101 1984		143.3	0.200	0.916	0.194	0.151	0.46	0.019	8.8	3	7
1985				0.787	0.176	0.146	0.44	0.020	1.1	3	3
Devon											
MH 49-48	300	78.3	0.148	1.40	0.190	0.111	0.30	0.018	7.5	2.9	7
Riverbend											
MH 66-65	200	92.5	1.41	0.350	0.461	0.057	1.02	0.010	0.19	0.1	6
MH 22-23 1983	300	53.9	0.340	1.39	0.279	0.100	0.64	0.017	5.1	3	7
MH 26-25 1983	200	119.8	3.12	1.38	1.01	0.087	3.5	0.0088	0.11	0.1	6
1985a				1.95	1.16	0.096	3.7	0.008	0.03	0.09	3
1985b				2.84	1.26	0.123	4.7	0.009	0.07	0.02	4
MH 23-22 1983	300	53.9	0.340	1.26	0.243	0.126	0.79	0.020	10.2	4	7
1985				4.49	0.426	0.207	1.24	0.015	3.8	0.9	6
Burntwood											
MH 303-302	200	67.1	0.570	4.79	0.432	0.380	2.3	0.021	15.5	5	9
Leduc Southpark											
MH 614-613	200	94.7	0.415	0.571	0.179	0.156	0.78	0.026	23	11	7
Leduc Romulus											
MH 10-10A	250	74.6	0.233	0.484	0.086	0.127	0.45	0.048	85	46	8
MH 9-10		71.0	0.357	0.415	0.141	0.082	0.45	0.027	22	11	9
MH S11-S12		81.0	0.354	0.660	0.247	0.102	1.04	0.015	3.2	0.3	3
Spruce Grove											
MH 124-123	200	68.7	0.568	0.480	0.274	0.125	0.87	0.017	5.4	2.4	7
Banff, 1985											
MH 39-40	366	100.7	0.300	24.0	0.588	0.410	2.3	0.017	8	1.3	8
MH 41-42		65.6	0.094	25.3	0.431	0.545	0.89	0.015	3.7	0.6	7
MH 42-44		67.6	0.250	22.5	0.394	0.534	2.3	0.027	50	6	6
MH 51-52		71.0	0.220	24.3	0.518	0.455	1.89	0.018	8.9	1.4	6
MH 58-59		39.9	0.220	19.0	0.493	0.393	1.66	0.017	8.1	2.4	6
Red Deer, 1985											
MH 13-12	380	72.6	0.150	19.0	0.290	0.602	1.55	0.029	73	37	4

made on 3 manhole-to-manhole lengths of PVC sewer and 68 measurements on 3 similar lengths of concrete sewer.

The measurements during each test consisted of (1) discharge measurement using continuous-injection fluorescent-tracer (Rhodamine WT) dilution measurements over the sewer length between manholes; (2) velocity measurement by timing passage of a salt slug over the length between manholes; and (3) measurement of the elevation of the sewer invert at each manhole to determine the slope. The first two measurements, with the nominal diameter, allow direct determination of the average hydraulic geometry over the length of the sewer between manholes. As a rough check on the results, flow depths were measured at each manhole and the nature of the deposits, if any, in the sewer invert were noted. Because of safety considerations, an important feature of the test procedure was that measurements could be carried out without entering the manholes. However, for six sites, special arrangements were made for entry to the manholes under City supervision to document the nature of the slime deposits on the pipe walls.

At each site, measurements were made for the normal flow existing at the time. Then, for the augmented-flow sites, tests were conducted using water added from a fire hydrant to a manhole located one or more manholes upstream of the test section. The flow in the sewer was increased in steps until the maximum hydrant capacity was reached, and then reduced in steps back to normal flow. After each step change in discharge, a period of 15–20 min was allowed for steady flow to be reestablished, and then at least three sets of measurements were

TABLE 2. Summary of hydraulic roughness measurements for normal (dry-weather) flows in concrete sanitary sewer lines

Location	Nominal diameter (mm)	Length (m)	Slope (%)	Average Q (L/s)	Average V (m/s)	Average d/D	Boundary shear, τ_0 (Pa)	Average n	Average k_s (mm)	Standard deviation of k_s (mm)	No. of measurements
Yellowbird — 17 Ave. MH 315–317	300	54.9	0.306	2.78	0.080	0.163	0.90	0.015	3.6	1.1	10
				9.32	0.512	0.298	1.53	0.015	3.6	0.5	3
Yellowbird — 16 Ave. MH 341–342	200	59.7	0.500	0.136	0.080	0.101	0.620	0.051	54	18	6
Yellowbird — 107 St. MH 350–348	300	81.3	0.331	3.94	0.339	0.216	1.26	0.020	11.8	1.3	10
Lake District — 95 St. MH 204A–204	380	61.2	0.027	8.62	0.161	0.477	0.24	0.021	22	12	6
St. Albert LaRose Dr. MH 100–101	250	77.3	0.379	2.40	0.362	0.167	0.95	0.016	4.3	0.7	9
St. Albert McKenney Ave. MH 17–16	200	64.1	0.941	0.87	0.337	0.131	1.50	0.019	7.3	2.2	10
Banff SVA55–SVB26	686	151	0.150	114	0.539	0.558	2.70	0.023	38	3.4	6
124 Ave. and 43 St. MH R6–R7	300	107	0.309	4.46	0.334	0.216	1.17	0.020	13.5	0.5	3
69 Ave. and 42 St. MH A14–A13	300	122	0.484	3.31	0.473	0.152	1.34	0.014	2.1	0.1	3

taken. As each measurement set involved taking a sample at the downstream manhole for tracer concentration determination, injection of a salt slug at the upstream manhole and recording of its passage at the downstream manhole, and depth measurements at both manholes, each flow increment was maintained for a further 15–30 min.

The average hydraulic radius of the waterway over the length between manholes was calculated from the average waterway area determined from the ratio of measured discharge to measured velocity, the properties of a circle, and the nominal pipe diameter. The "sand grain" hydraulic roughness, k_s, was then determined using the Colebrook–White equation.[2] Although not totally appropriate to sewer design, values of Manning n were also calculated from the relation $n \simeq 0.04\, k^{1/6}$ for comparison with existing standards. These n values were not "depth corrected." The results of the measurements are summarized in Tables 1 and 2 for normal flows in PVC and concrete sewers respectively. Table 3 gives the results for the augmented-flow tests at the maximum flow depth attained. The agreement of results for the sewer lengths that were tested twice is notable and gives some indication of the consistency of the measurement techniques. In most cases the repeat tests were done approximately 1 year later, by different personnel. Detailed descriptions of the test methods, and more details of the results, are given by Gerard *et al.* (1986).

The nature of the slime deposits around the sewer perimeter was documented at some of the sites tested. The typical slime distribution found is shown in Figs. 1 and 2. In most of the sewers inspected, no slime was evident in the invert, presumably because of abrasion by grit. The distribution is similar to the deposits described by Perkins and Gardiner (1982, 1985), except for the clean invert. The difference in this latter regard is likely because the wastewater used by Perkins and Gardiner had received primary treatment that would have removed much of the grit. Although the pattern was similar from site to site, the thickness and hardness of the slime deposits varied. The harder thinner slime was at levels within the daily water level fluctuations; the softer slime was found above this. The overall hardness of the slime varied. The softest could be removed with a finger with some difficulty; the hardest could only be removed with a pocket knife. The hardness seemed to depend on the age of the sewer — the older the sewer, the harder the deposit.

Discussion

In the Perkins and Gardiner (1982) tests, wastewater was run through specially constructed test lines for almost a year. The discharge was varied over each day to simulate typical daily hydrographs of dry-weather sanitary sewer flow. Periodically the flow was augmented to carry out hydraulic roughness measurements with a full pipe.

The roughness was found to vary systematically over the year owing to variations with time of slime deposits that formed around the perimeter. Typical augmented, full-pipe flow results for PVC and concrete are reproduced in Fig. 3. The conduit diameters were 231 and 222 mm respectively. The general difference in the measured roughness between the two materials was due to differences in the slime growth on the two materials, which was found by these investigators and others to be worse for rough substrates like concrete than for smooth ones like PVC. The median values for the PVC and concrete were 0.6 mm (Manning $n = 0.012$) and 2.3 mm ($n = 0.015$) respectively. On the other hand, the maximum *part-full* roughnesses, which typically occurred for a depth about the maximum daily flow depth (about half diameter), were 3.0 mm ($n = 0.015$) and 7.0 mm ($n = 0.017$) respectively. The comparable clean-water

[2]This equation is rearranged into a more meaningful and convenient form in Appendix 2, and justification is provided for emphasizing the hydraulic roughness rather than the more traditional Manning n. Note that all roughnesses given in this paper are based on the original or nominal pipe diameter. Hence the quoted roughness includes the effect of diameter reduction by slime growth and other deposits.

TABLE 3. Summary of hydraulic roughness measurements for maximum augmented flows in PVC and concrete sanitary sewers

Location	Nominal diameter (mm)	Slope (%)	Length between manholes (m)	Average Q (L/s)	Average V (m/s)	Maximum d/D	n	k_s (mm)	Standard deviation of k_s (mm)	Total No. of measurements
ſhorndale Industrial* MH 17–15 (PVC)	300	0.393	58.28	38.55	0.921	0.58	0.013	1.3	0.1	30
ˌeduc Romulus† MH S11–S12 (PVC)	250	0.354	80.99	18.09	0.841	0.45	0.011	0.3	0.05	14
ˌtony Plain 1984 MH 103–102 (PVC)	250	0.180	49.47	25.54	0.647	0.74	0.012	0.7	0.09	17
1985‡				27.1	0.646	0.79	0.012	0.7	0.07	24
	Cycle 1			29.7	0.623	0.93	0.012	0.75	0.14	18
	Cycle 2			28.7	0.622	0.89	0.012	0.86	0.26	18
24 Ave. and 43 St. MH R6–R7 (Concrete)	300	0.309	107.25	24.53	0.680	0.50	0.015	3.2	0.1	20
ˌ9 Ave. and 42 St. MH A14–A13 (Concrete)	300	0.484	122.22	28.94	0.993	0.42	0.012	0.7	0.09	27
ˈellowbird — 17 Ave. MH 315–317 (Concrete)	300	0.306	54.86	25.66	0.705	0.50	0.014	2.5	0.31	21

*The minimum rougness did not occur at the maximum d/D for the Thorndale Industrial site. The minimum roughness, k_s = 0.58 mm and Manning n = 0.0113, ccurred at d/D = 0.26.
†At the Leduc Romulus site, flow tests were only conducted up to the maximum d/D. The flow was not brought back down to its original level ecause the downstream manhole flooded out.
‡Two-cycle test.

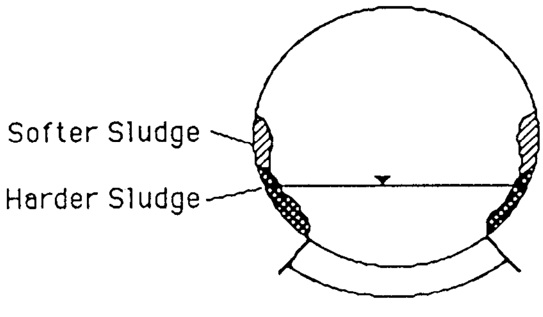

FIG. 1. Typical slime distribution.


FIG. 2. Example of normal flow and the characteristic slime deposits — Riverbend MH23.

ıew-pipe roughnesses for these two materials were measured to ɔe about 0.07 mm (n = 0.009) for PVC and 0.14 mm (n =).010) for concrete.

In the Alberta tests the hydraulic roughnesses for normal 'lows, for which the depths were usually less than half diameter, were generally substantially higher and showed a much greater variation. The results are shown in Fig. 4. The apparent roughness determined for these low flows in small sewers would likely be a function of (1) the boundary roughness associated with the slime and (or) the pipe material; and (2) profile irregularities due to poor construction, postconstruction settlement, joint eccentricities, and other bulk obstructions to flow, like rags and other detritus, that find their way into sanitary sewers.

The Perkins and Gardiner results give information on the first item. However, in the inspected Alberta sewers, the inverts were slime free, unlike those in the Perkins and Gardiner tests. The only significant slime growth in the inspected Alberta sewers was near the normal flow waterline, a zone of low shear and velocity for normal flows. Therefore, for normal flows the influence of these deposits on the effective waterway roughness would be slight.

A preliminary assessment of the influence of profile irregularities was made analytically. The effect on the apparent roughness caused by a sag (taken as a parabolic deviation from a straight line profile) of an 80 m length of 250 mm sewer free of slime (k_s = 0.08 mm) was estimated. The effect could be substantial. For example, for a flow depth of 0.3D (where D is the pipe diameter) and a maximum sag of 20% of the pipe diameter, the apparent roughness was about 25 times the actual roughness (an increase in the Manning n of 40%) for a manhole-to-manhole slope of 0.05%, and 13 times (or 26% increase in n) for 0.1% slope. At 0.8 relative depth, the factor was about 3 (or 8% for n) for both slopes. Although this effect should decrease with an increase in roughness, it was evident that the effect of profile irregularities on the apparent hydraulic roughness can be large for low slopes and is very sensitive to flow depth, sag, and sewer slope. Hence, profile irregularities may be the simple explanation for at least a portion of the large

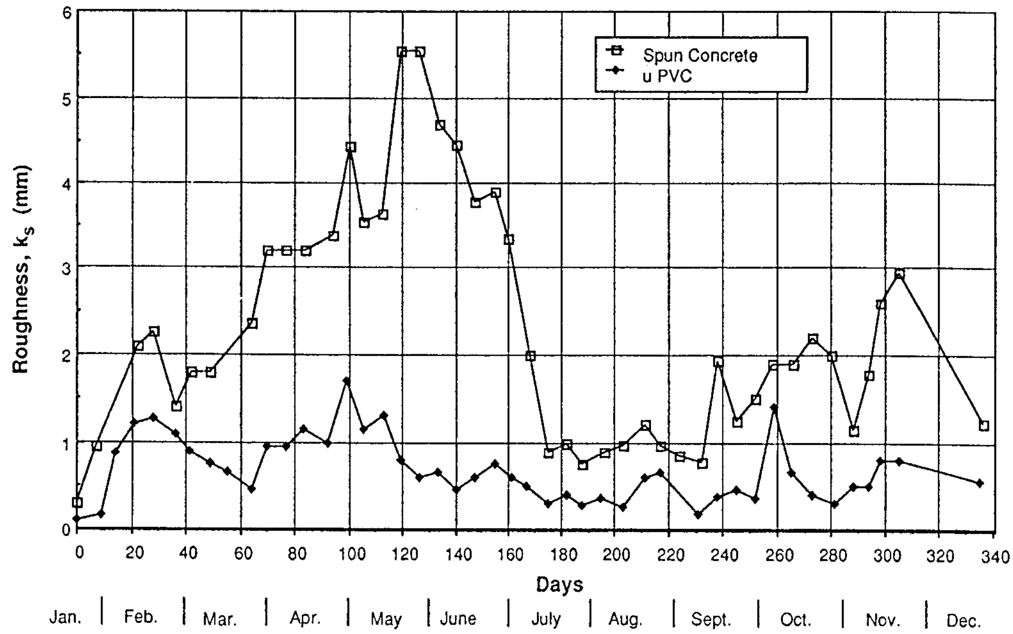

FIG. 3. Variation of pipefull hydraulic roughness with time for PVC and concrete sanitary sewers (after Perkins and Gardiner, 1982).

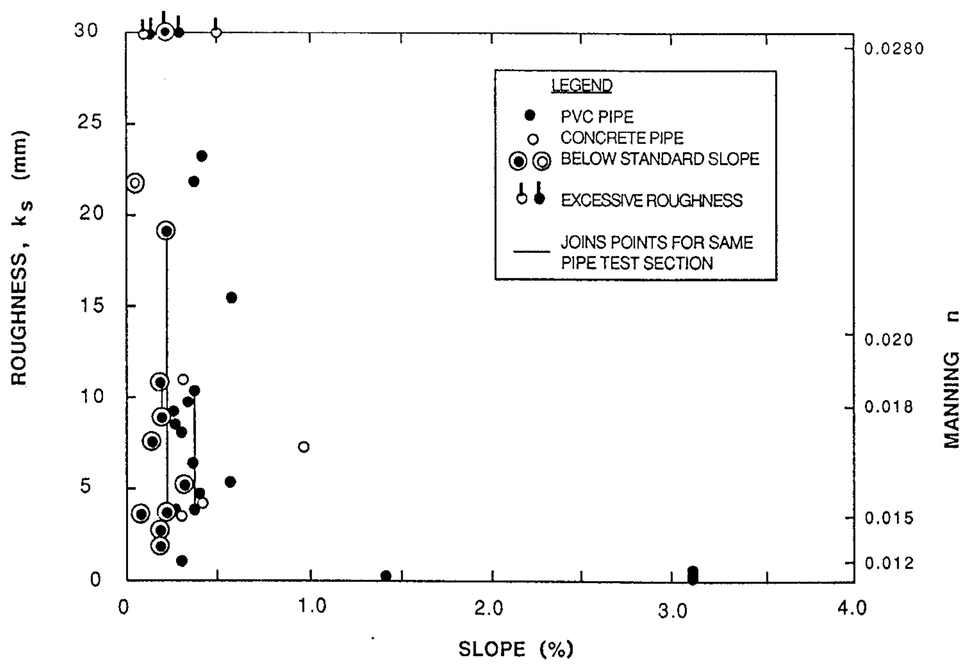

FIG. 4. Measured hydraulic roughness for normal (dry-weather) flows in sanitary sewers.

apparent roughness, and its variation, found for low relative depths.

From the above discussion, it would seem the apparent roughness for normal flows in the present tests may be due more to profile irregularities than to slime growth. Consequently, the apparent roughness should be quite variable at low slopes and decrease to near "as new" values as the sewer slope increases. This trend is evident in Fig. 4. The two sewers with slopes greater than 1% displayed as-new roughnesses, with one of these (PVC) having a roughness of about 0.07 mm (n = 0.009) in tests repeated at different times over some 18 months.

To try and isolate a reasonably homogeneous set of results for normal flow on low slopes, free of obvious "abnormalities," only those measurements for which k_s < 25 mm, slope < 1%, and d/D > 0.1 and with no sediment or other obstruction noted at the manholes were considered. This filtering left 11 PVC and 7 concrete reaches. For these sections, the median hydraulic roughness was essentially the same for the two materials — 7 mm (n = 0.017). No trend with other parameters such as slope, diameter, or relative depth was evident within the above bounds. This value is surprisingly similar to the "normal" flow values of 3.0 and 7.0 mm found for PVC and concrete, respectively, by Perkins and Gardiner. Possibly, the influence of the lack of slime in the inverts of the Alberta sewers was compensated by the effect of the profile irregularities, which would not have been present in the Perkins and Gardiner tests.

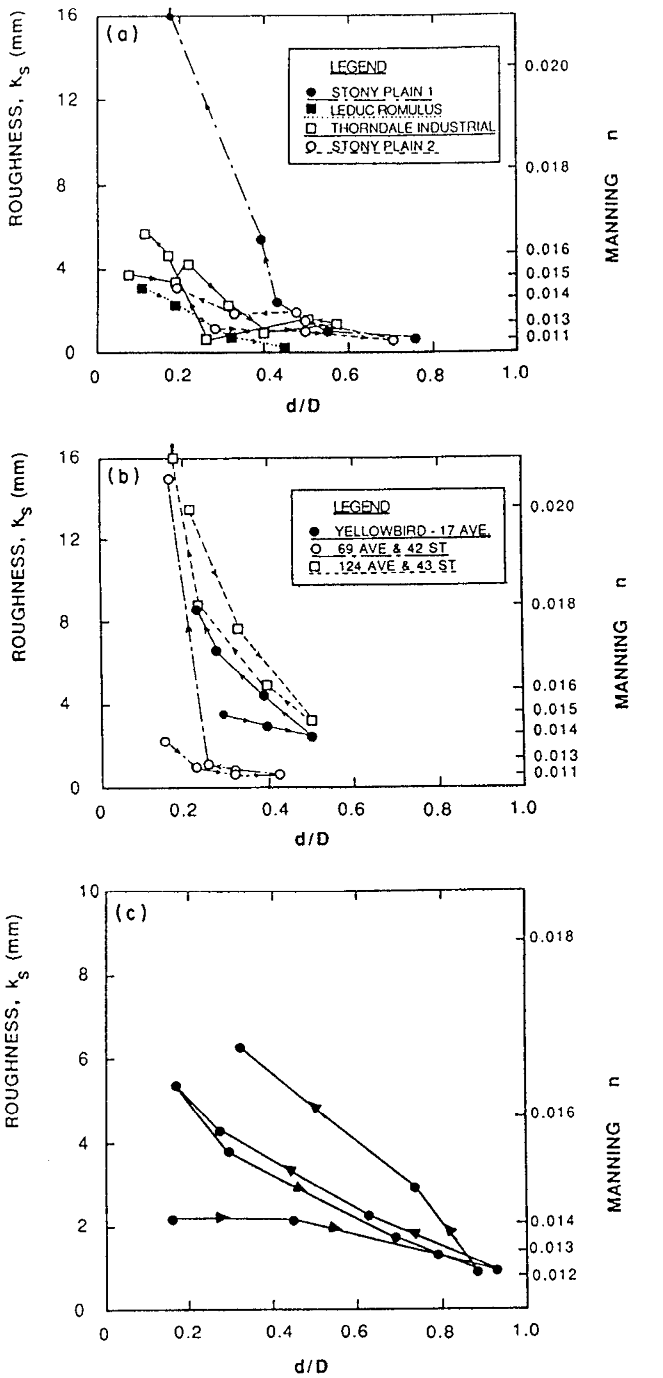

FIG. 5. Measured variation of hydraulic roughness with relative depth for augmented flows: (*a*) PVC sewers; (*b*) concrete sewers; and (*c*) two-cycle augmented-flow test (Stony Plain).

There are several other aspects of the Alberta tests worthy of comment. The first is the roughness variation with relative depth measured during the augmented-flow tests, shown in Fig. 5. This is a far different, and very much stronger, variation than that usually quoted in texts and design manuals. A similar result was found by Perkins and Gardiner. It is presumably due to the roughness variation around the perimeter caused by the slime deposits and, in the Alberta tests, the reduction in the influence of profile irregularities at increasing relative depth.

The effect of roughness variation around the perimeter can be estimated roughly by using the Horton-type relation (Chow 1959):

$$k = [(P_d k_d^{1/4} + P_c k_c^{1/4})/P]^4$$

or, in terms of Manning n,

$$n = [(P_d n_d^{3/2} + P_c n_c^{3/2})/P]^{2/3}$$

where P, k, and n are the wetted perimeter, the composite hydraulic roughness, and Manning coefficient, while the subscripts d and c indicate the parameters for the dry-weather and clean portions of the augmented-flow perimeter respectively.

For example, Table 4 and Fig. 6 show the comparison between the composite roughness so calculated for half-full flow for the six augmented-flow test situations, and the value of the half-full composite roughness measured on the rising limb. Evidently, the augmented-flow results are more or less what would be expected from the normal flow results, at least on the first rising limb of the cycle. The remaining disparity in Fig. 6 is likely due to variation in the influence of profile irregularities, as well as the approximations inherent in the above analysis.

Because of the similar ratio of augmented to normal flow depths, the half-full measurements should be comparable with the pipeful roughness measurements by Perkins and Gardiner. In the Alberta augmented-flow tests, the average half-full roughness was 0.6 mm (n = 0.012) for the PVC pipes and 1.7 mm (n = 0.013) for the concrete pipes. Again, although an average of three lengths of each pipe type is hardly definitive, these results are surprisingly similar to the Perkins and Gardiner results of 0.6 and 2.3 mm respectively.

Another intriguing aspect of the augmented-flow results is the hysteresis evident in the roughness variation. The roughness for a given depth was generally somewhat higher on the falling limb than on the rising limb. In the two-cycle test carried out (the results of which are shown in Fig. 5*c*), the hysteresis was repeated in the second cycle and caused even higher roughness. This behaviour is possibly caused by the disturbance of slime and sediment deposits by the higher flows. In the one instance where measurements were made before and after the sewer had been cleaned by City crews, the hydraulic roughness was higher afterwards.

A feature of the two-cycle test worthy of note is the remarkable similarity between the initial loop of the two cycles and the loop measured for the same sewer section almost 1 year earlier by different personnel. This again is a confirmation of the consistency of the field techniques, if not their accuracy.

Design implications

Prior to the present test series, Alberta Environment (1978) required that

> No public sewer shall be less than 200 mm in diameter .... All sewers shall be so designed and constructed to give main velocities, when flowing full, of not less than two feet (0.6 m) per second, based on ... Manning's formula using an 'n' value of 0.013. Use of other practical 'n' values may be permitted by the reviewing agency if deemed justifiable. The following are the minimum slopes which should be provided; however, slopes greater than these are desirable.

Sewer size (mm)	Minimum slope (%)
200	0.40
⋮	
600	0.08

TABLE 4. Measured and calculated half-full roughness for the augmented flow tests

Location	Clean roughness, k_c (mm)	Normal flow parameters		Half-full roughness, k (mm)	
		d/D	k_d (mm)	Calculated	Measured on rising limb
Thorndale Industrial MH 17–15 (PVC)	0.07	0.07	3.8	0.44	1.16
Leduc Romulus MH S11–S12 (PVC)	0.07	0.10	3.2	0.53	0.15
Stony Plain MH 103–102 (PVC)	0.07	0.27	1.2	0.62	1.12
124 Ave. and 43 St. MH R6–R7 (Concrete)	0.14	0.22	13.5	4.08	3.21
69 Ave. and 42 St. MH A14–A13 (Concrete)	0.14	0.15	2.1	0.70	0.55
Yellowbird — 17 Ave. MH 315–317 (Concrete)	0.14	0.30	3.6	1.93	2.54

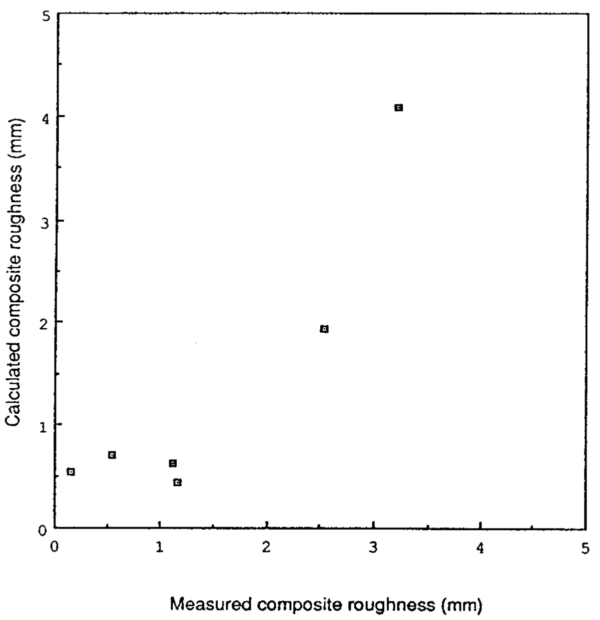

FIG. 6. Comparison of measured and calculated roughness for the augmented-flow tests.

> Under special conditions ... slopes slightly less than those shown may be permitted. Such decreased slopes will be considered only where the depth of flow will be 0.3 of the diameter for design flow ...

It is believed that these are typical of sanitary sewer regulations across the country and will serve as a suitable reference for the following discussion.

Because of its influence on the amount of excavation required, a major economic concern in sewer design in the flat country typical of the Prairies is the minimum slope. Hence a significant feature of the present tests is that several of the sewers tested had slopes substantially lower than the minimum required by the above specifications. Yet these sewers seem to have continued to operate satisfactorily and, as indicated in Fig. 4, the measured roughnesses do not differ substantially from the sewers of better-than-minimum slope. This suggests that the above specifications, which are of a type almost universally used, may be resulting in overdesign with regard to slope, despite the recommendation of what seems to be a lower-than-realistic roughness.

In the above specifications, the minimum slope is based on achieving a specified minimum velocity with the sewer running full. The minimum slope, which then varies approximately as $k^{1/3}$ or n^2, is then particularly sensitive to the specified roughness. This minimum-velocity specification seems to be based on the perceived need to define an equivalent, easily calculated, circumstance to ensure a reasonable scouring action by the flow. However, while velocity is a simply observed parameter in the field and its significance is easily comprehended by nontechnical personnel, it is not directly related to the scouring action of the flow. Such action is more likely to depend on boundary shear (the average force per unit area exerted on the wetted perimeter and deposits thereon). Unlike velocity, this is *independent* of pipe roughness for a given depth, being a direct and simple function of pipe diameter, slope, and flow depth. For example, in the above specification the resulting boundary shear varies from 2.0 Pa for the 200 mm diameter pipe to 1.2 Pa for 600 mm. Hence, if scouring action is really the objective, and about 1 Pa is apparently sufficient to achieve it (the value for the 600 mm pipe), the 200 mm pipe could presumably be laid at about half the specified minimum slope.

Sewers operate for considerable periods of time with just dry-weather flows and therefore, particularly in the upper reaches of the system where pipe size is governed by a specified minimum diameter rather than hydraulics, at much below scouring velocity or boundary shear. On the other hand, if a sewer is going to block up due to slime growth, sediment deposition, or other obstruction, it will likely not take long to do so. Yet lower-than-minimum slope sewers seem to have operated for some time without problems. This suggests that if a minimum scouring action is indeed a justifiable design objective, its magnitude and manner of specification should be reviewed. The background to the current specifications seems to be much less rigorous than is justified by the economic consequences of the specifications.

However, it is debatable whether scouring action is the appropriate design criterion. It may be that a better criterion is simply a slope sufficient to avoid significant "ponding" due to expected construction and postconstruction variation from the design profile. The minimum slope would then be a function of the construction performance and supervision, and foundation conditions, rather than hydraulics. For example, typical specifications allow an "as constructed" relative sag (ratio of sag to pipe diameter) of 0.04 for 200 mm pipe. If this could be enforced for a typical 80 m length between manholes, the slope could be as low as 0.02% and still avoid general ponding. However, it is apparently not unheard of for relative sags as high as 1 to be found following construction of small sewers. To avoid general ponding in this situation, the slope would have to be 0.5% for a 200 mm diameter sewer. Indeed, with ponding avoidance as the criterion, it can be argued that the minimum slope requirement could simply be replaced by the direct requirement that sags causing ponding be avoided.

With such an approach to minimum-slope specifications, the major role played by hydraulic roughness in sewer design would then be in sizing the sewer to carry the design flow. As the uncertainty of the discharge variation (including wet-weather flows) over the design life of the sewer should be considered in selecting the design discharge, a sanitary sewer should reasonably be designed to run near full under the design discharge. For sewers with little storm inflow, it should therefore be assumed that significant slime deposits will exist over much of the wetted perimeter, with just the invert kept clear by abrasion. Furthermore, at such large relative depths the effects of profile irregularities will be substantially reduced. Hence the appropriate design roughness should be close to that recommended by Perkins and Gardiner: about 3 mm (n = 0.015) for PVC and 7 mm (n = 0.017) for concrete. For steep sewers, say with slopes greater than 1%, where there is apparently sufficient scouring action to limit slime growth, roughness values closer to "as new" values may be more appropriate — 0.07 mm (n = 0.009) for PVC and 0.14 mm (n = 0.010) for concrete. (In the recent laboratory tests in Alberta, May *et al.* (1986) found values of 0.05 mm (n = 0.009) and 0.23 mm (n = 0.010) respectively for clean water flow in new PVC and concrete pipes.) For example, Perkins and Gardiner used velocity rather than slope as the criterion and, for velocities greater than about 1.75 m/s, recommended 0.15–0.3 mm (n = 0.010) for PVC and 0.6–1.5 mm (n = 0.012 – 0.014) for concrete.

If sanitary sewers are to be designed to carry substantial stormwater inflow from weeping tiles, roof drainage, cross-connections, and so on (Bodnaruk[3] indicated that stormwater flow in sanitary sewers is now commonly accepted to be at least 3–4 times the design dry-weather flow), the appropriate design roughness will depend on the fraction of stormwater in the design flow. For sewers on less than, say, 1% grade, the design roughness should be a composite of the dry-weather-flow roughness discussed above and the selected "near-new" roughness, with the composite roughness of the waterway determined from the Horton-type relation discussed earlier.

Another practical consideration is that the present tests make it clear that for low flows in sewers, the effective roughness can be substantially higher than those appropriate for design. Furthermore, the augmented-flow tests indicated that this effective roughness is much more depth dependent than usually assumed and that a substantial hysteresis is displayed by the roughness between the rising and falling limbs of a flow hydrograph in the sewer. These features should be considered when routing flows through sanitary sewers.

[3]Bodnaruk, L. 1988. Storm inflow into sanitary sewer systems. Unpublished presentation, Edmonton Hydrotechnical Conference, Edmonton, Alta.

Conclusions

There seems to be an unresolved conflict between reality and standards in the specification of minimum slope for sanitary sewers. In particular, it is difficult to defend the specification of a minimum slope through the artifice of a specified minimum velocity. It is suggested this might be replaced with either a specified boundary shear or, perhaps preferably, simply a *minimum-slope* specification designed to eliminate ponding in the sewer. Neither of these options involves the flow resistance coefficient for the sewer.

The recommended design hydraulic roughness for sanitary sewers should include prescribed variations on the basis of at least slope, pipe material, and the proportion of the design flow contributed by dry-weather flow. If the design flow is predominantly dry-weather flow, roughness coefficients somewhat higher than currently specified should be used. Based on the present tests and those of others, 3 mm (n = 0.015) for PVC and 7 mm (n = 0.017) for concrete are suggested. On the other hand, for slopes greater than about 1%, lower values are appropriate: 0.2 mm (n = 0.010) for PVC and 0.6 mm (n = 0.012) for concrete seem a reasonable compromise. For a lesser proportion of dry-weather flow, the appropriate design roughness should be a compromise between the dry-weather roughnesses given above and the near-new roughnesses of, say, 0.1 mm (n = 0.009) and 0.3 mm (n = 0.010) for PVC and concrete respectively. This compromise should be calculated on the basis of the maximum wetted perimeter occupied by the design dry-weather flow using a simple Horton-type relation to estimate a composite roughness.

When routing flows through sewers, consideration should be given to the possibility of large effective roughness at low depths, strong variation of roughness with flow depth, and a substantial hysteresis effect in the roughness variation with flow depth.

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ALBERTA ENVIRONMENT. 1978. Standards for water supply facilities, sewage work and storm drainage systems. Alberta Environment, Standards and Approvals Division, Municipal Engineering Branch, Edmonton, Alta.

CHOW, V. T. 1959. Open channel hydraulics. McGraw-Hill Book Co., New York, NY.

COMMITTEE TO STUDY LIMITING VELOCITIES OF FLOW IN SEWERS. 1942. Minimum velocities for sewers. Journal of the Boston Society of Civil Engineers, **29**: 286–363.

GERARD, R., BOUTHILLIER, P., and BESMEHN, J. 1986. Field measurements of sanitary sewer flow resistance coefficients. Water Resources Engineering Report 86–2, Department of Civil Engineering, University of Alberta, Edmonton, Alta.

KAZEMIPOUR, A. K., and APELT, C. J. 1981. Shape effects on resistance to flow in smooth semi-circular channels. Institution of Engineers, Australia, Civil Engineering Transactions, **CE23**: 246–252.

KEULEGAN, G. H. 1938. Laws of turbulent flow in open channels. Journal, National Bureau of Standards, **21**: 707–741.

MAY, D. K., PETERSON, A. W., and RAJARATNAM, N. 1986. A study of Manning's roughness coefficient for commercial concrete and plastic pipes. Limited distribution report, Department of Civil Engineering, University of Alberta, Edmonton, Alta.

PERKINS, J. A., and GARDINER, I. M. 1982. Measurements of the hydraulic roughness of slimed sewer pipes. Report No. IT237, Hydraulics Research Station, Wallingford, England.

——— 1985. The hydraulic roughness of slimed sewers. Proceedings of the Institution of Civil Engineers, Part 2, **79**: 87–104.

STREETER, V. L., and WYLIE, E. B. 1981. Fluid mechanics. McGraw-Hill Ryerson Ltd.

THIJSSE, J. T. 1949. Formulae for the friction head loss along conduit walls under turbulent flow. Proceedings, Congress of International Association for Hydraulic Research, Session 111-4, pp. 1–11.

WILCOX, E. R. 1924. A comparative test of the flow of water in 8-inch concrete and vitrified clay sewer pipe. Engineering Experimental Station Series, Bulletin 27, University of Washington, Seattle, WA.

YARNELL, D. L., and WOODWARD, S. M. 1920. The flow of water in drain tile. Bulletin No. 854, U.S. Department of Agriculture, U.S. Government Printing Office, Washington, DC.

Appendix 1. List of symbols

A	waterway area
C_*	conveyance coefficient, dimensionless Chezy coefficient and dimensionless average velocity
d	maximum flow depth
D	pipe diameter
f	friction factor based on pipe diameter (or $4R$ for other shapes)
g	gravitational acceleration
k	total hydraulic roughness $\equiv k_s + k_\nu$ or composite roughness
k_c	hydraulic roughness of clean pipe
k_d	hydraulic roughness of portion of pipe exposed to dry-weather flows
k_s	Nikuradse's "sand grain" roughness, referred to in the text as the hydraulic roughness
k_ν	apparent roughness due to viscous effects at the wall
n	Manning resistance coefficient
n_c	Manning n of clean pipe
n_d	Manning n of portion of pipe exposed to dry-weather flows
P	wetted perimeter
P_c	portion of wetted perimeter above dry-weather flow
P_d	portion of wetted perimeter exposed to dry-weather flow
R	hydraulic radius $\equiv A/P$
Re	Reynolds number $\equiv VD/\nu$
S_f	energy line slope
V	average flow velocity
ν	kinematic viscosity

Appendix 2. Recommendations for the calculation and use of hydraulic roughness

In North America, it is common to represent the flow resistance of sewers by Manning n. However, it is not difficult to show that this parameter is not particularly appropriate for this situation, especially for smooth sewers. Furthermore, and perhaps more importantly, the value of the coefficient gives little indication of the actual roughness of the sewer surface, particularly to the uninitiated, and can therefore be misleading. Because of these considerations, and others, it is becoming more common to see the "sand-grain" hydraulic roughness, k_s, and the Colebrook–White equation used, instead of the Manning n, in sewer analysis and design.

In the past, a strong deterrent to using the Colebrook–White equation was its semi-logarithmic form. However, this objection has been largely removed by the ubiquitous hand calculator. The advantages to be gained from the use of the hydraulic roughness now outweigh the slight remaining inconvenience in using the Colebrook–White equation. This inconvenience can be reduced even further if it is recognized that the Colebrook–White equation can be rearranged into a form that is at once more rational and more suited to use in sewer flow calculations, as shown in the following.

A traditional form of the Colebrook–White equation for use in pipe flow calculations is (Streeter and Wylie 1981)

$$\text{[A1]} \quad \frac{1}{\sqrt{f}} = -0.869 \ln\left(\frac{k_s/D}{3.7} + \frac{2.523}{\mathrm{Re}\sqrt{f}}\right)$$

where f is the Darcy-Weisbach friction factor, D is the pipe diameter, and Re is the flow Reynolds number given by

$$\text{[A2]} \quad \mathrm{Re} \equiv VD/\nu$$

in which V is the average flow velocity and ν the kinematic viscosity of the fluid. It can be shown that the friction factor is given by

$$\text{[A3]} \quad f = 8gRS_f/V^2$$

where g is the gravitational constant; S_f the energy line slope; and R the hydraulic radius of the flow given by A/P, A being the waterway area and P the wetted perimeter.

From these relations and some simple algebra, following Keulegan (1938) and Thijsse (1949), the Colebrook–White equation can be rearranged to

$$\text{[A4]} \quad V/\sqrt{gRS_f} = 2.5 \ln 14(R/k)$$

where k is the total hydraulic roughness given by

$$\text{[A5]} \quad k \equiv k_s + k_\nu$$

Here k_ν can be envisaged as an apparent roughness due to viscous effects at the wall and is given by

$$\text{[A6]} \quad k_\nu = 3.3\nu/\sqrt{gRS_f}$$

which, it is noted, is about the thickness of the viscous sublayer. For sewers flowing full, k_ν is usually $\leqslant 1$ mm; so, for an hydraulic roughness, k_s, of 1 mm or greater, $k \approx k_s$.

If the right-hand side of [A4] is designated as a conveyance coefficient, C_* (it is simply a dimensionless Chezy coefficient, or dimensionless average velocity), the flow in any conduit — pipe or channel — is described by

$$\text{[A7]} \quad V = C_*\sqrt{gRS_f}$$

where, in a pipe,

$$\text{[A8]} \quad C_* = 2.5 \ln 14(R/k)$$

(The results of Kazemipour and Apelt (1981) suggest the constant 14 should be increased to 17 for flow in part-full pipes for which $d/D < 0.3$. In the present analysis, the value 14, which results from the Colebrook–White equation, was used for all values of d/D. The difference due to the "shape effect" represents a change of about 20% in the apparent k.)