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FIELD MEASUREMENTS OF RESISTANCE COEFFICIENTS

IN SANITARY SEWERS

by

Joachim Besmehn

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research for acceptance, a thesis entitled **FIELD MEASUREMENTS OF RESISTANCE COEFFICIENTS IN SANITARY SEWERS** submitted by JOACHIM BESMEHN in partial fulfillment of the requirements for the degree of MASTER OF SCIENCE.

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ABSTRACT

The study reported herein involved field measurements of resistance coefficients in in-service suburban sanitary sewers in Edmonton and the surrounding areas. As plastic (PVC) and concrete are now the most commonly used sewer piping materials in subdivisions, the emphasis in this study was on these types in the size range from 200 mm to 400 mm. A wide range of slopes, from steep to flat, was investigated. Most of the measurements were made at the normal flows occurring in the sewers. Because most of these flows were low under normal circumstances, augmented-flow tests were conducted at selected sites to determine the resistance under conditions more representative of design conditions.

The in-service resistance coefficients were much higher than those determined from clean water new-pipe laboratory tests commonly used in sewer design. The resistance at low flow was very high, being **orders of magnitude** higher than those usually assumed in the evaluation of "scour velocities". There was a strong variation in the effective hydraulic roughness with relative depth (d/D). This variation in hydraulic roughness with relative depth is much stronger than the usual variations presented in standard textbooks.

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I. INTRODUCTION

Currently common sewer design standards recommend that a Manning n of 0.013 should be used and that minimum slopes should produce a velocity of at least 0.61 m/s (2 ft/s) flowing full or half-full. At the present time (1986) no distinction is usually made between different pipe materials. This sewer design criteria is based on recommendations made by a committee, appointed by the Sanitary Section of the Boston Society of Civil Engineers, in 1942, following enquiries made of the municipal engineers and various State Health Departments as to their design regulations and requirements.

Another item of interest has been the variation in resistance with relative depth, (d/D). In conduits flowing part-full laboratory tests have indicated a variation in the roughness coefficient relative to depth. Most of the hydraulic elements graphs (for example, a plot of Q/Q_{full} versus d/D) in common use have been prepared on the assumption that for the particular conduit shape Manning n does not change with flow depth. A review of the literature has indicated that the hydraulic elements graphs available in most standard textbooks which take account of the variation of n with depth have been prepared from the data of Wilcox (1924) and Yarnell and Woodward (1920).

A properly functioning sewer must transport solids in such a manner that deposits in the sewer and associated odour nuisances are kept to a minimum. The invert slope must be sufficient to ensure adequate cleansing velocities at a reasonable minimum flow. Sediment is transported by flowing water in three ways:

1. Bed load movement: In this mode the solid particles are dragged or rolled along the bottom of the pipe by the water.

2. Suspended load movement: As the term suggests in this mode the solid particles are continuously enveloped by flowing water and are transported in suspension.
3. Saltation. The solid particles move alternately in bed load movement and suspension.

Solids which are transported in suspension move with the velocity of the water, whereas solids moved as bed load progress along the sewer at much lower velocity. To ensure freshness at the point of disposal it is desirable that the sewage solids be transported mainly in suspension. Shields (1936) indicated that the boundary shear stress required to produce particle motion along the bottom is approximately proportional to the diameter of the particles and their submerged specific weight. From the findings of Shields, Camp (1946) derived the following equation for the velocity required to transport sediment:

$$V = \frac{1.0}{n} R^{1/6} \sqrt{B\ (s - 1\ D)} \qquad (1)$$

where V is the average velocity, s is the specific gravity of the particle, D the particle diameter, R the hydraulic radius and B is a dimensionless constant with a value of about 0.04 to move granular particles and of about 0.8 for adequate self-cleansing of sewers and drains. It is common to increase the minimum gradient by 50% for lines at the top of the system to prevent excessive deposition of solids since the flow depths and velocities are relatively small.

Fair et al. (1966) suggested that for part-full flow the slope required to transport the same size particles that would be transported by the sewer flowing full is given by.

$$\frac{S}{S_f} = \frac{R_f}{R} \qquad (2)$$

This follows from the requirement that the average boundary shear stress be the same in the two cases. Equation 2 suggests that the slope must be doubled when the depth of flow drops to 0.2 of the diameter and quadrupled at 0.1.

The peak hourly flow in a sewer is often several times the average daily flow. The depth and mean velocity thus vary considerably. Furthermore, the population that a sewer serves commonly increases over the design period, and therefore the peak flow also increases with time. Hence the early years are critical for adequate scouring velocities whereas the sewer must be sized to have adequate capacity for the peak flow at the end of the design period. Since there is a large factor of safety in the estimated peak flows most engineers design sewers to flow just full at peak design flow. There will then be a free surface for ventilation at all lesser flows.

Traditionally, the results of clean water new-pipe laboratory tests have been used to define the resistance coefficients of sewers. These tests do not, however, simulate the effects of scaling, or slime build-up, which occurs on the inner pipe surface under operating conditions.

However, a number of field investigations on the in-service roughness of sewers have been reported in the literature. Extensive field testing of sewers has been conducted in Britain, which has recently led to recommendations for in-service roughness coefficients for different piping materials. For example, the Hydraulics Research Station (1983) recommends the following roughness coefficients for sewers, slimed to half depth, and flowing at a velocity of approximately 0.75 m/s when half full;

PVC;	$k_s = 0.6$ mm	$n \approx 0.011$
Concrete;	$k_s = 3.0$ mm	$n \approx 0.014$

where k_s is the equivalent sand grain roughness used in the Colebrook-White equation. These values are very much higher than those used for sewer design in North America.

The purpose of this investigation was to assess the applicability of these results in Western Canada. As plastic (PVC) and concrete are now the most commonly used sewer piping materials, the emphasis of this study was on PVC and concrete in the size range from 200 mm to 400 mm.

II. REVIEW OF PREVIOUS WORK

A. Laboratory Tests

Although resistance coefficients determined from clean water laboratory tests should not be used in sewer design, such tests provide an indication of how resistance varies with geometry, pipe material, and other pertinent variables.

Yarnell and Woodward (1916, 1917) carried out intensive and painstaking research on velocities in pipes flowing part-full. Experiments were made with a variety of the usual commercial sizes of sewer pipe, both of clay and concrete, from 4 to 12 inch (102 mm to 300 mm) inside diameter. Each pipe was tested at the following slopes: 0.0005, 0.0010, 0.0020, 0.0030, 0.0050, 0.0075, 0.0100, 0.0125 and 0.0150. The pipes were laid in about 7 inches of soil in the bottom of a wooden flume 570 ft (174 m) long. The pipes were plain-ended with laying lengths of 1 ft (0.3 m) for the smaller sizes and 2 ft (0.6 m) for the larger. Each length was abutted without sealing. Flow depths were measured by piezometer taps in the pipe wall. To measure the water entering and discharging from the line, 90°-notch weirs were used. Kutter's n (Kutter's n is considered equivalent to Manning's n) and Chezy's C were determined for all runs. Measurements were made for 824 flow conditions. These included 237 measurements at relative depths from 0.83 to 0.99. The data of Yarnell and Woodward showed that Kutter's (or Manning's) n increased as the relative depth (d/D) decreased. Examples of their data are shown in Figure 1. As mentioned earlier, the variation of Manning n with depth found by Yarnell and Woodward is still used in sewer design.

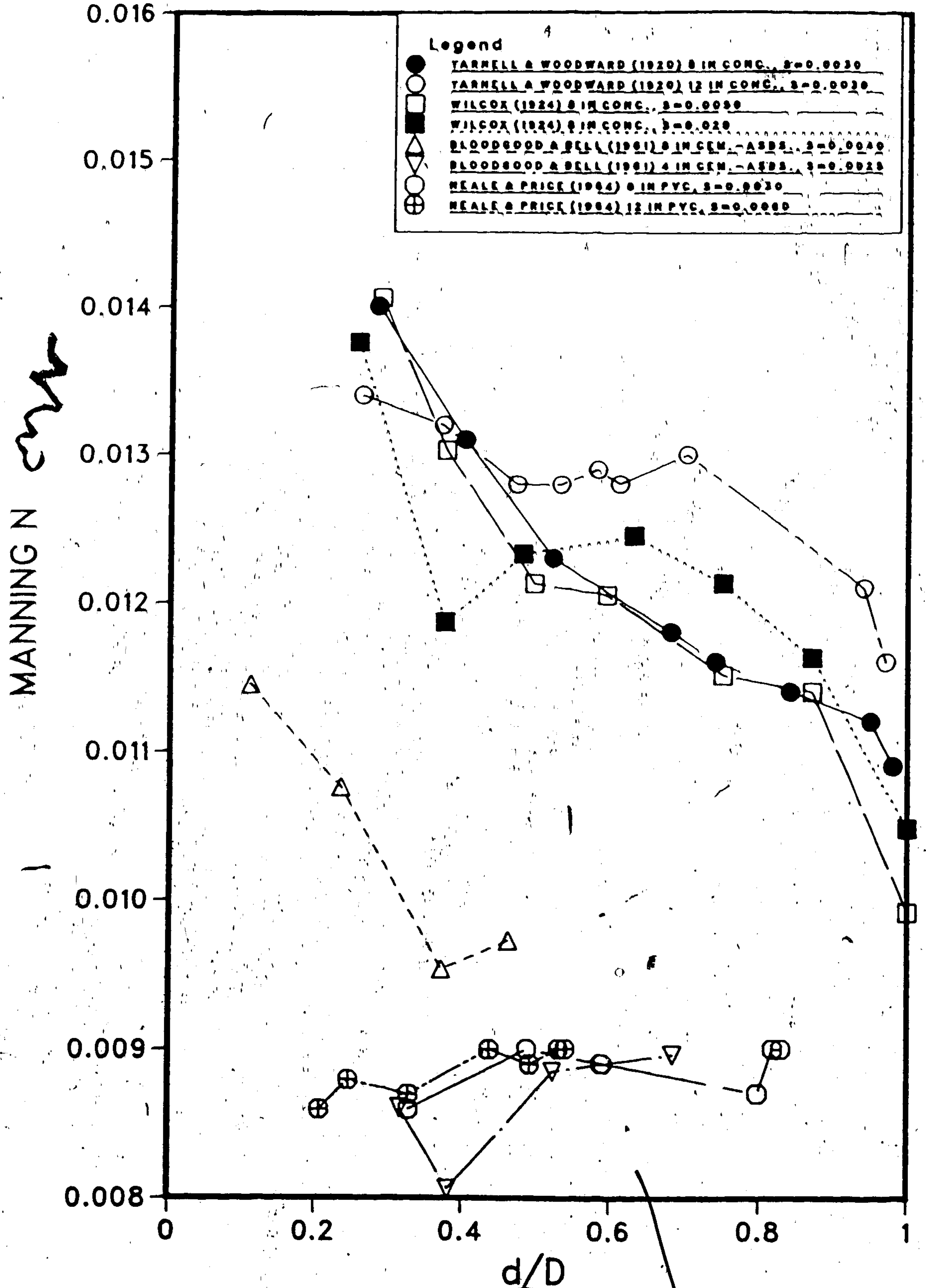

Figure 1. Composite plot of Manning n vs d/D from experimental results of early investigators.

Wilcox (1924) carried out laboratory tests on 8 inch (200 mm) concrete and vitrified clay pipe. The pipe slopes used were 0.005, 0.01, 0.015, 0.02, 0.03 and 0.04. The length of each line was 98.3 ft (30 m). Wilcox measured velocities and depths, as well as discharge and depths, to determine the resistance coefficients. The depths were measured by piezometer taps in the pipeline, discharges by orifice meters and velocities by the salt dilution method. For the latter measurements, salt solution was added to the flow through a slot in the intake end of the pipe. A headphone, probe and voltmeter were used, downstream, to measure the variation of salt concentration as the salt slug passed by the probe. The change in conductivity caused a change in the tone in the headphone. The time from the instant the salt solution struck the water to the first sound in the headphore, and the time to the last audible sound, were both measured with a stopwatch. The mean time between the first and the last sound was used to determine the average velocity. No less than five observations were taken for the velocity at each depth and grade. Quite different results were obtained for the resistance coefficient based on whether the velocity or the discharge was measured. When velocity was used Manning's n was generally quite low for low values of d/D and increased as d/D increased, to some peak value then, as d/D was increased further, Manning's n became lower. When discharge and depth were used to compute velocity, Manning's n was high at low values of d/D and became lower as d/D increased. The difference in the resistance coefficients is likely attributed to experimental error in the velocity measurement: the two sound signals would not provide a good definition of the true average velocity. Wilcox's data showed that for the same d/D the pipe at a steeper slope had a higher resistance. When depth and discharge were used the average n (d/D > 0.8) was 0.011 for both the 8 in (200 mm) vitrified clay tile and concrete tile. Typical results given by Wilcox, based on the depth and discharge measurements, are reproduced in Figure 1. They can be seen to follow those of Yarnell and Woodward quite closely.

Bloodgood and Bell (1961) measured the flow resistance of 4 in (100 mm) vitrified clay, 4 in (100 mm) centrifugally-spun cast iron, 4 in (100 mm) cement-asbestos, 8 in (200 mm) vitrified clay and 8 in (200 mm) cement-asbestos pipes. The lines were supported on a trestle and were about 300 ft (91 m) long. There were 7 to 9 piezometer taps installed along the line. Openings were made in the top of some of the pipes so that the depth of flow could be measured directly and the surface of the water observed. The direct measurements confirmed the piezometer measurements. The flow rate for each test was determined using a calibrated V-notched weir as the flow passed into a stilling tank, from which the water entered the line being tested. Measurements were taken for the 4 in (100 mm) pipe tests at slopes of 0.0025 and 0.0040, and relative depths from 0.23 to 0.75. For the 8 in (200 mm) pipe tests, measurements were taken at a slope of 0.004 and relative depths from 0.11 to 0.46. There were 381 determinations of n for clay pipe, 387 for cement-asbestos pipe, and 104 for cast iron pipe. There was no significant variation in Manning n with pipe material (for the 4 in (100 mm) pipes at the two slopes the average values were: cast iron, 0.00835; clay, 0.00865 and cement-asbestos, 0.00853; for the 8 in (200 mm) pipes the values were: cement-asbestos, 0.01037 and clay, 0.01031) but the results indicated that n values for clay and cast iron pipe were significantly lower for the steeper slope. From these results Bloodgood and Bell concluded that that the relative depth d/D appeared to be the principal factor in the variation of the n values obtained from the test data. Examples of this data are also given in Figure 1.

Ackers (1961) tested vitrified clay tile, spun precast concrete and pitch fibre pipes in the laboratory, flowing both full and part-full. The vitrified clay tile line was 180 ft (55 m) of 12 in (300 mm) diameter pipe, half in 3 ft (0.91 m) long units and the remainder 2 ft (0.61 m) units. The first series of tests were made on the vitrified clay line with joints set as accurately as possible. Both the pipe-full and part-full runs covered hydraulic gradients from 0.001 to 0.020. Data for full- and part-full flow were obtained after imposing eccentricity at the joints, as follows:

Series A	Near perfect joints.
Series B	0.4 in (10 mm) step-up at each joint.
Series C	0.3 in (8 mm) step-up.
Series D	0.2 in (5 mm) step-up.
Series E	0.35 in (9 mm) step-down.
Series F	0.25 in (6 mm) horizontal displacement at each joint.
Series G	Random steps, up and down, at each joint, with average step of 0.193 in (5 mm).
Series H	Joint eccentricities of random amount and in random direction, with average of 0.194 in (5 mm).

Next, 120 ft (36.6 m) of 12 in (300 mm) diameter spun-concrete pipes in 6 ft (1.8 m) long units were installed at a slope of 0.01. Cornelius-type joints were used (i.e. spigot and socket joints sealed with a solid rubber ring): these automatically ensure accurate centering of each pipe with its neighbours, so that the pipeline tested had joints which were virtually free from lips. Moreover, it was aligned and levelled to within 0.0625 in (1.6 mm), this representing the best attainable quality of workmanship.

After completing tests on the concrete pipes, 96 ft (29.3 m) of 6 in (150 mm) diameter pitch-fibre pipes in 8 ft (2.4 m) long units were tested. These pipes had a glassy internal surface of pitch, and the machined taper joints provided near perfect-mating between the pipe units.

For all tests the discharge was measured by a weir on a tank as the water entered the pipeline and the hydraulic gradient was measured from piezometer taps. Under part-full flow conditions, the depths measured at the piezometer taps were averaged (keeping the 3 ft and 2 ft pipes separate, and omitting the upstream taps at which uniform flow conditions had not been established).

The flows tested in all the pipelines were in the smooth wall - rough wall transition region and the Manning equation did not fit the experimental data very well. Instead, the Colebrook-White equation was used and the hydraulic resistance represented by k_s, the Nikuradse equivalent sand grain roughness. This parameter is now almost universally used for sewer design in Britain.

Mean roughness values obtained for pipe-full conditions for the vitrified clay pipes with velocities in the range 1 to 10 ft/s (0.30 to 3.1 m/s) are given in Table 1. For the vitrified clay pipes with badly-made joints the effective roughness of the pipe in 2 ft (0.6 m) units was greater than that in 3 ft (0.91 m) units. With near perfect joints, the 3 ft (0.91 m) long pipes were very similiar to the 2 ft (0.61 m) long ones. This indicates the influence joint alignment and spacing can have on the hydraulic performance of a pipeline.

The average roughness obtained under part-full flow conditions for vitrified clay pipe are also given in Table 1 for values of $0.2 < d/D < 0.8$ (Ackers noted that at depths below 0.2D the analysis becomes very sensitive to errors in depth measurement, and the k_s values at these low discharges are unreliable). These results show that there is no great change of mean roughness over the range of proportional depths $0.2 < d/D < 0.8$, although there is a slight tendency for part-full k_s values to be higher than their pipe-full values except when the joint eccentricity consists of steps down at each joint (Series E). Ackers' results indicated that with a step-up at each joint the roughness increased as the depth decreased, and the reverse for a step-down at each joint.

For concrete pipes the mean roughness for full flow was 0.040 mm. For part-full flow it was 0.14 mm for $0.2 < d/D < 0.8$. The data for the pitch-fibre pipes indicated an average $k_s < 0$ for full-flow while for part-full flow the average was 0.034 mm.

It was evident that for all pipe types tested there was an increase in apparent roughness as the flow went from full-flow to part-full flow.

From these laboratory tests and field measurements of joint eccentricity, Ackers recommended k_s values for different diameter pipes based on the workmanship of pipe installation. A table of k_s values was given for vitrified clay tile based on the worst conceivable standard of pipe laying, i.e. one which resulted in the outer surface of the spigot

Table 1. Mean roughness values k_s, for 300 mm vitrified clay tile (from Ackers, 1961)

Series	Joint	Average Lip at Joints (mm)	Roughness k_s in mm			
			Full pipe		Part-full $0.2 < d/D < 0.8$	
			3 ft (0.91 m) long pipes	2 ft (0.61 m) long pipes	3 ft. (0.91 m) long pipes	2 ft (0.61 m) long pipes
A		2.13*	0.04	0.04	0.06	0.03
B	step-up	10.06	0.12	0.16	0.17	0.19
C	step-up	7.62	0.09	0.10	0.12	0.11
D	step-up	5.18	0.06	0.07	0.08	0.07
E	step-down	8.84	0.10	0.12	0.08	0.08
F	step sideways	6.40	0.07	0.07	0.09	0.06
G	Random: up and down	4.88	0.08	0.07	0.07	0.03
H	Random: up, down and sideways	4.88	0.06	0.07	0.10	0.06

* The residual lip at invert and soffit with joints centered as accurately as possible.

touching the inner surface of the socket. This table of k_s values for new pipes badly laid is reproduced here as Table 2.

Table 2. Predicted roughness values for the poorest standard of pipe laying (new vitrified clay tile) (from Ackers, 1961)

Pipe Diameter Inches		Assumed Lip at Joints (mm)	Predicted k_s value mm	
			Pipes in 3 ft units	Pipes in 2 ft. units
3	(76 mm)	7.92	0.09	0.12
6	(150 mm)	10.97	0.12	0.18
9	(230 mm)	12.80	0.18	0.24
12	(300 mm)	15.85	0.24	0.34
18	(460 mm)	15.85	0.24	0.34
24	(610 mm)	18.90	0.34	0.46
30	(762 mm)	18.90	0.34	0.46
36	(914 mm)	25.30	0.55	0.79

In an earlier investigation Ackers (1959) had measured the headlosses at open invert manholes and found these to be small, except when surcharge occurs. If the manhole contains a bend and sewer velocities are high, the headlosses under surcharge may be considerable.

Neale and Price (1964) measured the flow resistance of plastic (PVC) pipe. Both full-flow and part-full flow tests were performed. The part-full flows were varied from 0.25D to 0.75D at slopes of 0.0030, 0.0063, 0.0105 and 0.100. The pipe sizes tested were 8 in (200 mm) and 12 in (300 mm). A total pipeline length of 100 ft (30.5 m), in 20 ft (6.1 m) laying lengths, was used in each test arrangement. The average Manning n for the full flow tests was 0.0082. They also noted an apparent increase in the roughness for part-full flows, even though there was a considerable scatter in their data: the average Manning n for the part-full flows was 0.0086. Some of these results are shown in

Figure 1. Like Ackers, they show little variation with d/D.

There is a significant difference in the part-full flow results between the recent Ackers (1961) and Neale and Price (1964) and older Yarnell and Woodward (1920) and Wilcox (1924) results. The older tests indicated a significant change in Manning n for $0.2 < d/D < 0.8$ whereas the recent tests do not. Ackers' believes this is likely due to the poor jointing between the pipes in the old experiments.

Bock (1966) performed experiments on part-full flow in smooth pipes. The results indicated that the hydraulic roughness showed little variation with depth for $d/D > 0.3$ but increased substantially for relative depths below this. The concommitant variation in Manning n shown in Figure 2 would be almost imperceptable for $d/D > 0.2$, as found by Ackers and Neale and Price.

The above clear water lab-tests indicate that the roughness of a pipeline is influenced by:

1. depth of flow
2. pipe material
3. spacing and alignment of joints

The laboratory tests indicated that there is an increase in hydraulic roughness when the flow goes from full to part-full due to a shape effect. However, various researchers have not been in agreement on how the roughness varies with relative depth.

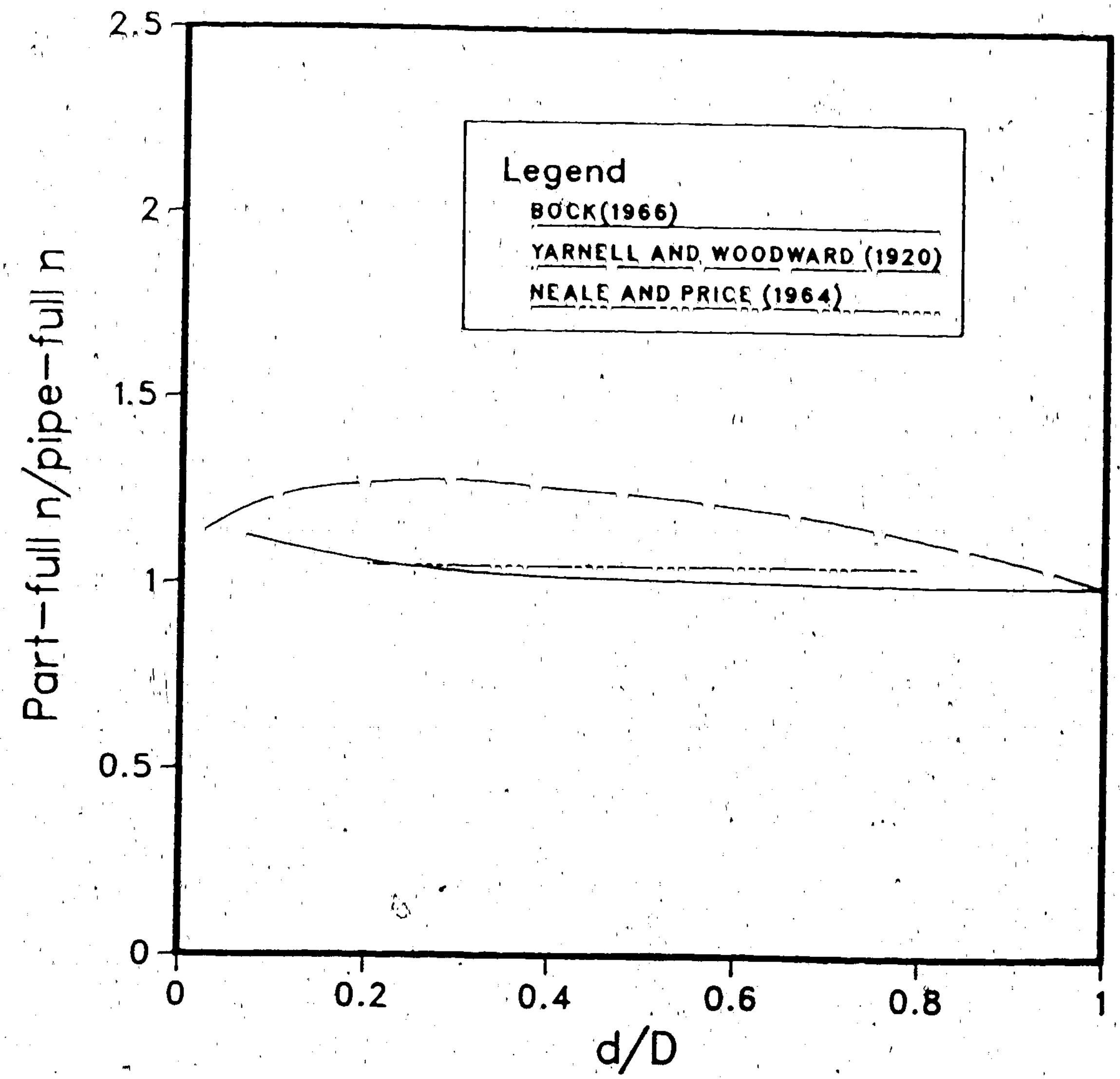

Figure 2. Variation of ratio of part-full resistance coefficient to pipe-full resistance coefficient with relative depth for various researchers.

It remained to be seen if resistance to flow in sanitary sewers will be similar to that indicated by the clear-water tests. By far the most comprehensive and realistic tests with this objective in mind were those carried out under simulated field conditions by Perkins and Gardiner (1982) for the Hydraulics Research Station at Wallingford, England.

The experimental rig consisted of a pipeline 157 m long, 225 mm in diameter, laid above ground near a sewage pumping station. The site did not permit a continuous length of straight pipe, so it was necessary to include a 180° bend in the middle of the pipeline. Fresh sewage was circulated through the pipe, with the flow varying with time in a manner similar to the flow variation in a normal gravity foul sewer, the flow being part-full at all times.

The arrangement for supplying flow to the test rig was to pump from the wet well of the sewage pumping station to a constant head tank 8 m above the ground. From here the flow passed to the test pipeline via a tilting weir, whose angle (and hence discharge) was controlled to a predetermined sequence by a rotating cam mechanism. A vertical drop pipe was installed at the upstream end to allow the escape of any air that had been entrained in the sewage. A weir was installed at the downstream end of the pipeline for measuring the discharge. The sewage in the wet well had already received some primary treatment in that grit and most of the rags had been removed by grit channels and coarse screens.

The whole pipeline was insulated to maintain the sewage at the same temperature as the sewage arriving at the pumping station.

Special access holes were cut into the soffits of the pipes at the upstream ends of the test lengths to allow insertion of a camera for taking regular photographs of the interior of the pipes.

To determine the clean pipe roughness before any sliming had taken place, the rig was designed so that clean water could also be circulated through the experimental pipeline.

Five test sections, from 13 to 20 m long, were incorporated in the pipeline. These consisted of asbestos-cement (20 m length), spun concrete (20 m), vertically-cast concrete (13 m), unglazed clay (20 m), and plastic (PVC) (18 m). Upstream from each of the test lengths, there was a short length of pipe of the same material as the test length (these lengths were 20, 5.5, 3, and 11.5 m respectively). This was to serve as a transition between test lengths of pipes of different materials. Downstream from each test length there were short pipe sections (4 m) of the same material, specially jointed to allow them to be removed easily from the test rig for photographing and documentation of the slime layer around the pipe perimeter. One of the aims of these tests was to determine if a correlation existed between the measured hydraulic roughness and the slime thickness and weight on the pipe wall.

Pressure taps were located at 5 or 6 different sections along each test length. At each section there were four pressure taps. The four taps were at 0° (soffit of pipe), 60°, 137° and 240°, around the pipe circumference and were inter-connected so that they measured a mean pressure over the cross-section of flow. The various sets of pressure taps were linked to manometer boards fitted with vernier scales that enabled the piezometric head to be measured within ±0.1 mm. **The experimental rig was made to run full for all roughness measurements.**

Three different 24-hour hydrographs were used in the experiments.

The shape of all three were similar: the difference between them was in the discharges that occurred at the peaks and troughs. The characteristics of the different hydrographs are given in Table 3. During any run, the hydrograph was repeated daily. Three separate, long term runs, were carried out as follows:

Run 1: Slope 0.004, hydrograph 1, total run time 335 days.

Run 2: Slope 0.004, hydrograph 2, total run time 206 days.

Run 3: Slope 0.01, hydrograph 3, total run time 188 days.

The general procedure followed in each of the three runs was first to determine the hydraulic roughness of the pipes in a clean condition, using clean water. These tests were carried out with the pipe running full at Reynolds numbers ranging from 1×10^5 to 3×10^5. Following the clean water tests, sewage was passed through the rig and the roughness was determined at regular intervals.

The method for determining the roughness was to stop the discharge variation, make the pipe flow full by closing a valve at the downstream end, and then to measure the pressure distribution along the conduit. A best-fit hydraulic gradient was then determined for each test length, from which the hydraulic roughness was calculated using the Colebrook-White equation. It was assumed that the pipe diameter was the original clean diameter; no allowance was made for any reduction in effective diameter due to sliming. Because the pipe was made to run full for the roughness measurements, the roughnesses determined were for a pipe where the perimeter was partially slimed (up to the maximum depth of flow for the hydrograph) and partially clean. The slimed portion usually had a very uneven surface.

Table 3. Hydrograph Characteristics (from Perkins and Gardiner, 1982)

	Peaks						Troughs					
	Maximum			Intermediate			Maximum			Intermediate		
Hydrograph No.	Discharge l/s	Velocity m/s	Prop'l Depth*	Discharge l/s	Velocity m/s	Prop'l Depth*	Discharge l/s	Velocity m/s	Prop'l Depth*	Discharge l/s	Velocity m/s	Prop'l Depth*
1	20.25	0.78	0.6	10.9	0.67	0.42	4.7	0.52	0.27	1.7	0.37	0.16
2	15.0	0.75	0.5	10.0	0.66	0.41	6.8	0.59	0.33	1.8	0.40	0.17
3	26.0	1.18	0.54	17.5	1.07	0.43	11.5	0.95	0.34	3.0	0.64	0.18

* The proportional depths and mean velocities are calculated assuming that the pipe surface is slimed, with an equivalent sand roughness of k_s = 1.5 mm.

On days when the roughness was being determined, the practice was to carry out three seperate tests at Reynolds numbers ranging from 0.85×10^5 to 1.5×10^5 for Runs 1 and 2 and from 1.3×10^5 to 1.8×10^5 for Run 3. The Reynolds number was restricted to this range to ensure the shear stress during these tests was not greater than the shear stress being generated during the slime building process. The maximum shear stress in Runs 1 and 2 (during the slime building process) was approximately 2.5 Pa, whereas the maximum during the roughness determination tests was 2.2 Pa.

As well as determining the pipe roughness, a complete photographic record was made of the changing sliming pattern in each of the test lengths. To do this the removable sections were periodically taken from the pipe, the interiors photographed, and the slime scraped off, dried and weighed.

From time to time, dissolved oxygen levels were measured in the sewage in the rig and in the sewage arriving at the pumping station. A continuous record of the sewage temperature in the rig was also maintained.

Between each of the three main runs, the slime was removed from the pipes by means of a pressure jetting system.

The results of the clean water tests are given in Table 4. These would include the effect of surface texture as well as joint discontinuites and pipe misalignments, although the latter two factors were not considered to have a great influence because of the care taken in assembling the pipeline.

Table 4. Roughness of Clean Pipes (from Perkins and Gardiner, 1982)

Material	Roughness, k_s (mm)		
	Before Run 1	Before Run 2	Before Run 3
Vertically cast concrete	0.09	0.25	0.09
PVC (plastic)	0.04	0.10	0.06
Spun concrete	0.09	0.18	0.16
Asbestos-cement	0.02	0.07	0.04
Clay	0.07	0.18	0.13

When sewage was passed through the pipe the hydraulic roughness increased very rapidly over a short period of time (30 days or less). After this initial rapid increase, the roughness fluctuated with time, as shown in Figure 3, suggesting that it is a function of the growth and distribution of the slime on the pipe wall, which varied with time.

Slime is formed by bacteria, protozoa and fungi in the sewage. The populations of these various organisms are influenced by the sewage temperature, the food that it contains, the amount of dissolved oxygen and its chemical composition. It was felt the sewage slime was influenced most by the fungi present, e.g. during winter, the fungi tend to be dominant and produced a slime with a tough skin, whereas in summer the fungi had less influence and the slime was affected more by the other organisms in the sewage.

Temperature was a significant parameter. According to information supplied to Perkins and Gardiner, bacterial growth rate is a direct function of temperature and doubles when temperature increased from 10°C to 20°C. Thus if the temperature of the sewage was increased, without changing any of its other characteristics, the slime growth would also increase.

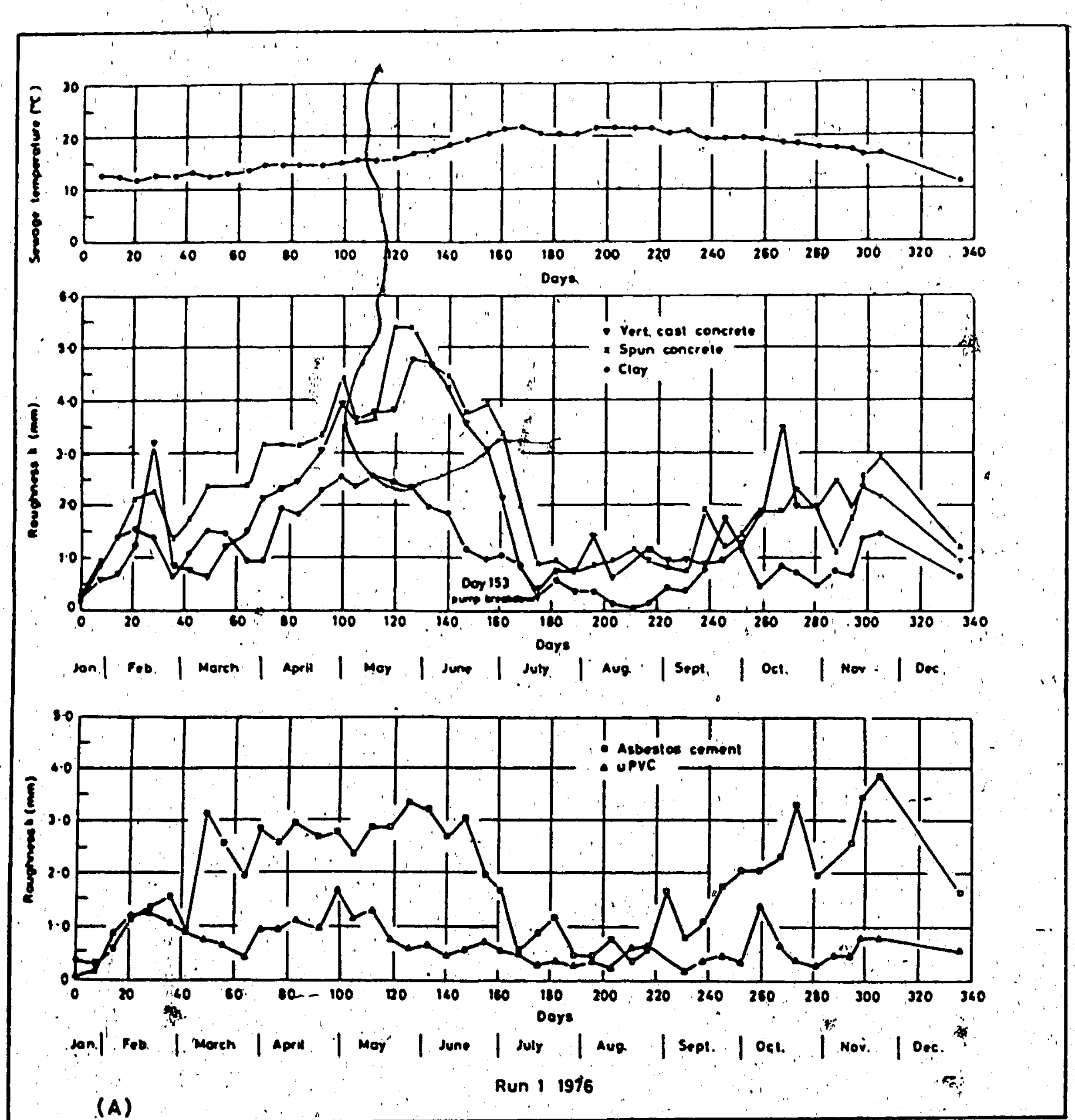

Figure 3. Example of Perkins and Gardiner's (1982) Experimental Results - adapted from Perkins and Gardiner (1982)

After the initial rapid slime growth period the results showed that:

1. In the spring (March to June) roughness increased at a steady rate until the sewage temperature reached a limiting value, after which the roughness stayed constant. For concrete pipes, based on Run 1, this temperature seemed to be about 16°C. The limiting temperatures were lower for the other materials, probably about 15°C for clay, 14°C for asbestos-cement and 13°C for PVC.
2. In the summer (July and August) roughnesses were low relative to other times of the year. The exact mechanism which triggers the reductions was not clear.
3. In the autumn (September to November) the roughness increased again.
4. Though no runs were made during the winter, it appears that when sewage temperatures are lower the roughness falls to a low value, though probably not as low as mid-summer.

Perkins and Gardiner could offer no precise explanation of these findings but they offered some thoughts on features that likely influence the slime layer. The slime thickness, and roughness, arises from a balance between the growth rate, the rate of sloughing due to the shearing action of the flow and to the natural life-death cycle of the slime itself. Thus if the rate of growth was reduced, the slime thickness, and therefore the roughness, would reduce. It was suggested that the low roughnesses in the summer were due to growth being inhibited by the factors related to the incoming sewage. In winter growth is inhibited by the low temperatures. Between these two conditions there is a range of temperatures where growth is more rapid and roughness increases. Various hypotheses were examined in an attempt to explain the relationship between roughness and sewage temperature for each pipe material. Some limited success was achieved but no hypothesis was able to explain satisfactorily all the observed results.

When the results of Run 2 were compared with the results of Run 3, it was found that the high and low roughness values for Run 3 were about one third of the corresponding values in Run 2. The flow velocities in Run 3 were higher than in Run 2 and Run 1.

During Runs 1 and 2 a few roughness measurements were carried out under part-full conditions to get some indication of the variation of roughness with depth of flow. The procedure during these tests was to set a steady discharge, using the valve at the downstream end to establish uniform flow conditions. This was difficult to achieve, partly because the valve did not allow fine adjustments to be made, and partly because the water surface was very undulatory, making it hard to determine when the flow was uniform. The water depths in the pipe were measured by the piezometer taps.

The roughness of the part-full pipes was estimated from the best-fit energy grade line, the mean geometric parameters over the reach, and the Colebrook-White equation, assuming that the flow was in the rough-turbulent region. The data for these part-full tests showed quite large scatter. There were significant changes in roughness for only small changes in relative depth. For example, in Run 1 two tests were carried out at virtually the same relative depth for PVC pipe but gave rise to part-full roughnesses of 2.0 and 3.4 mm. Despite such inconsistencies general trends were evident. The maximum roughness occurred when the flow depth corresponded to that of the maximum depth of sewage, and that the part-full roughness was significantly greater than the pipe-full roughness. Table 5 gives the salient features of these results for part-full flow.

Perkins and Gardiner pointed out that the variation of roughness with flow depth would produce a stage-discharge characteristic different from that of a pipe with uniform roughness around the periphery. For example, instead of the maximum capacity occurring at a proportional depth of 0.95, a slimed sewer with non-uniform roughness will have its maximum capacity when it is flowing full.

The initial growth period of the slime was very short (3-4 weeks), and after that the quantity of slime present on the walls appeared to depend on other factors which were not necessarily a function of period in use. Pipes removed late in the test period appeared to have the same pattern and degree of sliming as those removed one month earlier.

There was little difference in the general characteristics of the slime that grew on different pipe materials in different runs. Samples taken from the pipe inverts showed significantly higher bulk density and contained more sand than samples taken from near the waterline. There was also a difference in appearance. In the early stages of each run the bottom of the pipes were covered with a thin (1-2 mm) uniform layer of grey slime, while at the waterline white, gelatinous lumps up to 10 mm high were present.

The average dry weight of slime per unit area of slimed surface was used as a measure of the quantity of slime present on the pipe walls. This was determined for the proportion of the pipe perimeter that had slime attached. The results of these measurements indicated:

Table 5. Part-full flow (from Perkins and Gardiner, 1982)

Material	Run 1			Run 2		
	Maximum Part-Full Roughness k_s (mm)	Pipe-Full Roughness k_s (mm)	Relative Depth for Maximum Roughness	Maximum Part-Full Roughness k_s (mm)	Pipe-Full Roughness k_s (mm)	Relative Depth for Maximum Roughness
PVC	3.0	0.5	0.7	3.0	1.0	0.5
Spun-concrete	5.5	1.9	0.8	9.0	4.5	0.6
Asbestos-cement	5.0	2.5	0.7	10.0	4.5	0.4
Clay	7.0	0.6	0.4	5.0	1.5	0.5

1. The increase in slime weight with time showed a similar pattern to the increase in hydraulic roughness. The drop in hydraulic roughness on day 150 in Run 1 was coincident with a drop in slime weight. In both Run 1 and Run 2 the increase in roughness at the end of the test was reflected by an increase in the amount of slime present.
2. After the initial period of growth the spun concrete surface showed consistently more slime than clay or PVC. Although clean asbestos-cement pipe has a very low roughness (k_s = 0.02 mm), towards the end of each run both the roughness and the amount of slime present were higher than for either the PVC or clay pipes.
3. In Run 3 the higher velocity resulted in much less slime growth in all pipe materials, although the general pattern of a uniform, smooth layer on the invert with larger slime lumps at the waterline was still evident. The difference between the pipe materials was also much less marked, although the spun concrete and asbestos-cement surfaces still had slightly more slime than PVC or clay.

As both the slime weight and hydraulic roughness showed broadly similar trends, an attempt was made to correlate k_s and slime weight. Before correlating the data for roughness and slime weight it was necessary to compute k_s values for the slimed portions of the perimeter, so that the influence of the relatively clean pipe crowns could be eliminated. This was done using perimeter-weighted friction factors:

$$f_c = f_s p_s + f_n p_n \qquad (3)$$

where

f = friction factor

p = portion of total perimeter occupied by a surface roughness

and the subscripts are

c = composite surface

s = slimed surface

n = new (or clean) surface

The crown of each pipe was assumed to be completely unaffected by slime and to have clean pipe roughness values of:

	k_{sn} (mm)
clay	0.09
PVC	0.04
asbestos-cement	0.02
concrete	0.09

Although there was a clear trend for hydraulic roughness to increase with the amount of slime present, the correlation was not good enough for a reliable prediction of k_s from slime weight. This is likely because the textural 'roughness' of slime lumps at the waterline in part-full sewers has a much greater influence on hydraulic roughness than the total volume of slime present.

The HRS data indicates that the higher the velocity in the pipe the quicker equilibrium is established. Perkins and Gardiner determined the relation between slime weight and velocity from their data and data from other researchers. The best-fit line to all the data points for all pipe materials had the equation:

$$W = 48.9\, V^{-2.10} \qquad (4)$$

where

W = dry weight of slime per unit area of slimed surface in g/m^2

V = mean velocity in m/s.

The correlation coefficient r^2 was 0.91. The exponent suggests that the amount of slime present is indeed inversely proportional to the shear stress exerted by the flow.

The principal conclusions from Perkins and Gardiner's experiments are:

1. Slime builds up on the sewer very quickly to an equilibrium between the applied shear stress and the characteristics of the sewage. The subsequent roughness and slime mass variation is related to changes in the physical, chemical or biological nature of the sewage.
2. Pipe material does influence the equilibrium amount of slime, and hence the roughness. As a general rule the greater the initial roughness of the pipe, the greater the slimed roughness.
3. Slime thickness is inversely proportional to boundary shear.
4. The roughness is much greater for part-full flow than for full flow in sewers, primarily because of the non-uniform distribution of slime.

Based on their experimental data Perkins and Gardiner recommended the k_s values given in Table 6. These are for pipe-full flow and apply to pipes with velocities of about 0.75 m/s, carrying only sewage and slimed to approximately half depth. For steeper pipes with velocities around 1.2 m/s, corresponding to Run 3, high, low, and median values are roughly one third of those given in Table 6.

Table 6: Recommendations for Design and Analysis (from Perkins and Gardiner, 1982)

Material	High k_s (mm)	Low k_s (mm)	Median k_s (mm)
Vertically cast concrete	3.8	1.3	1.8
Spun concrete	4.2	1.8	2.3
Asbestos-cement	2.8	1.2	1.8
Clay	2.3	0.6	1.1
PVC (plastic)	1.1	0.6	0.6

B. FIELD TESTS

A number of field tests to determine roughness coefficients in sanitary sewers have been reported in the literature. Extensive field tests have been conducted in Britain, although many of these tests were on large sewers and pipe materials which are not commonly used for sewers today in North America. In these, and others in North America, many different methods have been used to determine the in-situ roughness of the sewers. It is worthwhile to review some of these field tests, to compare the techniques and results with the present study.

The only reported field test results for plastic sewer pipe are those by Bishop and Jeppson (1978). They reported a maximum k_s of

1.80 mm and a minimum value of 0.02 mm for a range of relative depths of 0.06 to 0.4 for their tests on 300 mm and 200 mm plastic sewer pipe. Bishop and Jeppson used a current meter, either an Ott propeller meter or a Marsh-McBirney electromagnetic current meter (Model 201) at the manholes to determine the average velocity in the sewer. This technique has strong limitations, because it does not obtain a true average velocity over the test length, nor over the cross-section at the manhole, and debris in the sewage interferes with the current meter operation. The Froude numbers of the flows were all greater than one, indicating that all the tests were on steep pipes where supercritical flow was occurring.

There have been more field tests of sewers fabricated from other materials. Johnson (1939-1940) conducted field tests on 3 non-circular sewers and one circular sewer. Johnson measured flow depth and velocity to determine the resistance coefficients. A number of techniques to determine the velocities were evaluated. One was the dye-velocity method. Dye was injected at the upstream manhole, and the time of its appearance at the downstream manhole was noted visually, as was the approximate time of the most intense colour and the time when the dye could last be detected. The time of the appearance of the dye was easily determined, but the exact times of the most intense colour and the last appearance of the dye were difficult to define because of the gradual changing or fading of the colour at these times. Also, the natural discolouration of the sewage and the necessity for using artificial light in the sewers hampered these visual determinations. Johnson also evaluated the salt dilution method which was used by Pomeroy (see below), using both chemical analysis of discrete samples and continuous measurement of resistance. The latter was done using copper electrodes connected to an 8 volt power supply and a meter reading to 300 mA. The electrodes were suspended in the sewage, all other equipment and the observers being above ground. Readings of the milliameter were taken at 2 s or 5 s intervals and were plotted against time. The centroid of the resulting curve was taken to indicate the time of passage of the salt. Johnson found that the electrical resistance method was the easiest, fastest and most accurate of the methods evaluated.

Johnson reported an average Manning n of 0.0201 at a relative depth of 0.115 for the 2 ft (610 mm) circular brick sewer that he tested. Like others, Johnson found that n was higher for low flows. These n values were higher than those found by others for conduits carrying clean water. He noted that this was probably because of the thick coating of slime and grease deposited on the sewer walls over time.

Schmidt (1959) conducted tests on a large trunk sewer (span 8 ft 6 in (2.6 m) height 9 ft 5 3/4 in (2.9 m)) built in 1926 and designed to carry both sanitary sewage and storm water runoff. The flow depth was measured in the sewer and the electrical resistance method employed by Johnson was used to determine the velocity. Schmidt also found that Manning n varied with flow depth. During the field observations, Schmidt discovered a short section of sewer which was 0.20 ft (61 mm) above the general invert gradient. Repeated observations indicated that although this condition had no measureable effect on depth of flow or velocities at high flows, at low flows a noticeable damming effect was created.

Field tests on small concrete, asbestos-cement and clay tile sewer lines were carried out by Pomeroy (1964). Pomeroy tested 95 lines using the salt dilution method. Salt was added to the sewer over a short interval of time, usually about 2s but sometimes as long as 10s if a greater spread in the downstream peak of salt concentration was desired, and the variation in salt concentration with time was measured at the next manhole downstream from discrete samples. After subtraction of the background concentration the discharge was determined from the area under the concentration-time curve and the known mass of salt. The flow velocity was obtained from the centroid of the concentration-time curve.

The largest diameter tested by Pomeroy was 24 in (610 mm) and the smallest was 6 in (150 mm). A large number of lines were 8 (200 mm), 10 (250 mm) and 12 inches (300 mm) in diameter. A large range of slopes, from steep to flat, were included. These measurements indicated that for a relative depth of 0.25 the average Manning n was 0.0122 for 34 measurements on asbestos-cement pipes, 0.0136 for 31 measurements on vitrified clay pipe, and 0.0165 for 11 measurements on concrete pipe.

The most comprehensive study of in-service hydraulic roughness of sewers was made by Ackers, Crickmore, and Holmes (1964). This study comprised 340 field experiments on sewers at 20 sites. All but one of the sewers carried perennial foul sewage, the remaining one being a storm water overflow conduit. The ages of the sewers ranged from two to one hundred years and were large, varying from 15 in (381 mm) to a 5 ft 6 in by 3 ft (1.7 x 0.9 m) culvert. Invert gradients were from 0.045 to 0.00043. The conduit materials were concrete, vitrified clay tile, brick, steel lined with bitumen, and brick lined with steel.

Ackers et al. pointed out that it is better to measure velocity directly than to estimate it from the discharge and cross-section, as relatively small discrepancies in measured depths can give considerable errors in the velocity deduced by the indirect method. Ackers et al. used a radioactive tracer to measure the velocity. No sampling was involved. The time of travel of a tracer cloud over a known length was determined remotely. In 16 of the sewers, the radioactive-tracer velocity method was used in over 200 separate velocity determinations. Three of the sites were investigated by two methods, the salt-velocity and 'radio-chemical dilution', and the rest of the field work was based on the 'dilution method'. Ackers et al. do not indicate whether they measured the velocity or discharge using the 'radio-chemical dilution' method. Altogether 63 salt-velocity and 56 'radio-chemical dilution' tests were made.

Two radioactive tracers were used for tne velocity measurements - Sodium 24 and Iodine 132. Sodium 24 was in the form of sodium bicarbonate pellets which were dissolved in acetic acid on site, using remote handling tongs. This solution was then poured into a spring loaded 'pop-valve' injector. Sodium 24 has a conveniently short half-life of 15 hours and emits hard gamma rays which are not absorbed much in the surrounding fluid and are readily detectable by a large-area geiger counter. The dosing rate was well below the accepted safe level.

The injection valve, when triggered, released about a millilitre of solution, with an activity of the order of a millicurie, in a very brief period of a millisecond or so. The injection was through a spray nozzle below the water surface, so that immediate mixing over a major part of the cross-section was assured. A geiger counter tube was suspended a few inches above the surface at the next manhole downstream and its output signal was recorded. A cable from the injection valve supplied a 'time of injection' signal and a corresponding event mark on the same chart so that the time of travel of the tracer was readily scaled to the centre of the geiger counter tracer, using the mid-ordinate rule which is an approximate method of determining the centroid of the time versus increased radioactivity plot. Consideration of an alternative tracer arose from the desire to carry out a series of tests over a period of a week or more, for which a supply of Sodium 24 with its comparatively short half life would have been inadequate. Iodine 132 can be converted to Tellurium 132 which has a half life of 78 hours, thus providing a source of tracer sufficient for a 4 to 5 day test series.

Vernier point gauges installed at each end of the test length were read at intervals depending on the stability of flow conditions. The depth of flow was averaged from a number of readings, either over the whole test period, or over several distinct parts of this period for each of which the flow remained approximately constant. The test length and invert slope were measured on completion of a series of runs. Observations were also made of the temperature of sewage, the condition of the conduit above and below water level, the condition and alignment of joints, and the presence and extent of sliming and silting.

Ackers et al. made a check on the adequacy of the mixing of the tracer. At two sites (in sewers of fairly large diameter) testing was carried out first in the usual manner with the timing section defined from the injector to a single geiger counter, and then the tests were repeated with a second geiger counter at the next manhole downstream from the first. As the discharge varied slightly between the tests, the velocities were reduced to comparable conditions by assuming the Manning equation to be correct over a small range of flow depths, i.e.

$V \alpha R^{2/3}$. Agreement between the single station and double station methods was within 4%. Further tests at the same sites with the position of the injector in the cross-section of flow varied for successive shots showed no measureable effect on the velocity recorded, and thus demonstrated that there was adequate mixing to permit the adoption of the simpler, single station method.

From the test data Ackers et al. evaluated the hydraulic roughness, k_s, of the sewers using the Colebrook-White equation. A wide range of roughness values were found, the limits being 0.009 mm in a clean 15 in (381 mm) diameter concrete pipe with well aligned joints to 121.9 mm in a slimed 27 in (686 mm) diameter concrete pipe with sediment on the invert causing standing waves in the test length. Both of these sewers had been laid only two years.

For precast concrete pipe sewers a typical value of roughness was 1.52 mm when sliming up to 1/4 in (6.4 mm) was present. In two concrete sewers with no sliming the much lower values of 0.030 mm and 0.009 mm were recorded (no greater than the values obtained in laboratory tests on new pipes), while slime in excess of 3/8 in (9.5 mm) or sediment in the invert gave significantly higher values. Only two vitrified clay pipes were tested. An 18 in (457 mm) diameter sewer with a thin slime layer gave a value of k_s of 0.9 mm compared with an anticipated value when new of 0.15 mm, and a 15 in (381 mm) diameter Oxford sewer with a slime layer up to 1/2 in (127 mm) thick gave a roughness value of 18.3 mm. The brick sewers tested had roughnesses in the range 3.1 mm - 15.2 mm, except for a site where standing waves were present in the test length. In general, the brickwork had good alignment, with joints varying from well-filled to slightly open. The two lined sewers tested gave values approximating those of the concrete pipes.

Ackers had anticipated that the measured roughness under free surface conditions might depend on the Froude number and that the boundary shear would also influence roughness through its effect on the growth of slime. However, when roughness was plotted against the Froude number, and the boundary shear parameter, RS, (where S is the invert slope) the correlation was not very clear. There was a distinct trend to high roughness values at Froude numbers under 1, where sliming was present, but Ackers et al. could not conclude whether this was due to free surface effects (which would disappear under full bore conditions) or the limited growth of slime that can be maintained at the high velocities (and hence high shears) that are implicit in the upper range of Froude numbers. Apart from the two cases where coarse sediment was present, the data did indicate a general reduction in k_s with increased shear stress at the boundary.

When Ackers et al. plotted the approximate slime thickness against the boundary shear parameter, RS, and the velocity of flow, it was not possible to establish a clear cut relationship between these variables. This may have been partly due to the fact that observations of pipe conditions were obtainable only at the ends of the test lengths. From the data no direct relationship between age and degree of sliming was evident and it appeared that a group of two year old sewers tested were as fully 'matured' as much older pipes.

Ackers et al. concluded that hydraulic roughness depends on the thickness and character of the slime layer. A growth of 1/8 in (3.2 mm) or less has little effect on pipe capacity but beyond this figure the resistance to flow increases rapidly with slime thickness.

Ackers et al. also concluded that sediment in the invert of a sewer can have widely varying effects on the roughness value, depending on its bed form and the flow conditions. It can either increase resistance to flow many times, or alternatively have little influence on flow capacity other than to decrease the available cross-sectional area of pipe.

Recently Ackers and Pitt (1984) surveyed 15 tunnels lined with precast concrete segments, ranging in diameter from 1,000 mm to 4,270 mm. The research consisted of two parts:

1. Surveys of 15 tunnels taking note of the tunnel alignment, the steps between adjacent segments, the type of lining and the ground conditions.
2. Hydraulic tests on 5 tunnels (2 flowing full as water supply conduits, and 3 flowing part-full as drainage conduits).

They found that the main source of hydraulic resistance in segment-lined tunnels comes from the joints between segments. The main findings of this study were:

1. The average values of hydraulic roughness, k_s, for the five tunnels tested were in the range 0.5 to 2.1 mm.
2. A correlation exists between hydraulic roughness and mean absolute joint step height and spacing.
3. The surface finish of precast concrete segments and the condition of the jointing compound were generally good, and therefore, unlikely to contribute significantly to hydraulic roughness.
4. Neither ageing nor deposition of thin (<2 mm) layers of slime or sediment was found to affect hydraulic roughness. However, thicker deposits might reduce the difference in hydraulic resistance between

segmental and in-situ linings.

Henderson (1984) calculated k_s values from a Water Research Centre (WRC) flow survey of 30 in-service combined sanitary and storm sewers. The materials of the sewers were brick, clayware and concrete. The largest size tested was 2,010 mm x 1,830 mm, and the smallest size was 380 mm. During site investigations the size and internal condition of the sewers were noted and measurements made of the depth of any deposits present.

The data of the WRC survey indicated a considerable range in k_s values for a given sewer material. The lowest k_s value measured was 0.2 mm at a d/D of 0.36 for a 1,140 mm diameter concrete sewer on a slope of 0.0069 and the highest k_s value was 360 mm at a d/D of 0.61 for a 610 mm diameter brickwork sewer on a slope of 0.0641.

Henderson concluded that in pipes conveying foul sewage the rapid development of slime over the perimeter wetted by the daily cycle of flow will significantly increase the roughness over that of a clean pipe. He noted that various studies confirm the rapid establishment of slime on the pipe wall accompanied by an increase in roughness. The slimed roughness is governed partly by pipe material, with the smoother surfaces tending to shed slime more readily. The flow velocity also acts as a control on the depth of slime that is able to form under the shearing action of the flow. Henderson noted that there was a greater tendency to thicker slime deposits in the flatter sewers than in the steeply sloping sewers.

Previous field studies on sewers with sediment in the invert had indicated that bedforms and the highest roughness values are associated with part-full flow when the Froude number is between 0.25 and 0.5. Washing out of bedforms occurs when the Froude number is between 0.55 and 0.9. This effect was observed in one of the sewers surveyed by the WRC. In a silted sewer at Littlehampton the roughness declined to 12 mm as relative depth increased but then began to rise again, peaking at 72 mm at Froude numbers around 0.3 and a relative depth of 0.5. As the Froude number increased beyond 0.4 the roughness declined, presumably due to washing out of bedforms, resulting in a minimum k_s of 16 mm.

The WRC data suggests that the age of a sewer gives no direct indication of the roughness. However Henderson indicated that several researchers have reported a progressive roughening of concrete and cement lined pipes, believed due to chemical interaction between the pipe surface and effluent. This phenomenon has been observed in both foul and storm sewers, the rapidity and eventual degree of roughening being dependent upon pipe composition, temperature and aggressiveness of the effluent. Vant (1963) suggests the following relationship.

$$e = 0.56 \log_{10} \text{Age} + 0.02 \text{ mm} \qquad (5)$$

where e is a depreciation coefficient (mm) to be added to the k_s value of the pipe when new, and age is measured in years. A recent survey of asbestos-cement sewers in the Middle East by Balfours Consulting Engineers, carried out in association with the WRC, reported extensive corrosion in the sewers from acid attack due to the formation of hydrogen sulphide in the sceptic sewage. This problem is greater in hot climates than in cold or moderate climates.

C. APPRAISAL OF PAST WORK

In the past design information for sewer pipes was based on laboratory tests with new pipes flowing full with clean water. These tests suggested lower roughness values than were warranted and significantly underestimated roughness values for in-service sewers.

As a result of studies using new, well-aligned pipes great emphasis came to be placed upon differences in the roughness of the pipe material surface. More recent field research suggests that the pipe material exerts little direct influence on the roughness because of the build-up of slimes and deposits. However, the development of this slime has been shown to be influenced by pipe material. Table 7 summarizes the salient features of past field testing.

No changes in design procedures for sanitary sewers have been made in North America to take account of the results of field measurements of resistance coefficients. However, in Britian new resistance coefficients for different piping materials used in sewers have been recommended. These values are given in Table 8. In addition, the results of field tests have changed design procedures for sanitary sewers in Britain. The Hydraulic Research Station (HRS) recommends using a composite roughness for sewers. Separate values of k_s are assigned to the slimed and clean portions of the pipe, then a composite roughness is determined based on the percent of the pipe perimeter which is clean. With this simple perimeter-weighted method, the composite roughness is given by:

$$k_{sc} = p_1 k_{s1} + p_2 k_{s2} + \dots p_n k_{sn} \qquad (6)$$

Table 7. Summary of past field testing

Researcher(s)	No. of Sites Tested	Pipe Material	Pipe Diameter Range	d/D Range	Slope Range	Velocity Range m/s	Roughness Range
Ackers (1964)	12	Concrete	15 in - 36 in 381 mm - 914 mm		0.045 - 0.0050	0.37 - 4.02	k_s: 0.009 - 12.2 mm
	2	Salt-glazed	15 in - 18 in 381 mm - 457 mm		0.0333 -0.00247	0.32 - 3.09	k_s: 0.91 - 18.3 mm
	4	Brick	30 in (5 ft.6in x 3 ft) 762 mm (1.7m x 0.9 m)		0.00197 - 0.00043	0.27 - 0.77	k_s: 3.05 - 81.0 mm
	1	Steel, lined with bitumen	30 in 762 mm		0.00311	1.18	k_s: 0.61 mm
	1	Brick, lined with steel	42 in 1067 mm		0.00155	0.76	k_s: 1.52 mm
Pomeroy (1967)	40	Asbestos cement	8 in - 24 in 200 mm - 610 mm	0.051 - 0.746	0.00094 - 0.0410	0.16 - 1.57	n: 00077 - 0.0409
	37	Clay Tile	8 in - 15 in 200 mm - 381 mm	0.028 - 0.672	0.00161 - 0.0425	0.17 - 1.56	n: 0.0079 - 0.0367
	15	Concrete	8 in - 24 in	0.064 - 0.761	0.0020 - 0.0162	0.25 - 1.55	n: 0.0116 - 0.0230
Bishop & Jeppson (1978)	25	PVC (plastic)	10 in & 8 in. 300 mm & 200 mm	0.063 - 0.41	0.0040 - 0.0409	0.30 - 1.82	k_s: 0.027 - 1.80 mm
Perkins & Gardiner (1982)		Vertically cast concrete	225 mm	0.16 - 0.6	0.0040	0.37 - 0.75	For full pipe flow k_s: 1.3 - 3.8 mm
		Spun concrete	225 mm	0.16 - 0.6	0.0040	0.37 - 0.75	For full pipe flow k_s: 1.8 - 4.2 mm
		Asbestos-cement	225 mm	0.16 -0.6	0.0040	0.37 - 0.75	For full pipe flow k_s: 1.2 - 2.8 mm
		Clay	225 mm	0.16 -0.6	0.0040	0.37 - 0.75	For full pipe flow k_s: 0.6 - 2.3 mm
		PVC (plastic)	225 mm	0.16 -0.6	0.0040	0.37 - 0.75	For full pipe flow k_s: 0.6 - 1.3 mm
Henderson (1984)	13	Brickwork	(2010 x 1830 mm)	0.48 - 1.00	0.0007 - 0.0641	-	k_s: 3.0 - 36.0 mm
	13	Clayware	305 mm - 700 mm	0.29 - 1.00	0.0012 - 0.0833	-	k_s: 0.7 - 12.0 mm
	4	Concrete	610 mm - 1250 mm	0.12 - 1.00	0.0031 - 0.960	-	k_s: 0.2 - 11.0 mm

where k_{sc} is the composite roughness, p is the proportion of perimeter occupied by the different texture (k_s) and the suffixes refer to the dissimilar sections. HRS suggests this simple method should be restricted to cases where roughness is not widely dissimilar ($k_{s\ max}/k_{s\ min}$ should not exceed 20). Tests indicate that the composite roughness calculated by this method tends to be higher than that observed (by up to 20%) where the scale of roughness dissimilarity approaches the above constraint.

Photographs taken during Perkins and Gardiner's tests suggest 4 distinct zones of sliming on the internal pipe surfaces for a typical daily variation in depth over a range of $0.16 < d/D < 0.6$. These are:

10% of the perimeter is heavily slimed at and below the d.w.f. depth (in the range $0.4 < d/D < 0.55$)*

40% of the perimeter is slimed (below the d.w.f. depth)

10% of the perimeter is lightly slimed (above the d.w.f. depth)

40% of the perimeter is clean.

The values given above appear to be supported by field evidence from foul sewers laid at gradients of 0.001 to 0.005. The assumed high roughness of the heavily slimed pipe wall is consistent with the findings of a number of hydraulic investigations into discrete roughness of a type similar to that presented by the bumps of slime. Equivalent sand grain roughness has frequently been observed to be up to 3 times the physical height of each element.

Based on approximated k_s values for the various sections given in

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* d.w.f. refers to the perimeter wetted by the daily cycle of flow.

Table 9, and the above values indicating the percent of the pipe perimeter which is slimed, the WRC developed estimates of typical pipe-full k_s values.

Table 8 Roughness values recommended for sewers by HRS (1983) (pipe-full flow)

	k_s (mm)	
	Normal	Poor
Slimed sewers. Sewers slimed to about half depth; velocity, when flowing half full, approximately 0.75 m/s		
Concrete, spun or vertically cast	3.0	6.0
Asbestos-cement	3.0	6.0
Clayware	1.5	3.0
PVC (plastic)	0.6	1.5
Sewers slimed to about half depth; velocity when flowing half full, approximately 1.2 m/s		
Concrete, spun or vertically cast	1.5	3.0
Asbestos-cement	0.6	1.5
Clayware	0.3	0.6
PVC (plastic)	0.15	0.3

	k_s (mm)		
Sewer Rising Mains. All materials, operating as follows	Good	Normal	Poor
Mean velocity 1 m/s	0.15	0.3	0.6
Mean velocity 1.5 m/s	0.06	0.15	0.30
Mean velocity 2 m/s	0.03	0.06	0.15

Table 9. Typical distributions of hydraulic roughness (mm) around the perimeter of sanitary sewers of various materials for flow velocity of approximately 0.75 m/s (from Henderson, 1984(a)).

Pipe Material	Clean Pipe 40%	Lightly slimed above d.w.f. level 10%	Normal sliming of invert 40%	Maximum sliming at and below d.w.f. level 10%	Typical pipe-full k_s (mm)
	Degree of sliming and typical percentage of perimeter				
PVC	0.1	0.2	1.1	6.0	1.4
Clayware	0.2	0.4	1.0	15.0	2.0
Spun Concrete	0.2	0.4	3.4	23.0	3.8
Cast Concrete	0.2	0.4	2.4	27.0	4.2
Asbestos-Cement	0.07	0.14	2.3	18.0	2.8

III. FIELD OBSERVATIONS

A. INTRODUCTION

Extensive field testing of sewers conducted in Britain has led to new recommendations for in-service roughness coefficients for different piping materials used in sewers. The purpose of the present investigation was to determine if similar conditions exist in Western Canada. Field measurements at various locations in Edmonton of the flow resistance of in-service sewers were conducted and are described herein.

As plastic (PVC) and concrete are now the most commonly used sewer piping materials in subdivisions, the emphasis in this study was on these types in the size range from 200 mm to 400 mm. A wide range of slopes, from steep to flat, was investigated. Because most of the test sites had low flows under normal circumstances, augmented-flow tests were conducted at selected sites to determine the resistance under conditions more representative of design conditions.

B. GENERAL DESCRIPTION OF INVESTIGATION

Only sites where close-to-uniform flow could be expected were investigated. This excluded sewers with a drop in the pipe invert at the manhole. Most of the sites tested had flat to moderate slopes, with the exception of two steep sites where PVC plastic pipe was tested. All of the pipes tested were between two and five years old.

Measurements were made on 16 lengths of PVC plastic sewer and 6 lengths of concrete sewer. The 16 lengths of PVC pipe included 14 single lengths (between two consecutive manholes) - 5 commercial and 9 residential sites - and 2 double lengths (between three consecutive manholes) - 1 commercial and 1 residential. The concrete sewers were all in residential areas.

In all, 117 resistance measurements were made for normal flow in the 16 lengths of PVC sewer, and 51 on the 6 lengths of concrete sewer. The relative depths at these normal flows were generally quite low.

After the normal flow tests had been conducted, augmented-flow tests were conducted for 3 PVC and 3 concrete sewers to assess the resistance to be expected under design flow conditions. In these tests the flow in the sewers was increased by adding water from a hydrant. In all, 85 resistance measurements were made on the 3 lengths of PVC sewer, and 68 resistance measurements on the 3 lengths of concrete sewer.

The field tests consisted of:

1. Discharge measurement using the continuous-injection fluorescent-tracer dilution method.
2. Velocity measurement using the salt-velocity method.
3. Flow depth measurement at the manhole using tape and weight, and
4. Average slope measurement between manholes using rod and level.

The hydraulic roughness of each sewer was determined using the Colebrook-White equation.

C. DISCHARGE MEASUREMENTS

Accurate methods of directly measuring discharge include installation of a weir device in the bottom of the manhole, and tracer dilution methods. Weir installation was considered inappropriate because:

1. A weir would prevent a direct measurement of the flow depth within the sewer.

2. Installation and reading of the head above the weir would require a considerable amount of work down the manhole. This would necessitate the presence of a city inspector and blower apparatus.
3. A weir would tend to trap tissue and debris, so altering its calibration.
4. It is desirable that the weir be calibrated in place, presumably using tracer methods to determine the discharge.

Therefore discharge measurement using tracer dilution was considered the best alternative. Previous tracer dilution studies have demonstrated that the flow rate can be measured within 5 percent with good mixing conditions.

The tracer used in this study was Rhodamine-WT. This fluoresces in the yellow-orange range of the visible spectrum, and can effectively be used as a tracer because most background fluorescence will not interfere with its analysis. In addition, Rhodamine-WT is both biodegradable and non-toxic.

A fluorometer was used to analyze the tracer concentration. This instrument measures the relative intensity of light emitted by a water sample containing fluorescent substances. The intensity of fluorescent light emitted is directly proportional to the amount of fluorescent substances present. Fluorescent tracer techniques allow detection of concentrations as low as parts per billion. This is far superior to tracer techniques using colourimetric dye or salt solutions.

Discharge measurements using tracer dilution techniques are based on the principle of conservation of mass. Fluorescent dye is injected into the sewer at a constant rate and concentration. Downstream the effluent is sampled and the sewer discharge determined using the following relationship, which is based on the assumption that the tracer is fully mixed with the effluent by the time the flow reaches the sampling station:

$$Q = q \frac{C_i}{C_m} \qquad (7)$$

where Q is the effluent flow, q the dye flow (q << Q), C_i the input concentration and C_m, the measured concentration in excess of background. Therefore, if a constant tracer input flow rate is maintained a series of discrete samples taken downstream provide a series of sewer discharge measurements. Collection of multiple discrete samples for later analysis was considered a simple and accurate method because any suspended sediment and/or sewage that was present in the sample would settle to the bottom of the container and would not interfere with the fluorometer analysis. Only the 1 ml of sample which was required for the fluorometer analysis was taken from the top of the sample.

The dye injection apparatus was set up at the upstream manhole as shown in Figures 4a and 4b. The constant tracer input rate was established using a drip bottle for the normal flow measurements. For the augmented-flow tests a positive displacement, variable speed, peristalic pump was used. In both instances periodic checks of the input concentration and flow rate were made during the injection period.

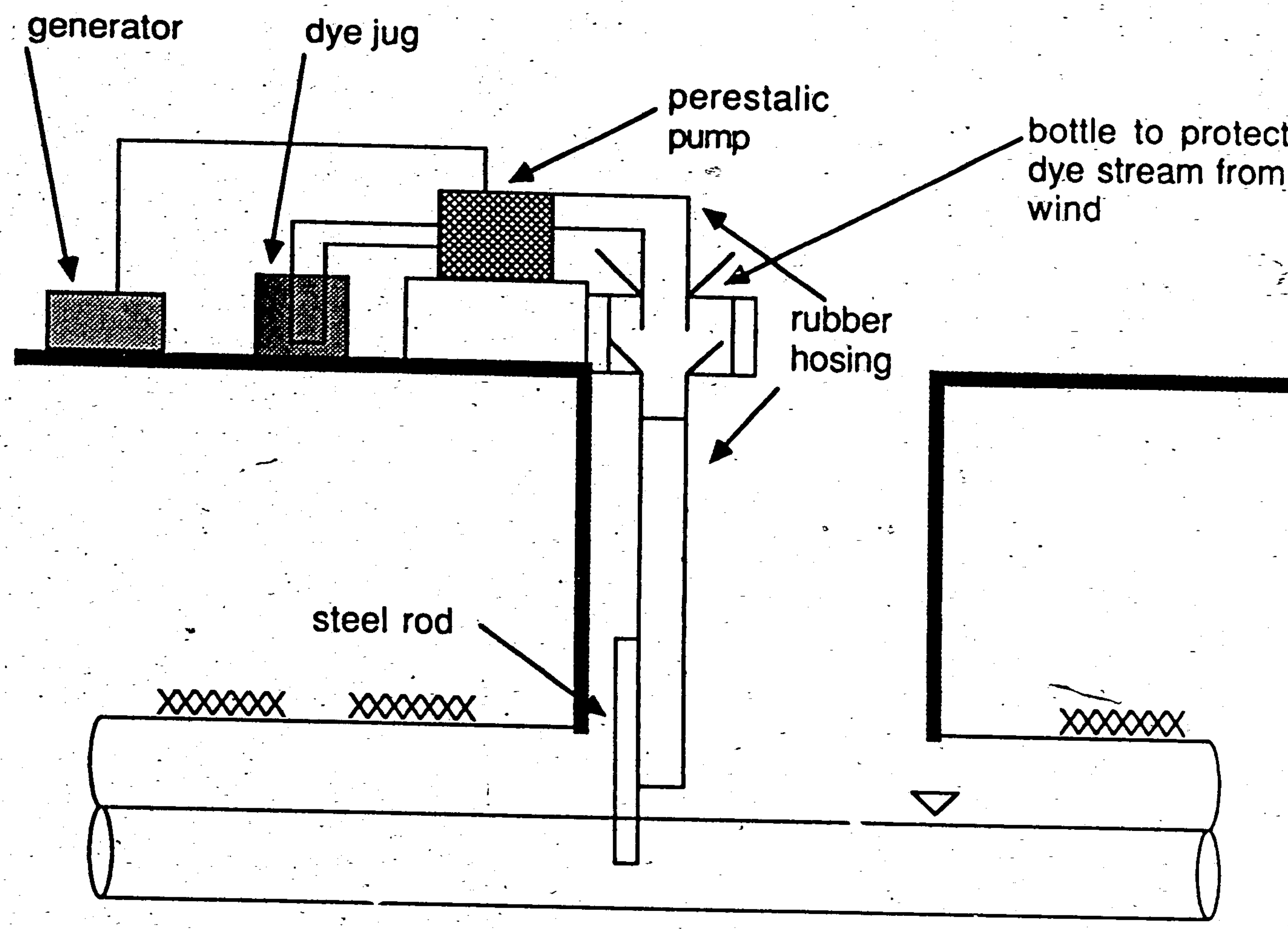

Figure 4a. Experimental set-up at upstream manhole (NTS)

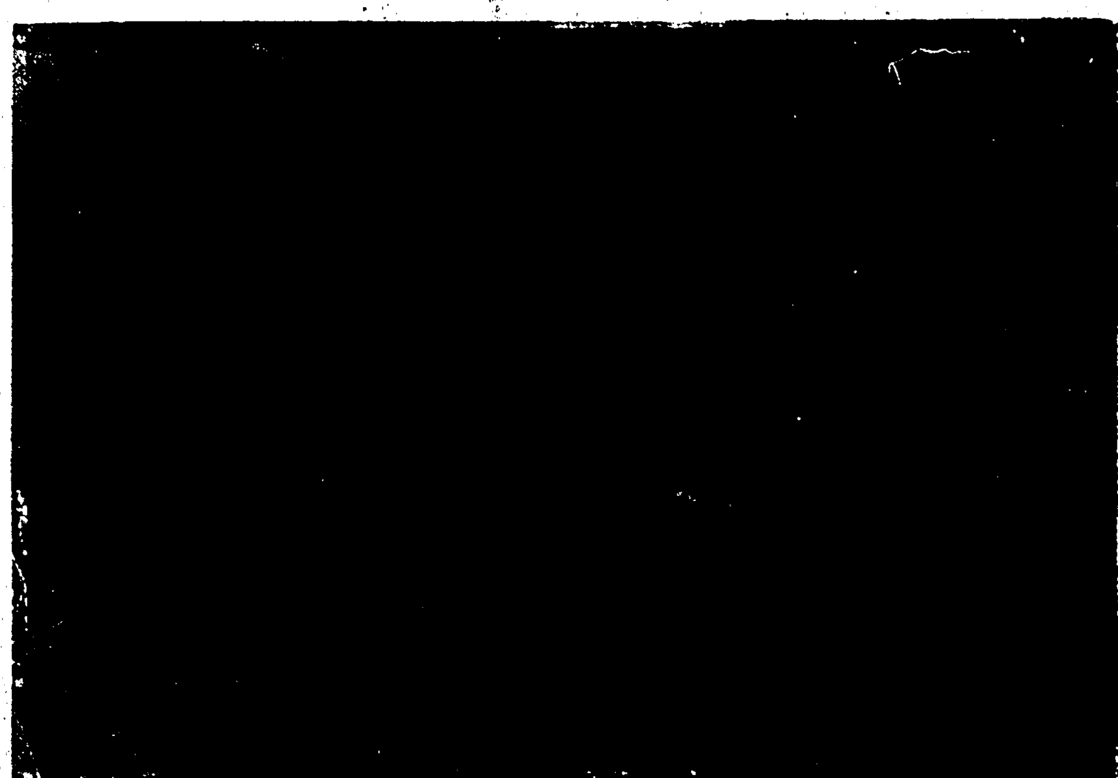

Figure 4b. Photograph of experimental set-up at the upstream manhole

For the present purposes a minimum target concentration at the downstream manhole was considered to be 300 parts per billion. For Rhodamine-WT this means that the sample would be light pink. Below this concentration significant errors would result from adsorption losses as discussed below. This target concentration was determined in the field by visual inspection. If a light pinkish colour (or darker) could be detected by the naked eye when a sample was taken, the concentration was considered to be sufficient. If no colour could be detected in the sample, the injection rate was increased until it was. Generally, a standard solution of Rhodamine-WT was used, this being 300 ml of approximately 20% solution of Rhodamine-WT mixed with distilled water in a 5 gal. jug (23 l).

Originally sampling was to be conducted using a self-contained wastewater sampling system capable of automatically collecting samples at desired intervals. However, experience during the first field test indicated the sampler intake would continually become clogged with tissue. This required the line to be manually cleaned before each sampling to ensure proper collection of the sample. For subsequent tests it was decided to simply manually draw samples from the sewer using a small bucket. Given the continous dye injection the timing of these samples is not critical. Samples were taken just after the salt peak.

After the field operation had been carried out the effluent samples were brought into the laboratory and analyzed. The samples were first diluted so that the fluorometer was operating in its linear range. The sample concentration was then measured and the flow rate in the sewer determined from Equation 7.

The fluorometer available at the University of Alberta, Environmental Engineering Laboratory was checked for calibration and scale alignment prior to analysis work. A typical calibration is shown in Figure 5. Note the calibration is linear over a large range (0.1 - 100 ppb) but at high concentrations readings are reduced due to a phenomenon called optical quenching.

The most significant errors associated with discharge measurements using fluorescent tracers are caused by dye adsorption and light scatter caused by suspended sediment. The light scatter problem may be solved by allowing sediment to settle out before analysis, or by centrifuging the sample.

Adsorption errors are caused by loss of tracer to surfaces of the pipe, sample container, and any sediments present. Unfortunately there is no way to prevent these losses. However by working with sufficiently high concentrations the losses may be reduced to an insignificant level. Losses may also be reduced by using a dye such as Rhodamine-WT which is less susceptible to adsorption problems. To aid in establishing an appropriate dosage level adsorption tests were conducted on samples of influent wastewater from the Gold Bar Pollution Control Plant. The results of the adsorption tests are shown in Table 10. Evidently if the sample concentration is above about 300 ppb the adsorption losses are acceptable, errors being less than 4 percent.

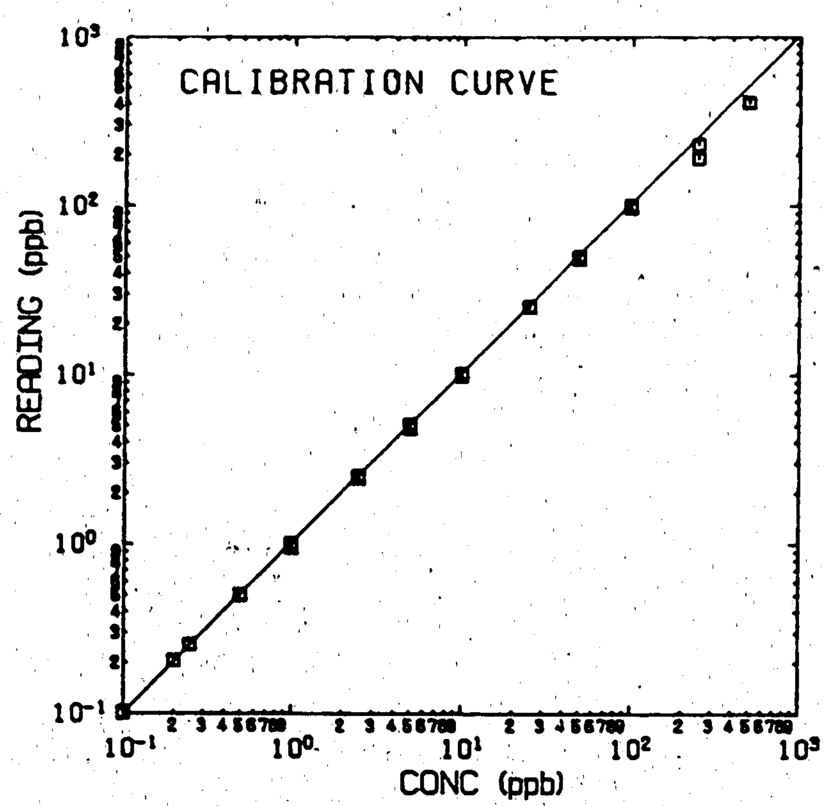

Figure 5. Typical Calibration Curve

Table 10. Adsorption Analysis - Gold Bar Influent

Dosed Concentration (ppb)	Measured Concentration (ppb)	% Recovery
1.00	0.40	40.0
2.00	0.82	41.0
10.0	4.10	41.0
100	94.5	94.5
500	490.0	98.0

D. **VELOCITY MEASUREMENTS**

Alternative methods considered for directly measuring the velocity within the sewer line were:

1. Measurement with a velocity meter.

2. Timing of floats travelling along the sewer line.

3. Salt-dilution method.

Velocity meter measurements were not considered a viable method because it is difficult to obtain the mean velocity from point measurements in such shallow flows and the meters are susceptible to clogging with tissue. Furthermore, measurements would only be possible at the manholes. Even if accurate, such measurements would not give a good indication of the average velocity over the length between manholes if the flow was non-uniform.

Surface floats, although simple and convenient to use, give velocities more representative of the maximum than the mean. To use this method a relationship between the maximum and the mean velocity would have to be established.

The salt-dilution method was considered the best. In this method a small volume of salt solution is instantaneously injected into the sewer at a manhole. The conductivity of the effluent in excess of background conductivity at the manhole downstream can then be monitored and recorded as a continuous function of time. The set-up at the downstream manhole is shown in Figures 6a and 6b. An example of the variation in conductivity as the slug of salt passes is shown in Figure 7. The centroid of the dispersion curve gives the mean travel time between the upstream injection point and the probe. The average velocity is then calculated from:

$$V = \frac{\bar{t}}{L} \qquad (8)$$

where L = distance between manholes

$\bar{t}$ = time to centroid of dispersion curve(s) from injection time. This represents an average with respect to both distance along the sewer and position within the flow cross-section.

The volume of salt solution in the injection bucket was generally about 800 ml, and it was poured from a height of 0.2 to 0.3 m above the water surface to assure that there was good mixing at the point of injection. With good mixing at the point of injection it is estimated the error of this method of determining velocity would be less than 1%.

The only problem encountered was that at very low flows the conductivity probe in the sewer would become clogged with tissue paper. When this occurred the dispersion curve would exhibit a very long tail and would not come back to zero. Where this occurred tests were repeated. Wash water discharged into the sewer line during a test also caused the conductivity of the effluent to increase considerably and this caused the dispersion curve on the chart recorder to peak. When this occurred it was quite obvious, and those tests were also rejected.

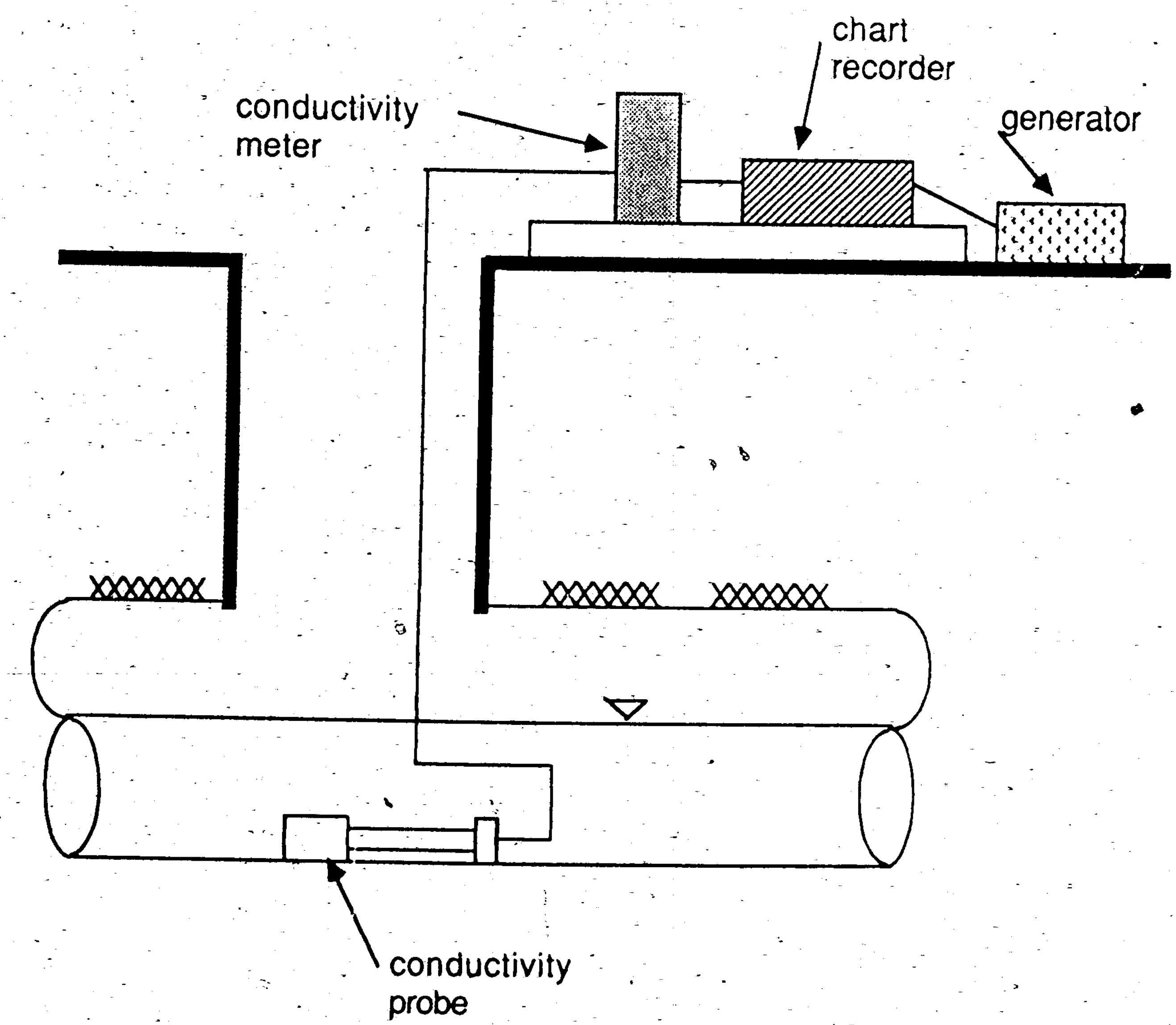

Figure 6a. Experimental set-up at downstream manhole (NTS)

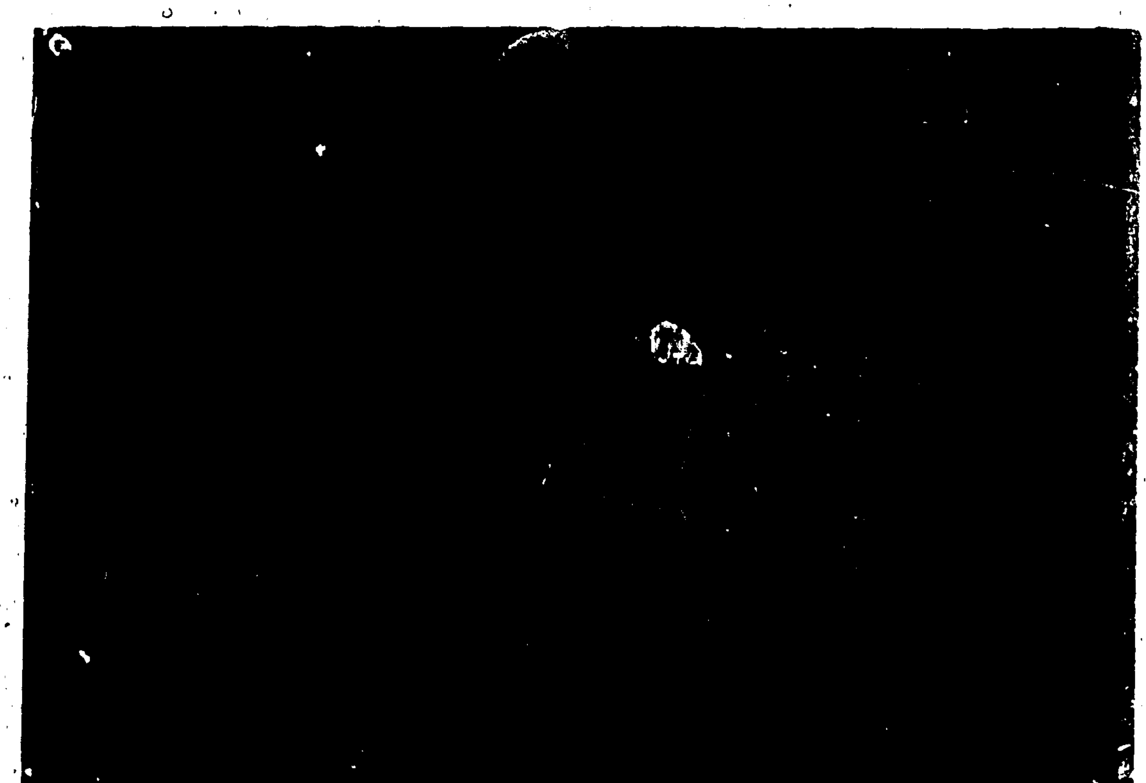

Figure 6b. Photograph of experimental set-up at the downstream manhole

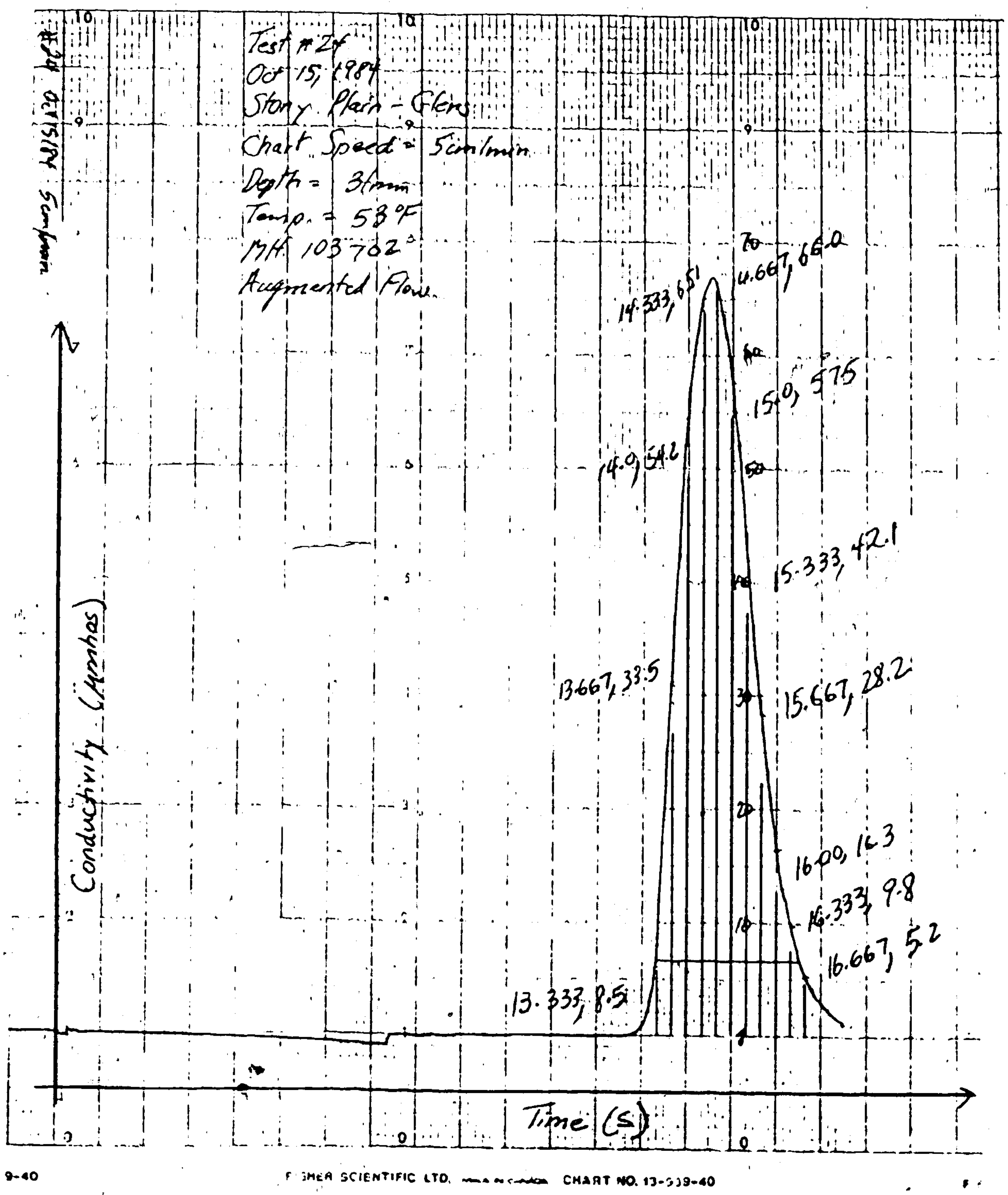

Figure 7. Typical dispersion curve of salt slug.

E. DEPTH MEASUREMENT

Depth measurements at the manhole could be made simply and accurately using a point gauge or scale at the bottom of the manhole. However, this method was not feasible because safety procedures require the manhole to be vented with a blower and a safety harness be worn every time the manhole is entered. A method of monitoring the water level from the surface was desirable. Several methods were investigated. They included:

1. capacitance probes;
2. dipping meter;
3. sonic level recorder;
4. bubbler-manometer system; and
5. tape and weight.

The capacitance probes, dipping meter and sonic level recorder were all considered unsatisfactory. The first two primarily because of problems which would result from tissue paper catching upon the probes. The sonic level recorder appeared promising but laboratory investigations indicated the calibrations were unstable at the very high sensitivity setting required for the proposed measurements. A bubbler-manometer system was therefore selected for initial tests.

The air line to the probe and manometer was pressurized from an air cylinder-regulator apparatus. The pressure could be adjusted to just allow air to bubble from the end of the probe. This pressure was measured with the manometer. A correction for the distance between the probe foot and air outlets and minor surface tension effects must be applied to the manometer reading. Laboratory testing of the system indicated a stable correction independent of the depth being measured.

During field tests the probe was suspended from a platform spanning the manhole lid. Sufficient slack was provided in the air line to allow the foot of the probe to rest upon the pipe bottom. The probe was manually raised and lowered to allow it to 'walk' to the pipe invert and then the line clamped in place. Experience showed that the probe must be cleaned of any tissue and relocated before each individual measurement.

Subsequently, it was decided to simply use a weight attached to a measuring tape to obtain the depth measurements. The weight was suspended from a reference point on the manhole rim until it just came into contact with the water surface. The depth to the sewer invert was also measured. The accuracy of these depth measurements would be ±2 mm.

The depth measurements at the manholes were not used directly in the analysis. Instead the depth was computed from the measured average velocity and discharge. This computed depth would be more representative of average conditions along the sewer length than a depth measurement at each manhole. Because the depth measurements were not used in the analysis only one depth measurement, generally at the downstream manhole, was taken to obtain a rough check on the calculated depth.

It was felt that irregularities in the pipe profile (eg. sag) may make a significant contribution to the apparent hydraulic roughness of a line. A device to measure the depth of flow at locations between manholes was therefore constructed and used at two sites. The device was similar to the manometer apparatus previously used to measure the flow depths, but with a horizontal air probe, which could be pulled along the invert, substituted for the vertical probe. It is described in more detail in Appendix 4. The horizontal air probe was used to measure the variation in depth of flow along two in-service sewer inverts: MH 23-22 in Riverbend and MH 17-15 in Thorndale Industrial. The average flow depth measurements ranged between 30-50 mm during the testing period. Simultaneous depth measurements were made at the upstream manhole and various positions along the sewer invert. The ratio of depth at the manhole to that at various positions along the invert are shown in Figure 8. It would appear that the flow at the Thorndale site was near uniform whereas that at Riverbend may not have been, but the scatter makes it very difficult to discern.

Slime deposits around the perimeter of the pipe were not sampled in detail at the manholes because previous studies indicated that there was not a good correlation between slime thickness and weight on the pipe perimeter and hydraulic roughness. Visual observations were made at the manholes, and the condition of the sewer pipe, and any depositions of slime and sediment noted. Sediment depths were measured at both the upstream and downstream manholes. The averages of these two depths were assumed to exist along the entire length of sewer. The depth to the surface of the sediment was measured by a tape and weight suspended from a reference location on the manhole rim and the depth to the invert was also measured with a sharp pointed probe attached to a tape.

To summarize, the general procedure that was finally developed was as follows:

1. The dye injection apparatus was set up at the upstream manhole and the desired mass flow rate set. This was determined by trial and error by visually inspecting a sample drawn from the downstream manhole. If the dye colour was easily visible then the injection rate was sufficient. The equipment was then let run for a period

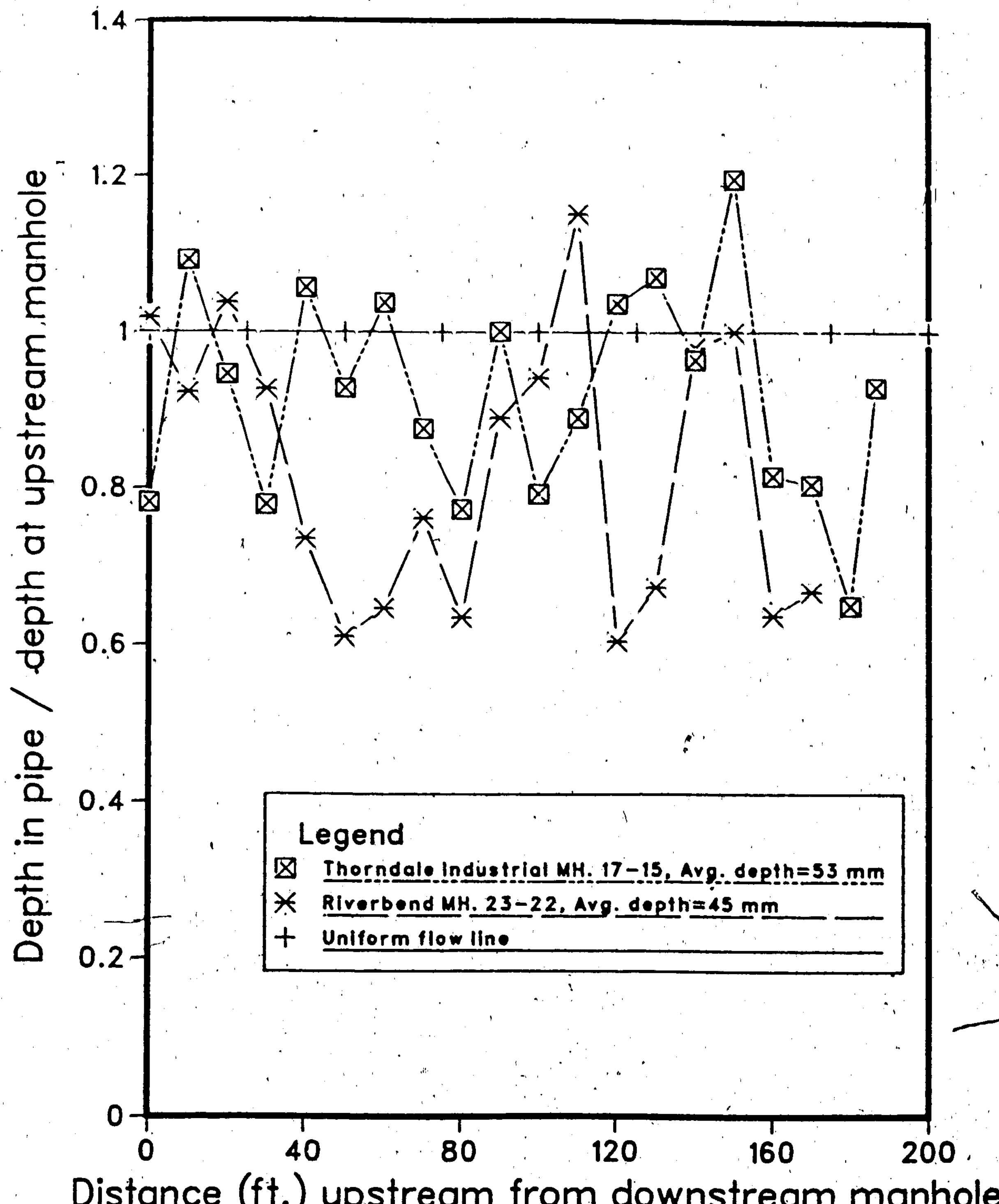

Figure 8. Variation of flow depth along pipe as determined by 'Air Probe'

sufficient to stabilize and to allow any initial dye adsorption to the sewer walls to occur. Generally this was about 15 minutes.

2. The conductivity meter, probe and chart recorder were then set up at the downstream manhole.
3. The dye feed was sampled for concentration determination and its flow rate measured. This step was repeated periodically throughout the entire test period to ensure a constant mass flow rate was maintained.
4. The conductivity probe was checked to ensure it was clear of tissue and debris.
5. The slug of salt solution was injected at the upstream manhole and the chart recorder was started simultaneously at the downstream location. The quantity of salt solution required was determined by trial and error. A jug of tap water and concentrated salt solution was on site. The salt solution and tap water were mixed in the injection bucket in various proportions depending on how high the dispersion curve peaked. Before testing was begun, initial trial runs were made to determine the mix of salt solution and water to inject at the upstream manhole.
6. The depth to the water surface was measured from a reference location on the manhole rim using a weight and tape.
7. After the slug of salt solution had passed a sample of effluent was taken for discharge determination.
8. Steps 4 to 7 were repeated for each measurement, with the time and the conductivity chart recorder speed noted for each test. Generally, six or seven discharge and velocity measurements were taken for the normal flow tests.

9. The effluent temperature was measured at some time during the tests.

10. The slope of the sewer line, as indicated by the manhole invert elevations, was surveyed either before or after the test.

11. The dye samples were brought into the laboratory and analyzed within two days after field testing.

Initially this testing procedure was conducted with the normal flow in the PVC and concrete sewer lines. Later augmented-flow tests were conducted at selected sites by adding water, from a nearby fire hydrant, to the sewer at a manhole located one or two manholes upstream of the test section.

The procedure in the augmented-flow tests was to start the test at low depths and then to increase the flow in steps with the hydrant water until the maximum hydrant capacity was reached. The flow was then gradually reduced in steps to approximately its original level. Measurements for the normal flow and at each step were not begun until uniform flow had been re-established at the upstream manhole of the test section, which was generally, 15 to 20 minutes. Generally, three sets of measurements were made for each flow increment, except at the peak flow, for which 5 or 6 sets were made. Each flow increment was maintained for 15 to 30 minutes. At the higher flows less time was required to complete the measurements because the velocities were greater.

IV. RESULTS AND ANALYSIS

The average depth in the sewer was determined from the average flow area given by:

$$A = \frac{Q}{V} \tag{9}$$

where Q = discharge determined from the dye dilution measurements

V = average velocity determined from the salt-velocity measurement.

The hydraulic radius, R, was then calculated from the average flow area and the properties of a circle, and Manning n determined from

$$n = \frac{R^{2/3} S^{1/2}}{V} \tag{10}$$

The equivalent sand grain roughness, k_s, was determined from the Colebrook-White formula:

$$k_s = 14,800 \left(\mathrm{Exp}\left(\frac{-1}{0.86\sqrt{f}}\right) - \frac{2.51}{R\sqrt{f}}\right) \tag{11}$$

where R = Reynolds number $= \frac{4VR}{\nu}$

and ν = kinematic viscosity (m^2/s).

Tables 11 and 12 give the results obtained for the plastic and concrete sewers at normal flows. Table 13 gives the results for the augmented-flow tests at the maximum d/D attained. Additional sites had

Table 11. Resistance Measurements on PVC Plastic Sewer Lines - Normal Flow

Location	Nom. Dia (mm)	Services	** Sediment Depth (mm)	$^*\varrho_{max}/\varrho_{min}$	Comments
Thorndale (commercial)					
MH 19-17	300	none	15	1.6	oily sediment
MH 17-15	300	none	15	1.3	oily sediment
MH 15-12	300	none	9	1.5	oily sediment, suspect backwater profile due to debris
MH 19-15 (double)	300	none	15	1.5	oily sediment, test through three consecutive manholes
Stony Plain (residential)					
MH 103-102	250	none	0	2.8	cleaned several weeks before measurement
MH 102-101	250	none	0	2.0	cleaned several weeks before measurement
MH 103-101 (double)	250	none	0	1.6	test through three consecutive manholes
Devon (residential)					
MH 49-48	300	none	34	1.4	sandy sediment, suspect backwater profile due to debris blockage
Riverbend (residential)					
MH 66-65	200	10	0	3.1	supercritical flow, large variation in flow rate
MH 23-22	300	2	10	2.7	a sample of sediment was taken and analyzed in the lab
MH 26-25	200	3	0	2.0	
MH 23-22	300	2	0	2.0	line was cleaned before testing
Burnewood (residential)					
MH 303-302	200	12	0	1.8	

Table 11. Resistance Measurements on PVC Plastic Sewer Lines - Normal Flow (continued)

Location	Nom. Dia (mm)	Services	** Sediment Depth (mm)	*Q_{max}/Q_{min}	Comments
Leduc Southpark (residential)					
MH 614-613	200	~11	0	3.1	very small patches of sediment, large variation in flow rate
Leduc Romulus (commercial)					
MH 10 -10A	250	none	22	2.5	sandy sediment, suspect backwater due to debris blockage
MH 9-10	250	none	12	2.3	sandy sediment, suspect backwater due to debris blockage
Spruce Grove (residential)					
MH 124-123	200	~1	0	2.9	cleaned approximately 1 week before measurements, large variation in flow rate

Note:
* Ratio of maximum discharge to minimum discharge measured during the normal flow tests.

** Average of sediment depths measured at the upstream and downstream manholes.

Table 11. Resistance Measurements on PVC Plastic Sewer Lines - Normal Flow (continued)

Location	Slope %	Length Between Manholes (m)	Average Q (L/s)	Average V (m/s)	Average d/D	Average n	Average k_s (mm)	Standard deviation of k_s (mm)	No. of Measurements
Thorndale (industrial)									
MH 19-17	0.275	60.05	2.29	0.313	0.13	0.016	3.8	0.8	6
MH 17-15	0.393	58.28	1.88	0.320	0.11	0.016	4.7	0.5	
MH 15-12	0.356	109.52	1.38	0.274	0.10	0.017	6.2	2.3	6
MH 19-15	0.333	118.33	1.03	0.220	0.09	0.020	9.7	3.2	6
Stony Plain									
MH 103-102	0.180	49.47	0.82	0.215	0.13	0.015	2.6	0.5	13
MH 102-101	0.212	93.87	0.71	0.208	0.12	0.016	3.5	0.5	7
MH 103-101	0.200	143.34	0.92	0.194	0.15	0.019	8.8	3.4	7
Devon									
MH 49-48	0.148	78.26	1.40	0.190	0.11	0.018	7.5	2.9	7
Riverbend									
MH 66-65	1.41	92.45	0.35	0.461	0.06	0.010	0.2	0.1	6
MH 25-26	0.340	53.90	1.39	0.279	0.10	0.017	5.1	3.0	7
MH 26-25	3.12	119.80	1.38	1.01	0.09	0.0088	0.1	0.1	6
MH 23-22	0.340	53.90	1.26	0.243	0.13	0.020	10.2	3.7	7
Burnewood									
MH 303-302	0.570	67.09	4.79	0.432	0.38	0.021	15.5	4.7	9
Leduc Southpark									
MH 614-613	0.415	94.72	0.57	0.179	0.16	0.026	22.5	11.3	7
Leduc Romulus									
MH 10-10A	0.233	74.62	0.48	0.086	0.13	0.048	85.2	45.8	8
MH 9-10	0.357	70.96	0.41	0.141	0.08	0.027	21.8	11.4	9
Spruce Grove									
MH 124-123	0.568	68.72	0.65	0.274	0.13	0.017	5.4	2.4	7

Table 12. Resistance Measurements on Concrete Sewer Lines - Normal Flow

Location	Nom. dia (mm)	Services	** Sediment Depth (mm)	* $\frac{Q_{max}}{Q_{min}}$	Comments
Yellowbird-17 Ave. (residential) MH 315-317	300	1	0	1.7	no sediment in line, pieces of mortar from manhole top appear to have dropped into the sewer in the past.
Yellowbird-16 Ave. (residential) MH 341-342	200	1	0	1.8	small amount of sandy, gritty, sediment on the bottom; sandy, gritty, slime on side walls; upstream manhole has a lot of paper obstruction upstream of the dye injection.
Yellowbird - 107 St. (residential) MH 350-348	300	3	0	1.3	
Lake District-95 St (residential) MH 204A-204	380	0	0	1.2	large flow; no sediment; slime growth on side walls; had to measure the water surface level at the u/s manhole and d/s manhole simultaneously to determine the slope of the hydraulic grade line.
St. Albert LaRose Dr. (residential) MH 101-101	250	0	7.5	1.6	slime growth on side walls of MH 101, none at MH 100; lots of sediment at MH 101; not much at MH 100.
St. Albert McKenney Ave (residential) MH 17-16	200	0	0	2.5	no sediment; no growth on pipe walls; pipe appeared clean.

Note:

* Ratio of maximum discharge to minimum discharge measured during the normal flow tests.

** Average of sediment depths measured at the upstream and downstream manholes.

Table 12. Resistance Measurements on Concrete Sewer Lines - Normal Flow (continued)

Location	Slope %	Length Between Manholes (m)	Average Q (L/s)	Average V (m/s)	Average d/D	Average n	Average k_s (mm)	Standard deviation of k_s (mm)	No. of Measurements
Yellowbird-17 Ave. MH 315-317	0.306	54.86	2.78	0.356	0.16	0.015	3.6	1.1	10
Yellowbird-16 Ave. MH 341-342	0.500	59.74	0.14[illegible]	0.0804	0.10	0.051	53.7	17.5	6
Yellowbird-107 St. MH 350-348	0.331	81.32	3.94	0.339	0.22	0.020	11.8	1.3	10
Lake District-95 St. MH 204A-204	0.0270	61.17	8.62	0.161	0.48	0.021	21.7	12.1	6
St. Albert LaRose Dr. MH 100-101	0.379	77.28	2.40	0.362	0.17	0.016	4.3	0.7	9
St. Albert McKenney Ave. MH 17-16	0.941	64.10	0.87	0.338	0.13	0.019	7.3	2.2	10

Table 13. Resistance Measurements for Augmented-Flow Tests for Maximum Flow Depth Attained During the Tests.

Location	Nom. Dia, (mm)	Slope %	Length Between Manholes (m)	Average Q (L/s)	Average V (m/s)	Average d/D	Average n	Average k_s (mm)	Standard deviation of k_s (mm)	No. of Measurements
Thorndale Industrial* MH 17-15 (Plastic)	300	0.393	58.28	38.55	0.921	0.58	0.0128	1.33	0.008	5
Stony Plain MH 103-102 (Plastic)	250	0.180	49.47	25.54	0.647	0.74	0.0117	0.67	0.094	5
Leduc Romulus MH S11-S12 (Plastic)	250	0.354	80.99	18.09	0.841	0.45	0.0106	0.31	0.051	5
Stony Plain MH 103-102 (Plastic)	250	0.180	49.47	27.13	0.646	0.79	0.0118	0.70	0.074	6
124 Ave. & 43 St. MH R6-R7 (Concrete)	300	0.309	107.25	24.53	0.680	0.50	0.0146	3.21	0.007	5
69 Ave. & 42 St. MH A14-A13 (Concrete)	300	0.484	122.22	28.94	0.993	0.42	0.0117	0.69	0.091	6
Yellowbird - 17 Ave. MH 315-317 (Concrete)	300	0.306	54.86	25.66	0.705	0.50	0.0141	2.53	0.31	6

Note:

* The minimum roughness was not measured at the max d/D for the Thorndale Industrial site. At d/D = 0.26, the minimum roughness, k_s = 0.58 and Manning n = 0.0113.

** At the Leduc Romulus site flow tests were only conducted to the maximum d/D, and the flow was not brought back down to approximately its original level because the downstream manhole flooded out.

*** The Stony Plain site was retested because on of the dye jugs became contaminated during the initial tests.

to be found to conduct augmented-flow tests for the concrete sewers because most of the sites where normal flow tests were conducted did not have a hydrant nearby. Detailed results are contained in Appendix 6.

At two sites roughess measurements were made over the two lengths of sewer line between three consecutive manholes at low flows, $d/D < 0.2$. The two sites tested were between manholes 19-15 at Thorndale and between manholes 103-101 at Stony Plain. The results were much higher than the averages of the two roughness values of the individual lines for both sites tested. The reason for this is not clear.

The data in Appendix 6 shows considerable variability in the values of the roughness coefficients for any one line. This variability is not surprising, however, considering the circumstances of the measurements. In many instances the flows in the 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 flow change, the undefined transient nature of these flows can cause considerable scatter in the computed roughness coefficients. This problem was quite significant in residential areas where there were many service connections coming into the sewer line being tested. To overcome this problem to some extent, the dye injection equipment was set up upstream of the test section, and flow samples were taken at both the upstream and downstream manhole of the test section. If the indicated discharges were significantly different the test was rejected.

Accuracies for the dye dilution technique for wastewater flow measurement have been reported in the literature to range between 1 and

5%. It is estimated that the accuracy of the discharge measurements in this present study were better than 3%. The salt-velocity technique for measuring velocities in sewers is among one of the most accurate techniques known, since only the time of travel of a tracer cloud over a known length need be determined to arrive at the average velocity. Because a chart recorder was used to monitor the conductivity of the effluent as a continuous function of time, no sampling was required. The accuracy of the salt velocity method would be less than 1%. The accuracy with which the sewer slope could be determined (disregarding the unknown irregularities in the profile) was within 0.1%.

A sensitivity analysis was carried out to determine what influence the measured parameters have on the roughness coefficient. The analysis was carried out for the following extremes:

Parameter	Extremes	
Diameter	Large	Small
Relative Depth	More than half full	Low
Roughness	High	Low
Slope	High	Low

The detailed results and analysis are given in Appendix 5. For convenience in presentation the variation in error of roughness at the extremes was approximated by a linear function between the error in roughness and those in the measured parameters (i.e. V, Q, S) viz

$$\varepsilon_k = a\varepsilon_V + b\varepsilon_Q + c\varepsilon_s \qquad (12)$$

where ε_V = % error in V

ε_Q = % error in Q

ε_S = % error in S

ε_k = % error in k_s

and a, b, and c are coefficients. The values of the coefficients are given in Table 14. They were determined from the results given in Appendix 5. The sensitivity analysis indicates that k_s is most sensitive to V and least sensitive to Q. For example, for a 300 mm diameter pipe with a roughness of 0.1 mm, a slope of 0.0010, and a flow depth of 0.10D, if the error in the measurement of V was 1%, in Q was 3% and in S 0.1%, from Equation 12 the error in k_s would be approximately 110%.

From the sensitivity analysis and the errors in the measurements of Q, V and S discussed above, the error in k_s can range from 10 to 110%. An accurate evaluation of k_s therefore requires very great precision in measurements of the flow characteristics, particularly for low roughness and flat slopes. On the other hand the discharge capacity computed is not sensitive to the value of k_s used. For example, a three fold increase in k_s causes only about a 7% decrease in flow capacity, and a ten fold increase an 18% decrease. It should also be noted that out-of-roundness, or ellipticity, has only a minor effect in reducing the cross-sectional area. For a 5% out-of-roundness, the reduction in area is less than 1%.

Table 14. Sensitivity Analysis

Specified parameters				Sensitivity parameters in Equation 12		
D (mm)	d/D	S	k_s (mm)	a (V)	b (Q)	c (S)
300	0.11	0.0010	0.066	62.1	15.6	19.6
	0.15		10.84	6.4	1.9	1.9
	0.09	0.010	0.23	16.6	4.0	5.0
	0.12		10.19	6.0	1.8	1.8
	0.65	0.0010	0.079	37.2	5.9	13.0
			16.12	6.4	1.3	2.2
	0.67	0.010	0.17	16.8	2.5	6.0
			16.30	6.1	1.2	2.2
800	0.09	0.0010	0.088	44.5	10.8	13.3
	0.12		14.40	7.0	2.0	2.1
	0.09	0.010	0.21	18.6	4.3	5.4
			13.40	6.8	1.9	2.0
	0.64	0.0010	0.039	52.6	7.6	17.8
			19.86	7.6	1.4	2.6
	0.69	0.010	0.091	20.6	2.8	7.3
			20.00	7.1	1.3	2.7

V. DISCUSSION

The hydraulic roughness determined from the normal flow measurements are shown in Figures 9 and 10. It is evident that at these flows the flow resistance is significantly affected by more factors than just the pipe material. No significant difference is apparent between plastic and concrete sewer pipes. Other factors include debris and solids in the flow, sediment on the pipe invert and slime on the pipe walls. Any non-uniformity in the longitudinal pipe profile and joint eccentricities due to post-construction settlement would also contribute to the apparent roughness of the pipe at low flow. The result of all these influences is that the apparent hydraulic roughness at low flow in an in-service sewer is **orders of magnitude** larger than the clean water, new pipe roughness values typically used for minimum velocity assessments in sewer design.

Because these various factors have a larger influence at low flows than high flows it can be anticipated that the effective roughness should decrease with increasing relative depth. The results of the augmented-flow tests confirm this. These are shown in Figures 11 and 12. For example, Figure 11 shows that for some lines the effective roughness continues to decrease to relative depths as high as 0.8. Similar magnitudes and variations of roughness with relative depth for in-service sewers have been reported in the literature by Ackers et al. (1964) and Henderson (1984). This is in strong contrast to the measurements for clean water and new pipe for which the effective roughness is reasonably constant for relative depths above about 0.3 as discussed earlier. (Although based on several studies (Neale and Price, 1964; Bock, 1966) this latter conclusion is at variance with the results

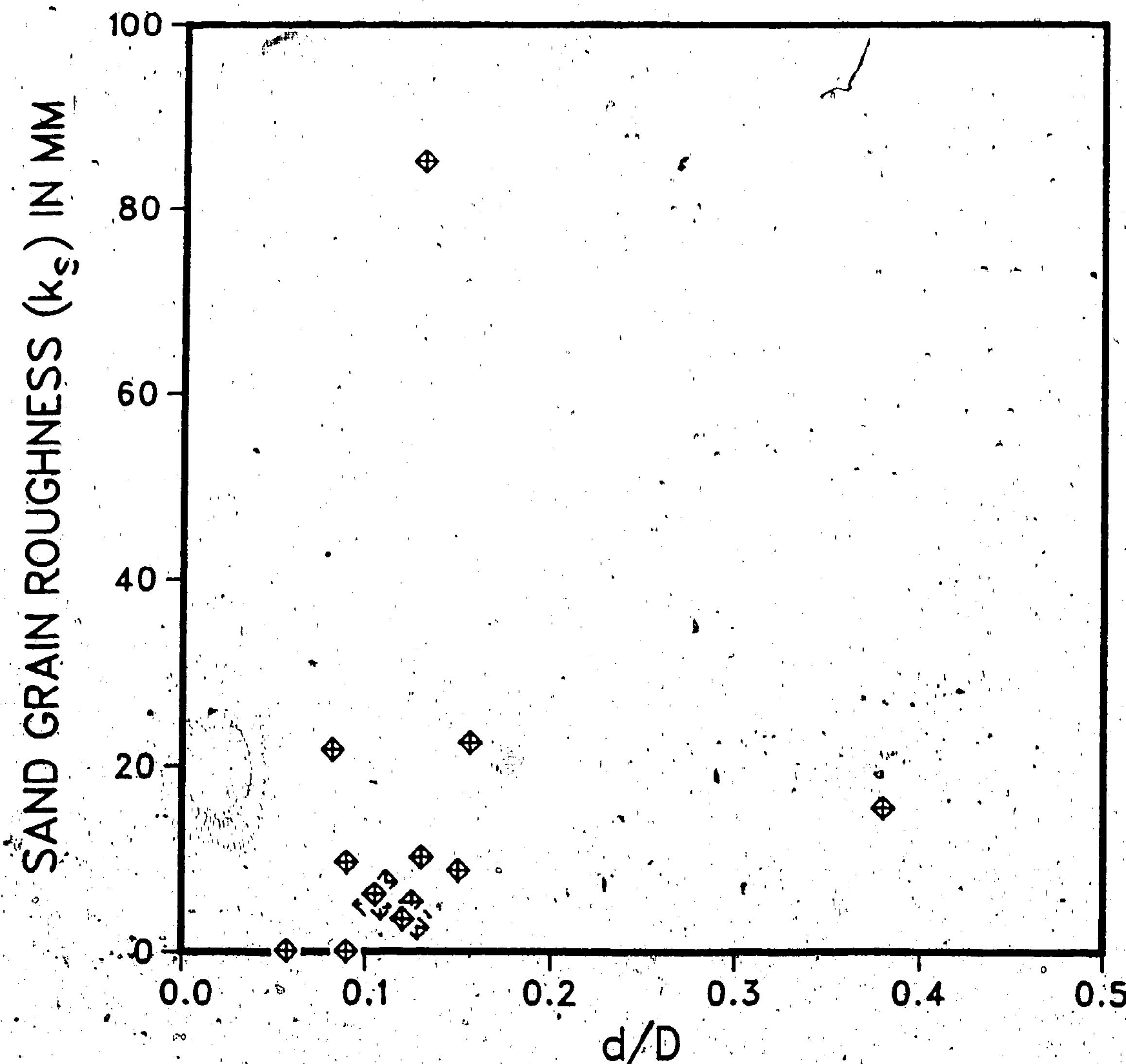

Figure 9. Variation of hydraulic roughness, k_s, with relative depth for normal flows in PVC sewers measured in the present study

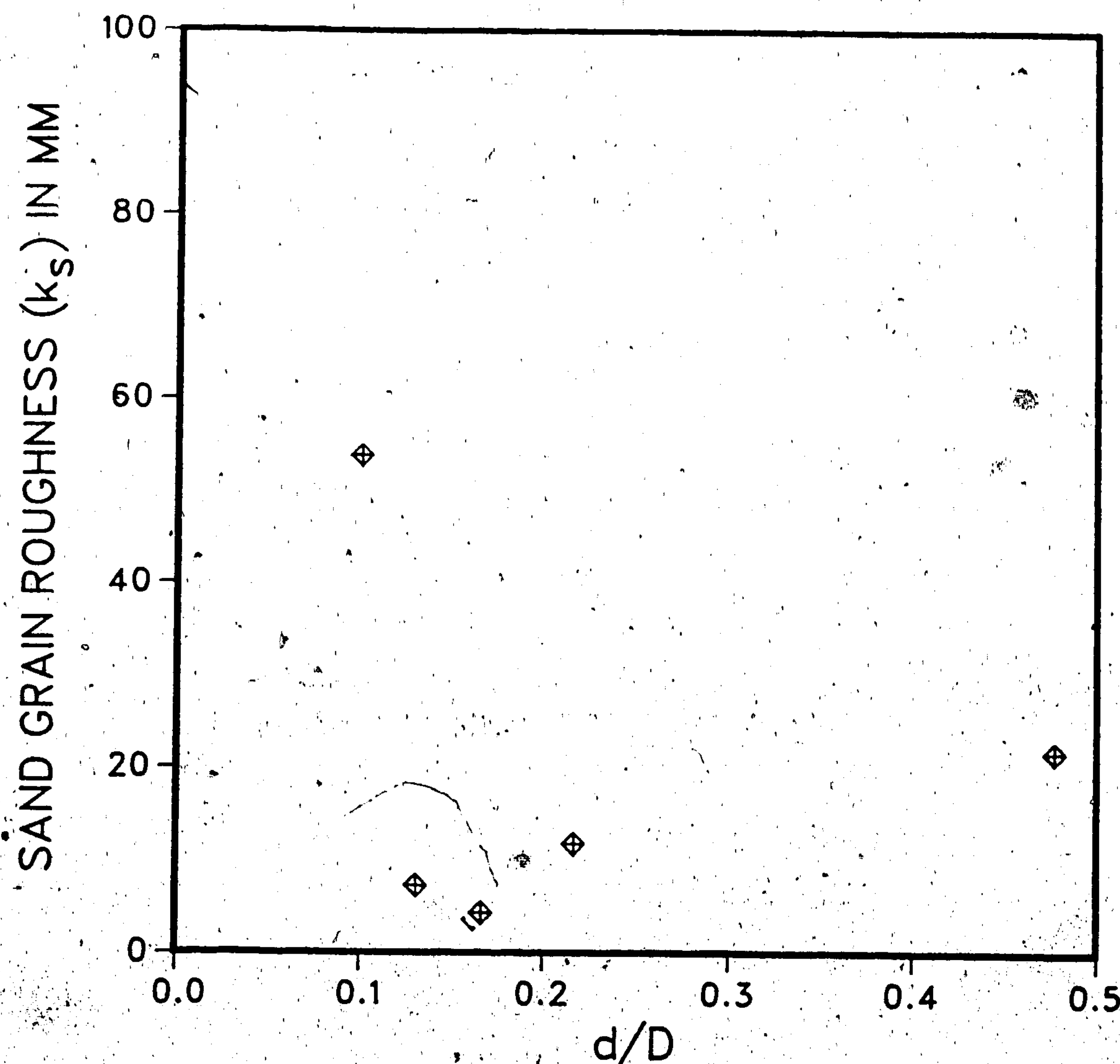

Figure 10. Variation of hydraulic roughness, k_s, with relative depth for normal flows in concrete sewers measured in the present study.

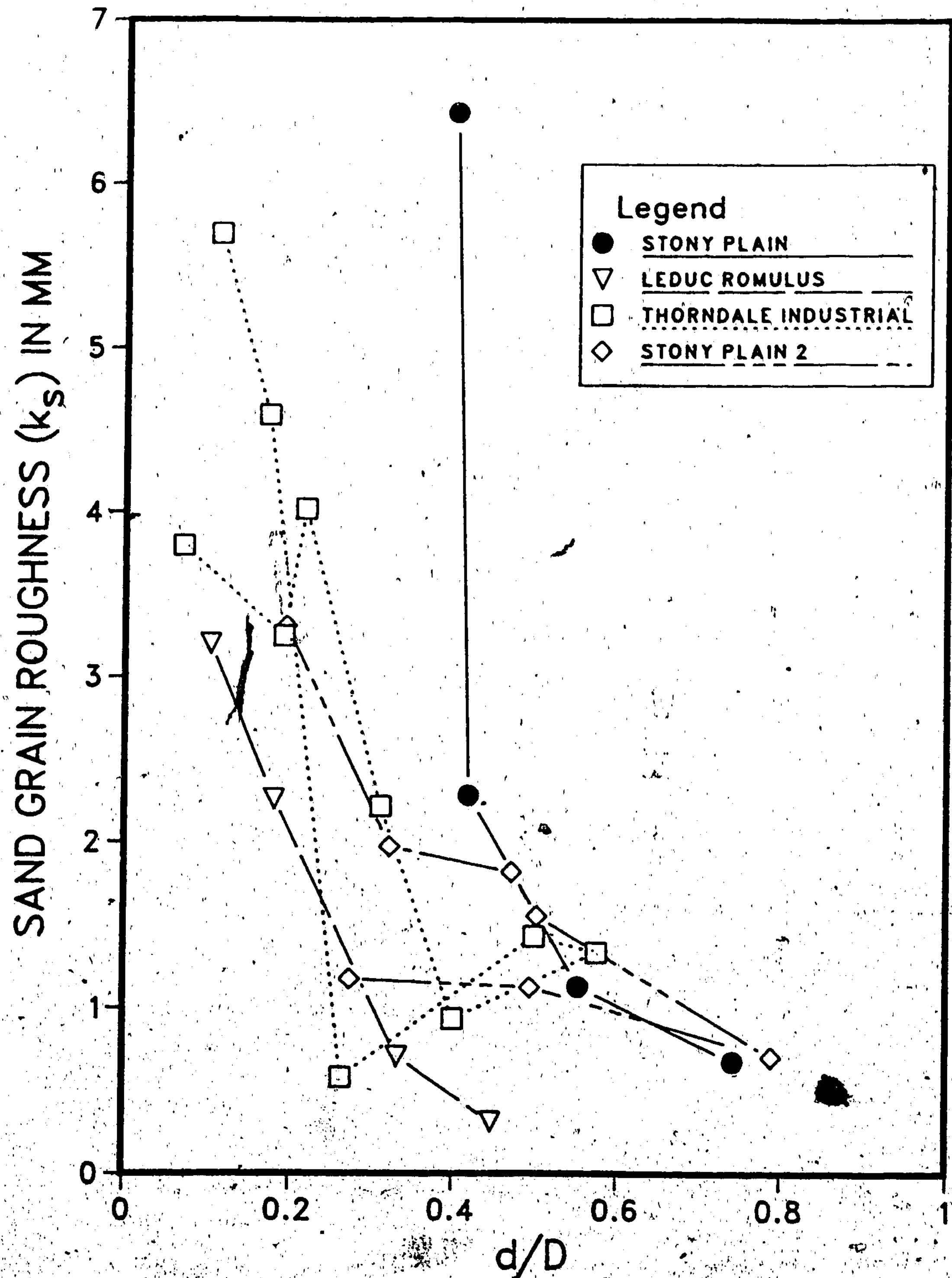

Figure 11. Composite plot of variation of hydraulic roughness, k_s, with relative depth for augmented-flows in PVC sewers measured in the present study

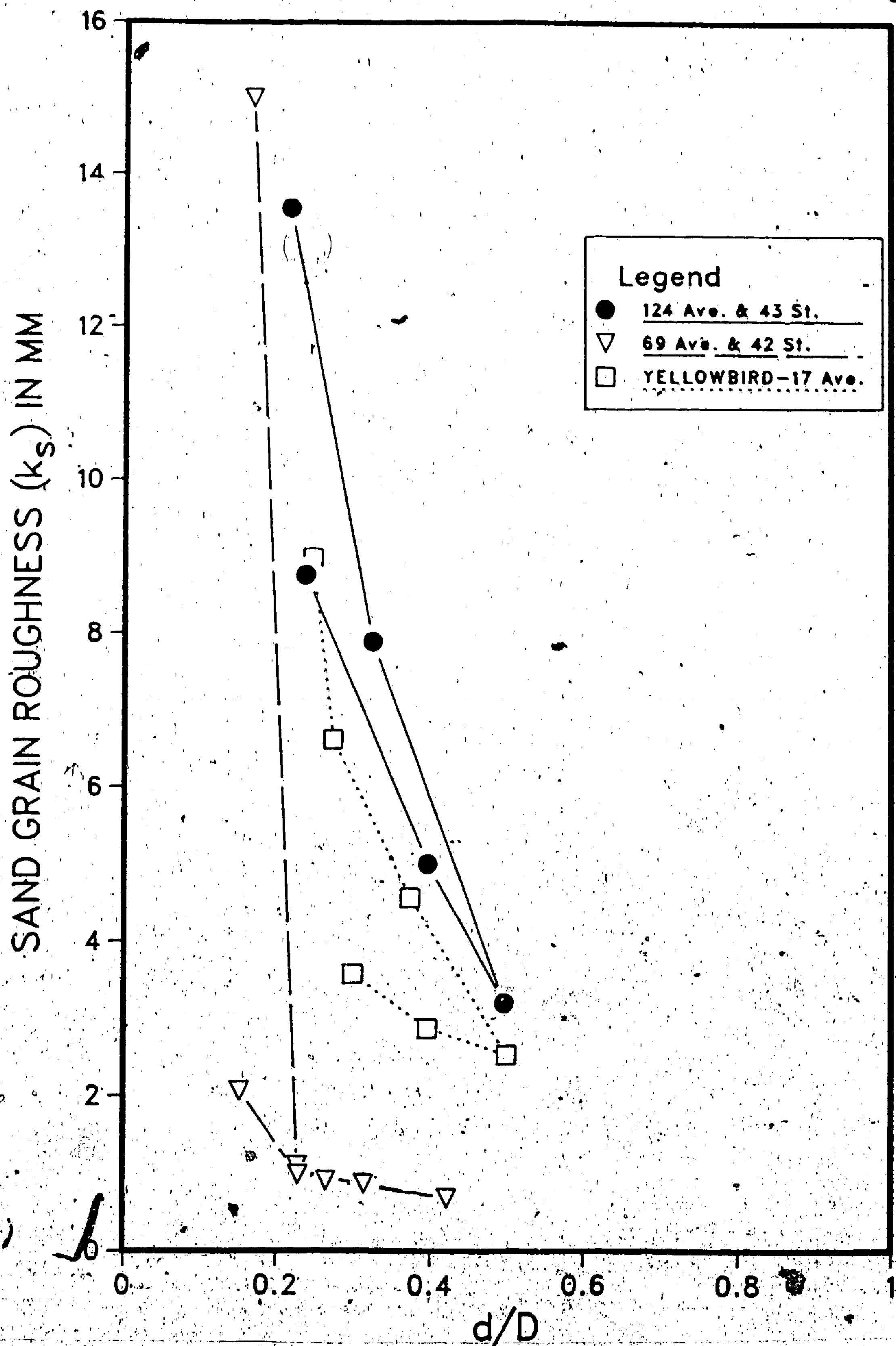

Figure 12. Composite plot of variation of hydraulic roughness, k_s, with relative depth for augmented-flows in concrete sewers measured in the present study

of several earlier test series (Yarnell and Woodward, 1920, Wilcox, 1924) that have since been accepted into the general literature on sewer design. To confirm the above conclusion simple analytical estimates were made of the variation in hydraulic roughness with relative depth. These estimates confirm the trend found by Bock and support the present conclusion. Details of these calculations are given in Appendix 2.)

Another item of note is the 'hysteresis' evident in the variation of k_s with d/D in the augmented-flow tests. The roughness was found to be much greater as the flows were being stepped down from the maximum than for the same relative depth as the flow was being stepped up to the maximum. The reason for this is not at all clear but it is likely related to the disturbance of the sediment and slime by the high flows. Some evidence for this is provided by the tests between MH 23-22 in Riverbend. The first test gave a roughness of 5 mm whereas the second test, carried out after the sewer was 'cleaned' by the City, gave a roughness of 10 mm.

Previous investigators have concluded that flow resistance is not governed so much by the age and pipe material as by such items as the sliming of the pipe wall, sediment on the invert, joint eccentricities and uniformity of the longitudinal profile of the sewer. Nevertheless, in as much as the age and pipe material can be expected to influence these latter parameters, they may have an indirect effect on flow resistance. For example, Pomeroy (1967) concluded from his extensive tests that in-service asbestos-cement sewers had a lower resistance than vitrified clay sewers and that concrete sewer lines had the poorest performance. Also, Perkins and Gardiner (1982) concluded that the pipe material has some influence on the amount of slime present and therefore on the in-service hydraulic roughness. They concluded from their tests that plastic sewers had the lowest in-service hydraulic roughness and that clay, asbestos-cement, and vertically cast concrete were rougher, with spun concrete having the poorest performance.

A similar difference between the hydraulic roughness of PVC and concrete lines for other than low flows is also evident in the present results. As a typical example, the hydraulic roughness of the sewers at a relative depth of 0.5 was compared. As shown in Figure 13, if the measured variations of k_s with d/D as the flow was being stepped up are extrapolated or interpolated to a fixed value of $d/D = 0.5$, the average k_s for PVC is about 0.8 mm while that for concrete is about 2.1 mm. (The normal relative flow depths in these sewers was about 0.1 to 0.2. This is presumably the depth to which significant slime layers extend if they exist.) It should be pointed out that the low roughness value for concrete in Figure 13 did not have 'hysteresis' evident in the variation of k_s with d/D, indicating that there may not have been any significant slime layers present for the higher flows to disturb. It should be noted that both these values are again much higher than those determined for clean water flows. Typical quoted values for this situation are 0.0015 mm for PVC and 0.3 mm for concrete (Uni-Bell Plastic Pipe Association, 1982).

Ackers et al. (1964), Henderson (1984) and Perkins and Gardiner (1982) indicate that at steeper slopes the slime build-up on the pipe wall is significantly less, presumably because the higher shear stress exerted on the boundary impedes the slime growth. As a result the flow resistance is less where they found limited slime growth. The boundary shear stress values where they found reduced roughnesses were greater

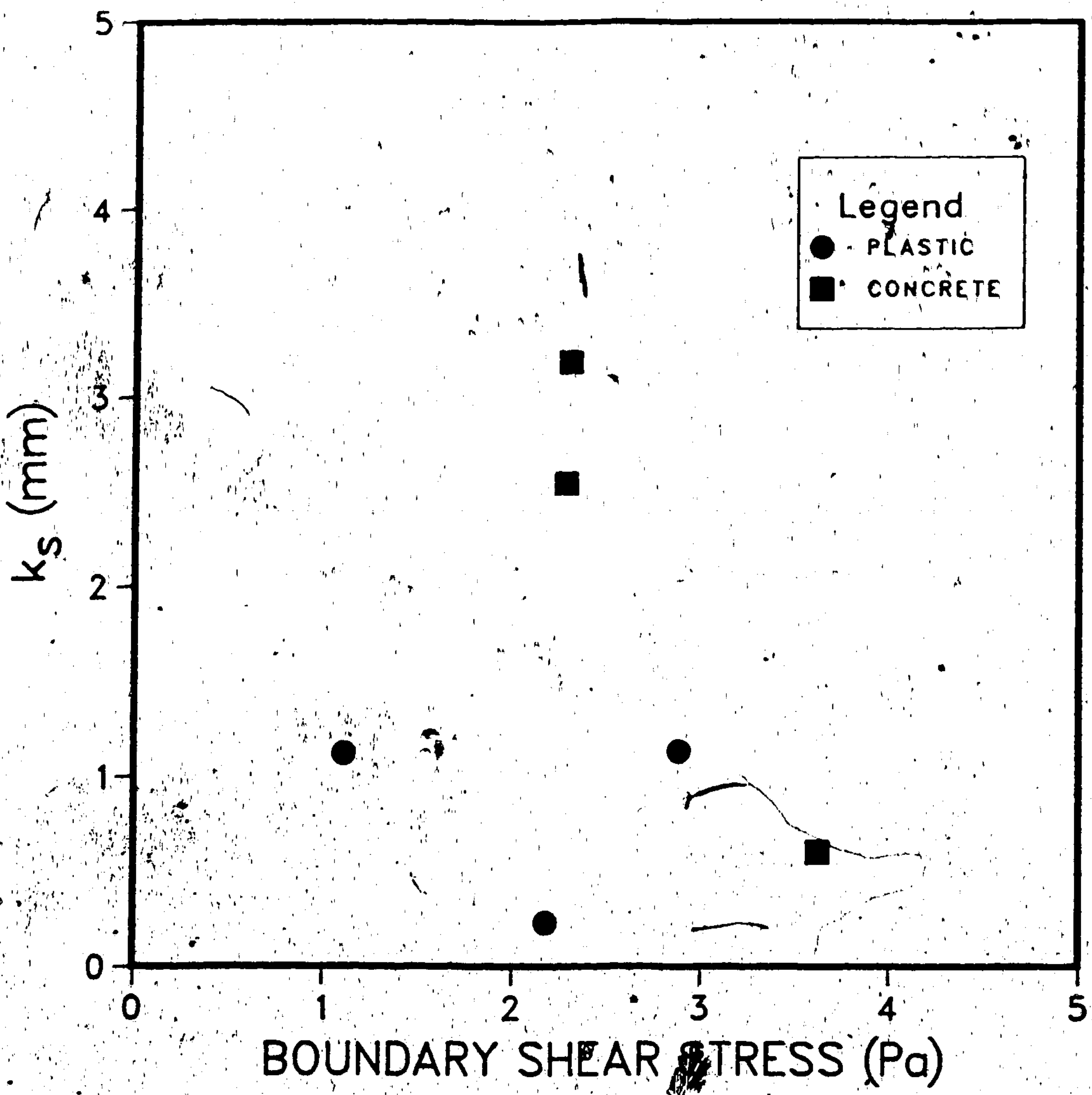

Figure 13. Variation of hydraulic roughness, k_s, with boundary shear stress for a fixed relative depth of 0.5 from measurements in the present study

than about 5 Pa. The normal flow tests on PVC pipe for the steep slope sites in Riverbend which, even at these low relative depths, sustain boundary shears of 3.4 and 1.0 Pa, are consistent with this, having measured roughnesses of only about 0.1 to 0.2 mm. It might also be noted that the flow at these sites had Froude numbers greater than one. Hence the lower roughness found is contrary to expectations based on clean water flows. Powell and Posey (1959) found that the flow resistance increased for Froude numbers greater than one, presumably because of the flow irregularities that can develop with such supercritical flows.

An irregular longitudinal profile can be responsible for a high apparent roughness. To isolate the contribution of this to the measured roughness it is necessary to know the in-service profile of the test sewers. A first effort to do this in the present tests utilized the airline apparatus described earlier to measure the variation in depth along the line under normal flow conditions but, as mentioned, this was felt to be unreliable. In the absence of measurments of the longitudinal profiles of the lines, an analysis was carried out to assess the sensitivity of the apparent roughness to variations in the profile of the sewer. A length of 120 m was used, and a parabolic deflection, either a sag or rise, was assumed to exist over 80 m of the central portion of the line, with the maximum deflection occurring at the center of the length. A gradually varied flow analysis was carried out, assuming uniform flow existed at the downstream end. From the calculated profile the apparent roughness was determined using simulations of the field measurements as described in Appendix 3. The results are shown in Figure 14. They confirm that the effective

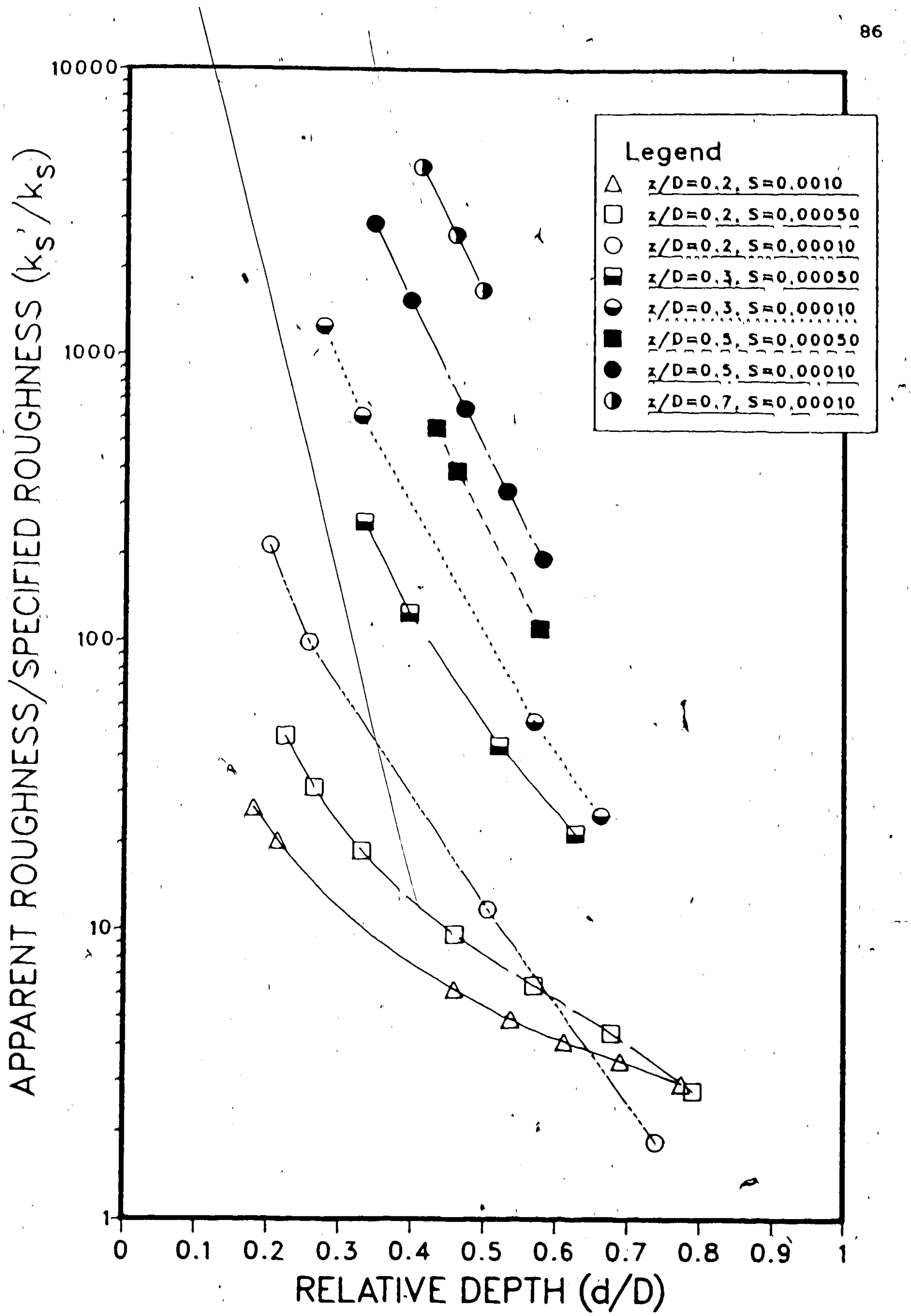

Figure 14. Calculated variation of ratio of apparent roughness to pipe boundary roughness, k_s'/k_s, with relative depth for lines with a sag.

hydraulic roughness is a strong function of the longitudinal profile, particularly for low flows, and could explain some of the very high hydraulic roughnesses measured. It is therefore desirable that future measurements of the invert profile be made using a slope indicator pulled through the sewer and that other low slope, high roughness lines be tested. There is little doubt that, for small sewers near minimum grade, settlement of the line after construction could cause sufficient variation in the sewer profile to significantly influence the effective hydraulic roughness.

For a sewer line with an irregular longitudinal profile, deposition of sediment and sewage slimes may actually improve the hydraulic characteristics at low flows. The sediments and sewage slimes would deposit at the low points and improve the longitudinal profile of the sewer. When the sediments and slimes are washed out the apparent roughness of the sewer line could then increase. This may explain some of the hysteresis evident in the curves for the augmented-flow tests.

There would seem to be three important considerations with regard to the resistance to flow in sewers. They are:

1. The resistance to flow for design flow conditions (full or near full). This is the item of primary interest when sizing the pipe.
2. The minimum grade for the pipe. Traditionally this has been governed by a specified minimum scouring velocity. In this circumstance the resistance to flow under low flow conditions is of interest.
3. The resistance to flow under all flow conditions to allow accurate analytical routing of flows through the system.

The primary aim of the present investigation was to determine the in-service characteristics of sewers with regard to item 1 and 2. The measurements to date have been for flows varying from low to moderate ($d/D < 0.6$) in both PVC and concrete sewers. These measurements have indicated that:

1. The effective roughness of the pipes is a strong function of the relative flow depth, being much higher for small depths and having its minimum value for full-pipe flow. The variation is consistent with what has been found from other, quite extensive, investigations elsewhere (Henderson (1984), Ackers et al. (1964), Perkins and Gardiner (1982)). The higher roughness at low flow is due to one or more of the following:

   (i) slime deposits over the portion of the pipe surface more or less continually in contact with the sewage.

   (ii) grade irregularities in sewers of low slope.

   (iii) sediment deposits in the invert.

   However, for steep slopes, and therefore higher boundary shear, there appears to be less slime build-up and the flow is much less sensitive to line imperfections. The measured roughness was then much closer to the clear water-new pipe values. It would be anticipated that at steep slopes the roughness of the sewer would not be such a strong function of relative depth due to there being less slime build-up on the pipe wall. Therefore, it appears that the hydraulic behaviour of a sewer will be considerably different at a flat slope than at a steep slope.

2. There is some indication from the measurements that there is a significant difference in the resistance to flow, for high flows,

between PVC ($k_s \approx 0.8$ mm) and concrete ($k_s \approx 2.1$ mm) sewers. This is remarkably consistent with the values of 0.6 mm and 1.8 mm respectively recommended for pipe-full flow by Perkins and Gardiner (1982) as discussed earlier.

3. For low flows the effective roughness can be very much higher than for $d/D \approx 0.5$ and varies substantially. It would appear from the field measurements and the theoretical analysis carried out that the greatest contributor to the high apparent roughness at low flows and flat slopes could be irregularities in the pipeline profile.

An important factor affecting the design of sewage systems is the minimum gradients at which pipes need to be laid. In areas of flat terrain the minimum gradients can have a major influence on the cost of the scheme because they tend to determine the depth of excavation and the amount of pumping needed to discharge the flow to a treatment works or outfall. Minimum gradients are normally set in an effort to make the pipe 'self-cleansing'. A self-cleansing velocity may be defined either as the velocity which prevents solids from depositing on the invert of the pipe, or as the velocity needed to remove any deposits that may have formed: the two definitions are not necessarily synonymous. The self-cleansing velocity for sewers given by Equation 1 is based on a minimum value of shear stress to move particles when the pipe is flowing full. However care is needed when applying the concept to the deposition of sediment in sewers which are designed to rarely flow full.

The present criteria for determining cleansing velocities in sewers presented in most texts in North America do not take into account that sanitary sewers rarely flow full, nor the significant increase in hydraulic roughness as the relative depth decreases, nor the influence of irregularities in the longitudinal profile. Nevertheless experience suggests that these criteria used are effective.

VI. CONCLUSIONS

Others have shown that the in-service hydraulic roughness of sanitary sewers is governed by sliming of the pipe wall, sediment deposition, and workmanship (e.g. joint displacement, post-construction settlement) rather than by pipe material. Nevertheless, the degree of sliming seems to depend on the pipe material, and hence, there is an indirect relation between in-service hydraulic roughness and pipe material. For example, previous measurements have suggested a significant difference exists between average in-service hydraulic roughness of PVC and concrete sewers, being about 0.6 mm for PVC and 1.8 mm for concrete. These values are for pipe-full flow and apply to pipes with velocities of about 0.75 m/s, carrying sewage and slimed to approximately half-depth. The results of field tests on such pipes in Edmonton are compatible with these findings, yielding average values for d/D = 0.5 of about 0.8 mm for PVC and 2.1 mm for concrete sewers with normal values of d/D ≅ 0.1. These roughnesses are very much higher than those recommended from clear-water new-pipe tests. Typical quoted values for this situation are 0.0015 mm for PVC and 0.3 mm for concrete (Uni-Bell Plastic Pipe Association, 1982).

There is a strong variation in the effective roughness with d/D. This is largely due to the non-uniform distribution of sewage slimes and sediment around the pipe perimeter. The variation in roughness with relative depth is much stronger than the usual variations presented in standard textbooks.

The apparent roughness at flat slopes and low depths was very high, being **orders of magnitude** higher than those usually assumed in the evaluation of "scour velocities". A major contributor to the high apparent roughness at low flows and flat slopes can be irregularities in the pipeline profile.

For steep slopes, and therefore higher boundary shear, there appears to be less slime build-up and the flow is much less sensitive to line imperfections. The measured roughness was then much closer to the clear-water new pipe values. It would be anticipated that at steep slopes the roughness of the sewer would not be such a strong function of the relative depth due to there being less slime build-up on the pipe wall. Therefore, it appears that the hydraulic behaviour of a sewer will be considerably different at a flat slope than for a steep slope.

VII. RECOMMENDATIONS FOR FURTHER WORK

To firmly establish the variations in the roughnesses of in-service sewers and the reasons for the differences, the following tests and measurements are recommended:

1. Two-cycle augmented-flow tests up to at least a relative depth of 0.6 on five or more lines of each type. The two cycles are required to assess the 'hysteresis' evident in the tests to date. The actual number of tests will depend on what is required to provide convincing evidence to the regulatory agencies and will likely require reassessment as the data becomes available.
2. Documentation of the slime deposits around the perimeter, other irregularities, and the in-service flow regime for each pipe tested.
3. For the sewers on a low grade, measurement of the longitudinal profile of the pipe.

The lines selected for these tests should include:

1. A range of normal flow depths up to at least 0.5D so as to vary the fraction of the perimeter covered with slime.
2. Small diameter pipes (200-300 mm) at grades well above the minimum to assess the effect of velocity on slime growth and its hydraulic characteristics, as well as the effect of velocity and/or slope on the other sources of apparent roughness.

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APPENDIX I

UNIFORM FLOW FORMULAS

From the summation of forces acting on an element of fluid for steady flow within a conduit of uniform cross-section, it can be shown that

$$\tau_0 = \gamma R S_f \tag{13}$$

where τ_0 is the shear stress at the pipe wall, γ the specific weight of the fluid, R the hydraulic radius and S_f is the slope of the energy line. The hydraulic radius is given by

$$R = A/P \tag{14}$$

where A is the cross-sectional area of the flowing fluid and P the wetted perimeter of the conduit. For open channel flow the shear stress on the channel boundary is not uniform and τ_0 is then the average.

From dimensional analysis τ_0 can also be written as

$$\tau_0 = c_f \, \rho \frac{V^2}{2} \tag{15}$$

where ρ is the density of the fluid flowing with mean velocity V, and c_f is a skin friction coefficient which is a function of Reynolds number $\mathbf{R}$ and a dimensionless length scale, k_s/D representing the boundary roughness. Reynolds number is given by

$$\mathbf{R} = \frac{4RV}{\nu} \tag{16}$$

where ν is the kinematic viscosity of the fluid.

Substituting equation 15 into 13 gives

$$S_f = c_f \frac{V^2}{2 g R} \tag{17}$$

For a pipe flowing full $R = D/4$, where D is the pipe diameter, and $S_f = h_L/L$ where h_L is the energy head lost over length L of pipe. Substituting into equation 17 and putting $f = 4c_f$ gives

$$h_L = \frac{fL}{D} \frac{V^2}{2g} \tag{18}$$

where f is the Darcy-Weisbach friction factor.

From the above development, considering only fully turbulent flow, it can be seen that resistance to flow depends upon two factors:

1. Fluid viscous forces expressed in terms of Reynolds number, and
2. The relative boundary roughness which may be expressed in terms of the ratio between the Nikuradse equivalent-sand-grain roughness k_s (ie. the roughness height projections of the pipe wall expressed in terms of the height of equivalent sand grains uniformly distributed on the pipe wall) and the diameter D for pipes flowing full, or its equivalent, 4R, for channel flow.

Experimental work by Nikuradse in the 1930's showed that the relative importance of these two factors depends upon the size of the viscous sublayer on the pipe wall in comparison to the height of the pipe wall roughness projections expressed by k_s. This led Nikuradse to develop two resistance laws, one for smooth pipes where the viscous sublayer deeply submerges the roughness height projections, and one for rough pipes where the roughness height projections protrude through and totally disrupt the viscous sublayer.

For smooth pipes the friction factor is primarily a function of skin friction (i.e. Reynolds number), whereas for rough pipes the friction factor is independent of Reynolds number and only a function of the ratio, k_s/D, as energy losses in fully rough flow are predominantly a result of drag on the roughness elements.

However, there is an intermediate range of flow which cannot be classified as wholly smooth or rough. In this case the roughness height projections are similar to the height of the viscous sublayer on the pipe wall. Therefore resistance to flow is a function of both skin friction and drag on the roughness elements. Colebrook and White developed an empirical expression for the friction factor for flow in this transition region for commercial pipes which also applies for the smooth and fully rough flow. It is

$$\frac{1}{\sqrt{f}} = -0.86 \ln \left(\frac{k_s/D}{3.7} + \frac{2.51}{R\sqrt{f}}\right) \qquad (19)$$

The Manning equation has been widely used for channel flow. It is

$$V = \frac{1}{n} R^{2/3} S^{1/2} \text{ (for SI units)} \qquad (20)$$

where n is an empirical roughness coefficient referred to as the 'Manning n'.

Equation 20 may be manipulated to reveal the relationship between f and Manning n. It is

$$f = \frac{8g\,n^2}{R^{1/3}} \tag{21}$$

Like f, it is only for fully rough flows that n is constant for a given relative roughness. For transitional flow conditions the Manning n will vary slightly with Reynolds number. Unfortunately, sewer pipe flow generally falls within this transitional category.

From f, k_s can be calculated using equation 19, and n using equation 21.

APPENDIX II

VARIATION OF APPARENT ROUGHNESS WITH RELATIVE DEPTH

Many investigators believe that it is not completely correct to use pipe flow relations to determine the resistance of non-circular conduits. Implicit in replacing the diameter D by 4R in pipe friction formulae is the assumption that the departures from uniform distribution of boundary shear stress have a minor influence on the resistance to flow. If this is not so the treatment of the conduit as an equivalent pipe with diameter equal to 4R does not fully account for the cross-sectional shape effect. This shortcoming of the hydraulic radius in representing waterway shape is well recognized (see, for example Shih and Grigg, 1967). The argument has been about the error due to this assumption. Some suggest that although there is an effect of channel shape on flow characteristics, it is negligible for regular shapes such as rectangular channels with aspect ratios (width/depth), as low as 2.

Recently Kazemipour and Apelt (1979) developed a practical method which takes account of the effects of the cross-section shape. On the basis of dimensional analysis they introduced two parameters to represent shape effect: the ratio of wetted perimeter to surface width, P/B and the ratio of surface width to average depth, B/y_{av}.

It is simple to show (Gerard, 1985) that the Colebrook-White equation can be rearranged to

$$\frac{V}{V_*} = 2.5 \ln \frac{R}{k} + 6.6 \qquad (22)$$

where V_* is the average boundary shear velocity given by

$$V_* = \sqrt{gRS_f} \tag{23}$$

where g is the gravitational constant, R the hydraulic radius and S_f the slope of the energy line. The boundary roughness parameter k is given by

$$k = k_s + k_\nu \tag{24}$$

where k_s is the Nikuradse equivalent sand grain roughness and k_ν can be envisaged as a roughness height that would have an equivalent flow resistance as the viscous shear at the boundary. It is given by

$$k_\nu = 3.3 \frac{\nu}{V_*} \tag{25}$$

where ν is the kinematic viscosity. It will be noted that

$$\frac{V}{V_*} = \frac{C}{\sqrt{g}} \tag{26}$$

where C is the traditional Chezy coefficient. That is, V/V_* is simply the non-dimensional Chezy coefficient. To distinguish between the two, C_* will be used for the latter. That is

$$C_* = \frac{V}{V_*} \tag{27}$$

As this is also a dimensionless average velocity an appropriate name would be conveyance coefficient.

Hence the discharge in a conduit is simply given by

$$Q = C_* A\sqrt{gRS_f} \quad (28)$$

where $C_* = 2.5 \ln \frac{R}{k} + 6.6$.

Knowing R and k, C_* is simply calculated. This will be the form of the Colebrook-White equation used herein since it is easy to analyze for shape effects.

As pointed out by Keulegan (1938) the effect of shape is primarily reflected in the constant 6.6. For example, for a wide channel it becomes 6.0. Hence this constant can be replaced by a shape parameter B which varies with cross-sectional shape. For example

$$\frac{V}{V_*} = 2.5 \ln \frac{R}{k} + B \quad (29)$$

The parameter B varies from 6.6 for pipe flow to 6.0 for flow in a wide channel with a commonly accepted value of 6.2 for channel flow. It would be expected that B is a function of the variables d/D, k_s/D and $V_* k_s/\nu$ or, considering the viscous effects to be included in k

$$B = f(d/D, k/D) \quad (30)$$

A simple analysis was carried out to estimate how the shape parameter varies in a pipe flowing part-full using a simple technique similar to that of Keulegan (1938) for other non-circular conduit shapes. The flow area in the pipe was divided into a number of small elements as shown in Figure 15. Coupling between the elements of fluid was not taken into account. This implies that the isovels are concentric circles so that there is no shear between the elements, and

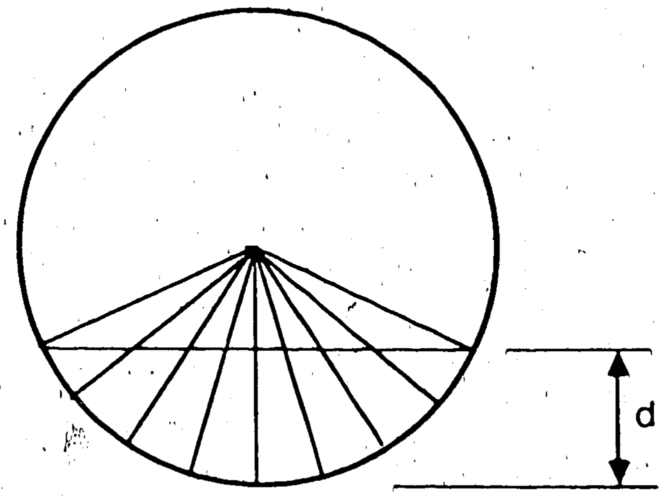

Figure 15. Flow elements in a part-full pipe.

that there are no secondary flows, neither of which is completely true. However, from experience with other sections (Gerard, 1974), it was felt to be an adequate approximation for the present purposes. Hence within each pie-shaped element it could be assumed that Equation 29 applied with B = 6.2. From the estimated velocities in the elements the average velocity over the waterway was calculated. Equation 29 was then solved for the shape parameter B. The variation of B with relative depth found for various values of d/k is shown in Figure 16. Calculations were only carried to half-full as with this simple algorithm the value of B must equal 6.2 at this depth and can vary little from that at higher depths.

It can be seen that the shape parameter should be a strong function of d/D at low flows, and varies slightly with d/k.

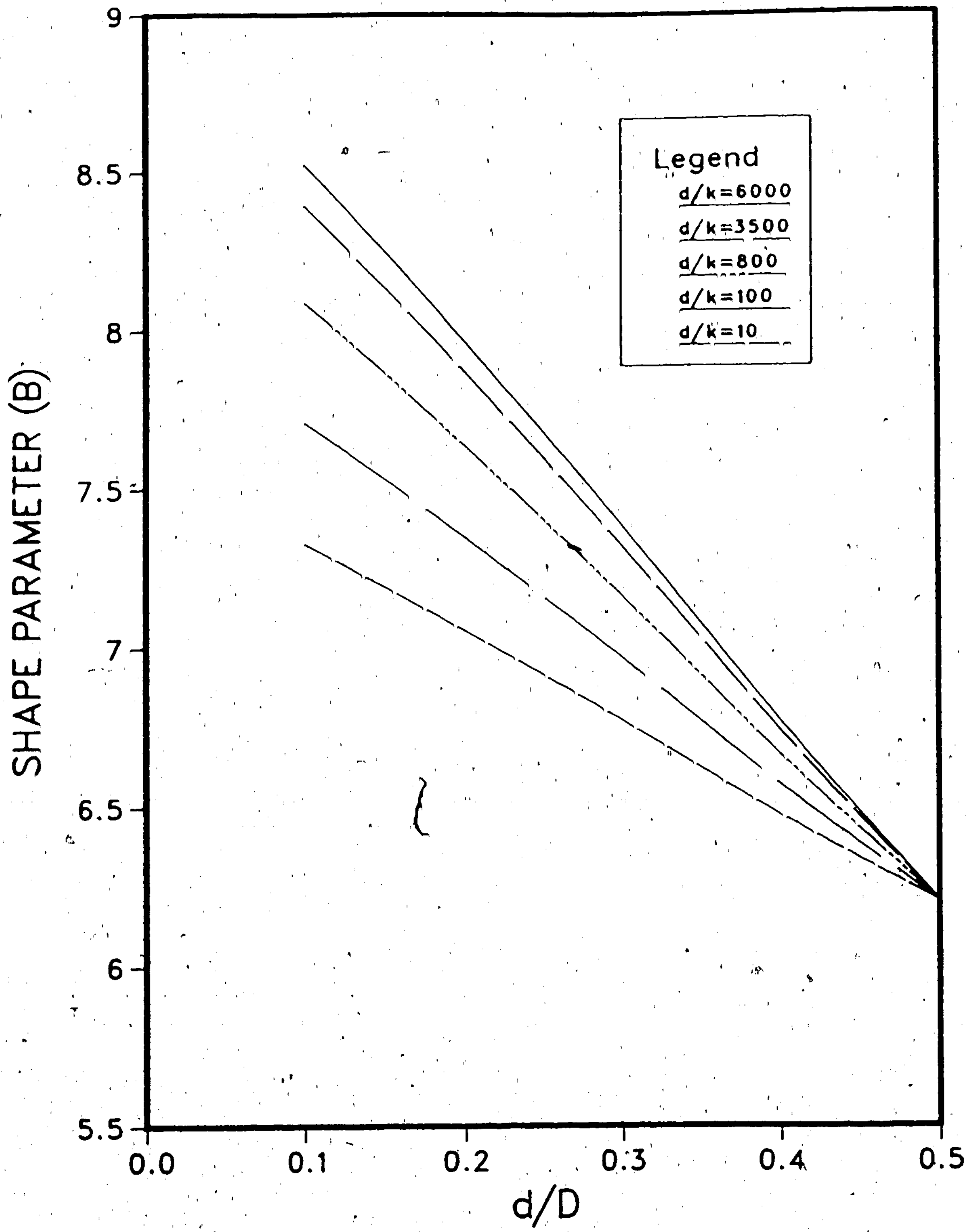

Figure 16. Calculated variation of shape parameter (B) with relative depth and d/k in a part-full pipe

The shape factor can be incorporated into an apparent hydraulic roughness, k_{app} where

$$\frac{V}{V_*} = 2.5 \ln \frac{R}{k_{app}} + 6.2 \qquad (31)$$

From this and equation 29 it is evident that

$$\frac{k_{app}}{k} = \frac{12}{e^{B/2.5}} \qquad (32)$$

The calculated shape effect, expressed in this form, is shown in Figure 17. It is evident that apparent roughness due to the shape effect should increase substantially at low relative depths. Also shown in Figure 17 is the variation in the shape effect determined experimentally by Bock (1966) for part-full flow in a smooth pipe. Considering the approximations involved in the simple algorithm used for the calculations herein, the agreement is remarkably good. The sense of the disparity is consistent with the observation that secondary flows act to reduce variations in boundary shear and hence shape effect (Gerard, 1974).

The shape effect can also be expressed in terms of Manning n. From the definition of n and the conveyance coefficient it is evident

$$n = \frac{R^{1/6}}{C_* \sqrt{g}} \qquad (33)$$

Hence

$$\frac{n_{app}}{n} = \frac{C_*}{C_{*\,app}} = \frac{2.5 \ln R/k + B}{2.5 \ln \frac{R}{k} + 6.2} \qquad (34)$$

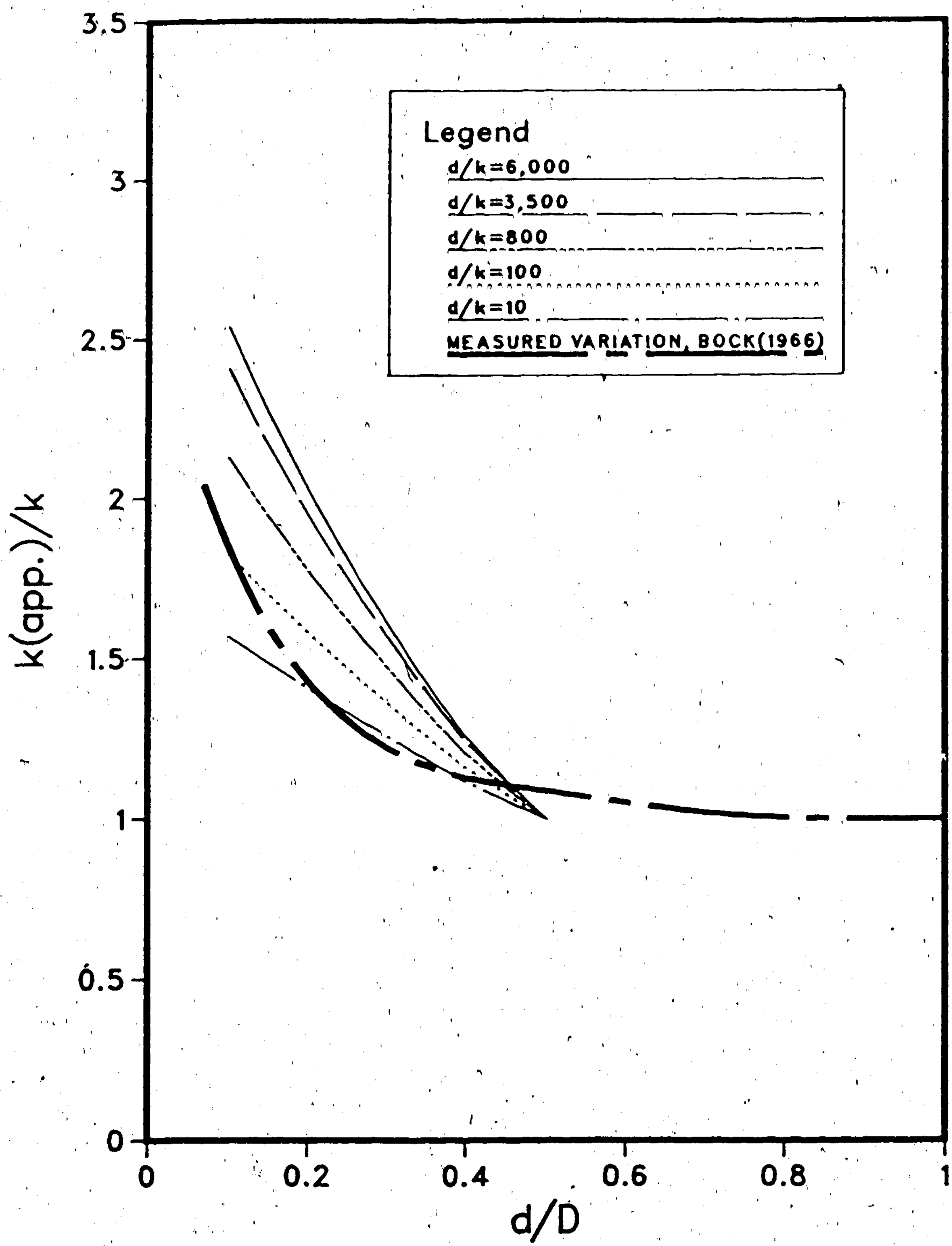

Figure 17. Calculated variation of ratio of apparent roughnesss to boundary roughess, k_{app}/k, with relative depth.

This variation, deduced from Bock's experimental results is compared in Figure 2 with the variation given in most textbooks on sewer design, which seems to be based on the early measurements of Yarnell and Woodward (1917). The two trends are obviously unrelated. The trend found by Bock is supported by the above analysis.

As mentioned above, the theoretical analysis indicates that the shape effect decreases with an increase in relative roughness. Its significance would be further reduced by the effects of variation in roughness around the boundary and other much larger influences on the apparent roughness, such as the variations in pipe profile discussed in Appendix 3, and sediment deposition in the invert. Hence no specific allowance for shape effect was made in analysing the field data.

APPENDIX III

GRADUALLY VARIED FLOW ANALYSIS FOR A PIPE WITH NON-UNIFORM SLOPE

As mentioned in the text, it was suspected that some of the high roughnesses measured were due to variations in pipeline profile. Attempts were made to determine the in-situ profile of a sewer line using the airline apparatus described in Appendix 4, but these had limited success. In the absence of measurements of the actual longitudinal profiles of the lines, an analysis was carried out to at least assess the sensitivity of the calculated apparent roughness to variations in the sewer profile from the straight line assumed.

A length between manholes of 120 m was assumed, with a parabolic deflection, either a sag or rise, over 80 m of the central portion of the line as shown in Figure 18. A gradually varied flow analysis was then carried out to define the actual variation in depth along the pipe for various discharges and profiles, assuming uniform flow existed at the downstream end. The direct step method was used for the gradually varied flow analysis, with step lengths of 10 m. For the purposes of the analysis the pipe boundary roughness was taken to be $k_s = 0.083$ mm (Manning n = 0.010). Only subcritical flows were considered.

From the results of the gradually varied flow analysis simulated salt-velocity measurements could be determined and the apparent pipe roughness calculated using the same procedure used to calculate the roughness from the field measurements. To simulate the salt-velocity measurement the average velocity over each segment was computed by

$$V_{av} = \frac{V_{(i)} + V_{(i-1)}}{2} \qquad (35)$$

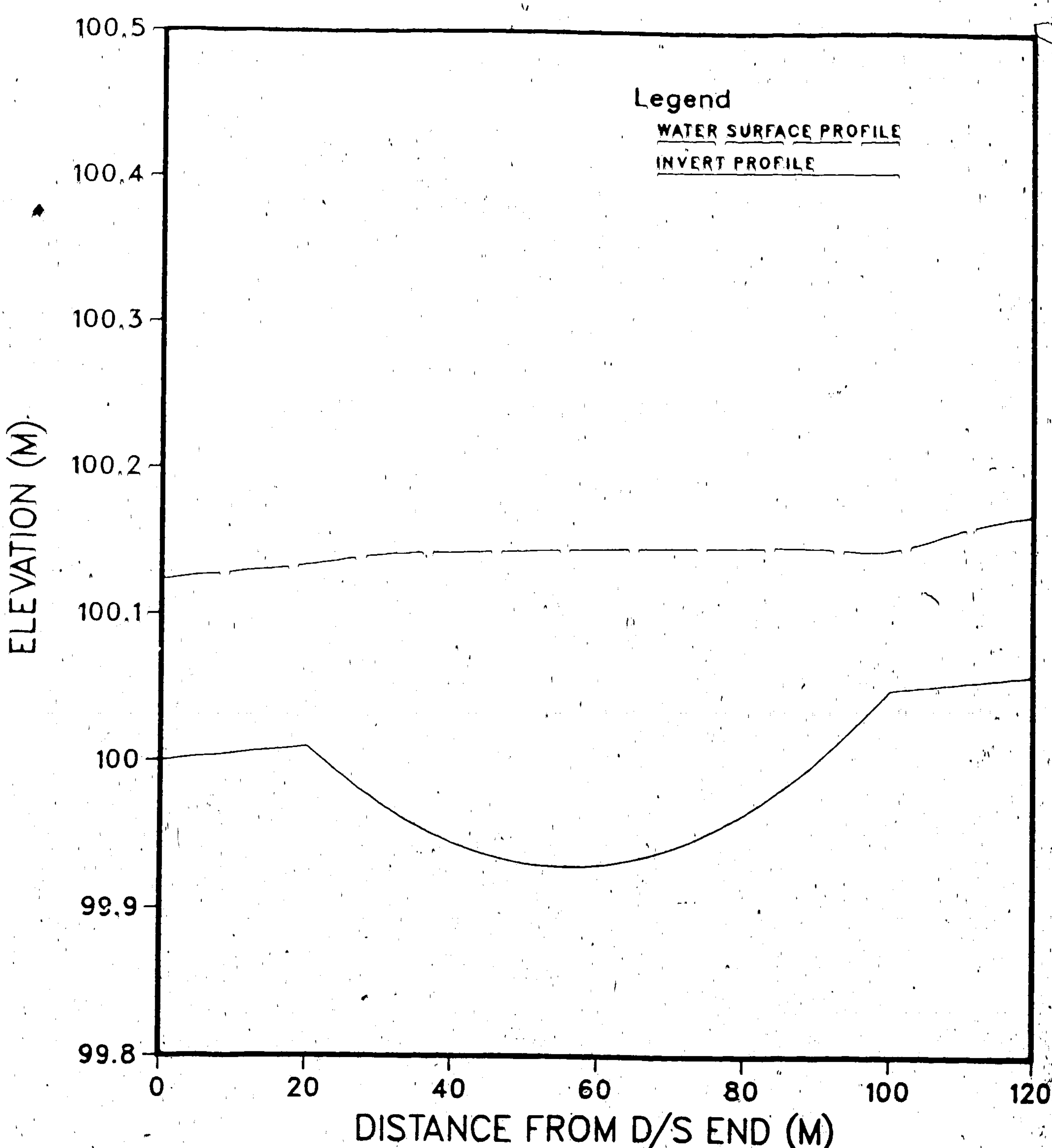

Figure 18. Calculated invert profile and water surface profile for a dimensionless sag (z/D) of 0.3 and slope of 0.00050

and the time for the simulated salt slug to travel between manholes was calculated from

$$t = \sum_{i=1}^{N} \frac{\Delta x}{V_{av}} \tag{36}$$

where Δx is the length of each step in the solution. As for the field measurements, the average flow area was computed by

$$A = \frac{Q}{V_{av}} \tag{37}$$

and the average flow depth was then determined from this area and the properties of a circle. The apparent roughness was then determined using the Colebrook-White equation. A comparison of this to the specified roughness then indicated how sensitive the field measurements are to an irregular longitudinal profile. Dimensional analysis indicates that these effects can be reasonably represented by

$$\frac{k_s'}{k_s} = f\left(\frac{z}{D}, \frac{d}{D}, S\right) \tag{38}$$

where k_s' is the apparent roughness, k_s the specified roughness, z/D the dimensionless sag or rise, and d/D the relative depth. The results are given in Table 15. The ratio of apparent roughness to specified roughness is plotted against the relative depth for various values of slope and of dimensionless rise in Figure 19. That for sag is given in the main body of the text, Figure 14. The ratio of apparent roughness to specified roughness is plotted against dimensionless sag or rise for a fixed relative depth of 0.5 in Figure 20. It is evident that even at

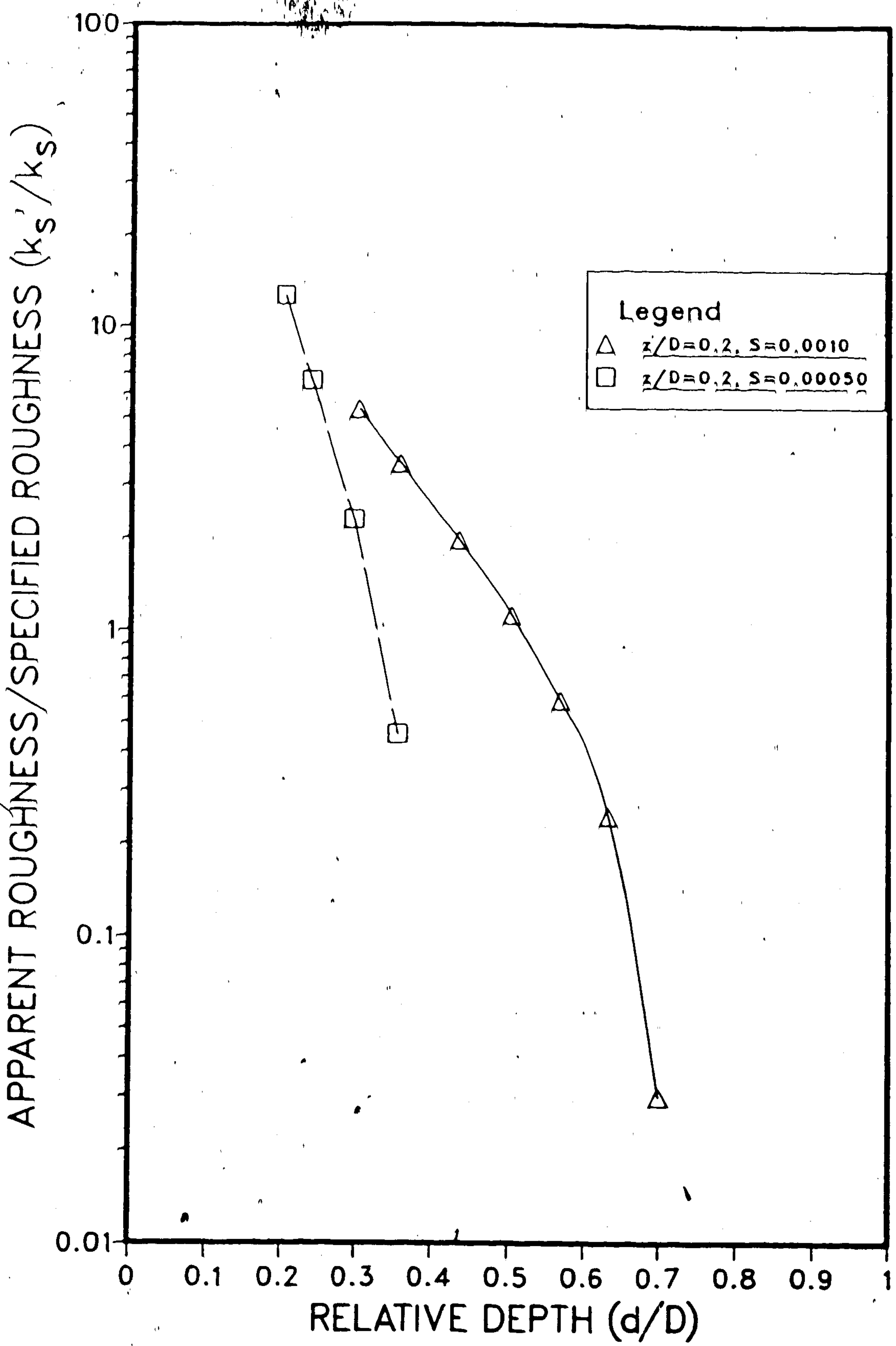

Figure 19. Calculated variation of ratio of apparent roughness to pipe boundary roughness, k_s'/k_s, with relative depth for lines with a hump.

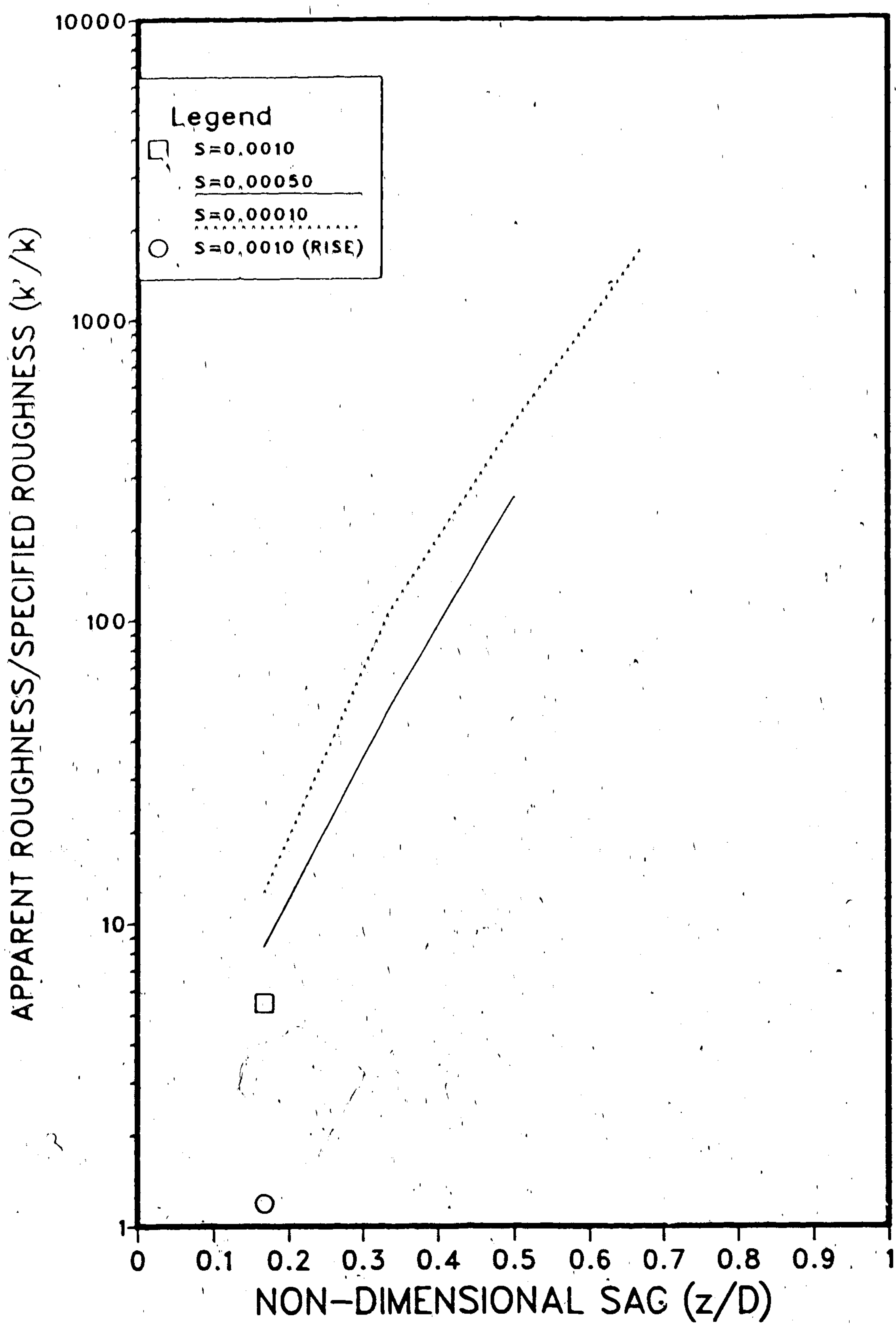

Figure 20. Calculated variation of ratio of apparent roughness to boundary roughness, k'/k, with non-dimensional sag (z/D) for a fixed relative depth of 0.5

Table 15. Calculated ratio of apparent roughness to pipe boundary roughness for a deflection in the pipeline profile. Pipe length 120 m; deflection over central 80 m.

n_{pipe} = 0.010 $k_{s\ pipe}$ = 0.0829 mm

Max. Defl. cm.	Sag or Rise	Q l/s	S	$\frac{d}{D}$	n'	k'_s (mm)	$\frac{n'}{n_{pipe}}$	$\frac{k'_s}{k_{s\ pipe}}$
5	sag	2.00	0.0010	0.18	0.0140	2.19	1.40	26.39
5	sag	3.00	0.0010	0.21	0.0133	1.68	1.33	20.26
5	sag	15.00	0.0010	0.46	0.0115	0.52	1.15	6.22
5	sag	20.00	0.0010	0.54	0.0112	0.41	1.12	4.90
5	sag	25.00	0.0010	0.16	0.0110	0.34	1.10	4.10
5	sag	30.00	0.0010	0.69	0.0109	0.29	1.09	3.51
5	sag	35.00	0.0010	0.77	0.0107	0.24	1.07	2.93
5	sag	2.00	0.00050	0.22	0.0155	3.89	1.55	46.93
5	sag	3.00	0.00050	0.26	0.0143	2.57	1.43	30.97
5	sag	5.00	0.00050	0.33	0.0132	1.55	1.32	18.65
5	sag	10.00	0.00050	0.46	0.0121	0.80	1.21	9.64
5	sag	15.00	0.00050	0.57	0.0116	0.53	1.16	6.40
5	sag	20.00	0.00050	0.68	0.0113	0.36	1.13	4.40
5	sag	25.00	0.00050	0.79	0.0109	0.23	1.09	2.77
5	sag	0.500	0.00010	0.20	0.0224	17.72	2.24	213.70
5	sag	1.00	0.00010	0.26	0.0181	8.18	1.81	98.63
5	sag	5.00	0.00010	0.51	0.0128	0.97	1.28	11.73
5	sag	10.00	0.00010	0.74	0.0113	0.15	1.13	1.84
10	sag	3.00	0.00050	0.33	0.0223	21.49	2.23	259.25
10	sag	5.00	0.00050	0.40	0.0185	10.36	1.85	124.96
10	sag	10.00	0.00050	0.52	0.0150	3.58	1.50	43.20
10	sag	15.00	0.00050	0.63	0.0135	1.77	1.35	21.43
10	sag	0.500	0.00010	0.28	0.0415	104.52	4.15	1,260.83
10	sag	1.00	0.00010	0.33	0.0292	50.20	2.92	605.58
10	sag	5.00	0.00010	0.57	0.0156	4.34	1.56	52.34
10	sag	7.00	0.00010	0.66	0.0140	2.05	1.40	24.74
15	sag	4.00	0.00050	0.43	0.0271	45.58	2.71	549.87
15	sag	5.00	0.00050	0.46	0.0243	32.30	2.43	389.61
15	sag	10.00	0.00050	0.58	0.0178	9.15	1.78	110.33
15	sag	0.500	0.00010	0.35	0.0644	239.27	6.44	2,886.22
15	sag	1.00	0.00010	0.40	0.0416	128.28	4.16	1,547.36
15	sag	2.00	0.00010	0.47	0.0283	53.24	2.83	642.23
15	sag	3.00	0.00010	0.53	0.0231	27.56	2.31	332.49
15	sag	4.00	0.00010	0.58	0.020	15.96	2.01	192.54

Table 15. Calculated ratio of apparent roughness to pipe boundary roughness for a deflection in the pipeline profile. Pipe length 120 m; deflection over central 80 m (continued)

Max. Defl. cm.	Sag or Rise	Q l/s	S	$\frac{d}{D}$	n'	k'_s (mm)	$\frac{n'}{n_{pipe}}$	$\frac{k'_s}{k_{s\ pipe}}$
20	sag	0.500	0.00010	0.41	0.0886	377.91	8.86	4,558.62
20	sag	1.00	0.00010	0.46	0.0540	218.96	5.40	2,641.24
20	sag	2.00	0.00010	0.50	0.0412	139.50	4.12	1,682.79
5	rise	7.00	0.0010	0.30	0.0112	0.44	1.12	5.37
5	rise	10.00	0.0010	0.36	0.0108	0.29	1.08	3.55
5	rise	15.00	0.0010	0.43	0.0104	0.16	1.04	1.99
5	rise	20.00	0.0010	0.50	0.0101	0.094	1.01	1.13
5	rise	25.00	0.0010	0.57	0.00981	0.049	0.98	0.59
5	rise	30.00	0.0010	0.63	0.00962	0.021	0.96	0.25
5	rise	35.00	0.0010	0.70	0.00949	0.0025	0.95	0.030
5	rise	2.00	0.00050	0.20	0.0126	1.05	1.26	12.69
5	rise	3.00	0.00050	0.24	0.0117	0.55	1.17	6.66
5	rise	5.00	0.00050	0.30	0.0107	0.19	1.07	2.33
5	rise	7.50	0.00050	0.35	0.0101	0.038	1.01	0.46

this large depth the error can be substantial particularly for low slopes.

The plots would have different values if another specified roughness had been used, but they would indicate the same trend: that the apparent roughness increases dramatically at low relative depths and flat slopes.

The steepest slope analysed was limited by the need for the flow to remain subcritical throughout for the algorithm used.

APPENDIX IV

AIRLINE DEPTH MEASUREMENTS

As mentioned in the text, it was anticipated that some of the high roughnesses measured may have been due to variations in pipeline profile. To measure the variation in depth along the sewer line that would exist with an irregular profile an airline apparatus was constructed.

The airline apparatus consisted of a probe, which was a metal tube 15.6 mm diameter and 1,016 mm long, with holes around the circumference at the centre of its length to emit air. As shown in Figure 21, the probe was connected to an air cylinder-regulator apparatus, and this in turn was connected to an open ended U-tube manometer. The pressure in the cylinder was adjusted to just allow air to bubble through the holes in the probe, this pressure being directly related to the water depth over the probe. The pressure was then measured on the manometer. The complete apparatus was calibrated by measurements in a flume. These measurements indicated a stable correction of 6 mm should be applied to the manometer reading.

In the field, the probe was pulled along the sewer invert from the downstream manhole. At regular intervals the manometer and the depth at the upstream manhole were read.

Two lines were tested with the airline apparatus: MH17-15 at Thorndale and MH23-22 at Riverbend. Initial flow resistance measurements between MH23-22 in Riverbend had given a roughness of 5 mm whereas the second test, carried out after the sewer was 'cleaned' by the City, gave a roughness of 10 mm. It was anticipated that a sag in the line might have caused these results. To investigate this

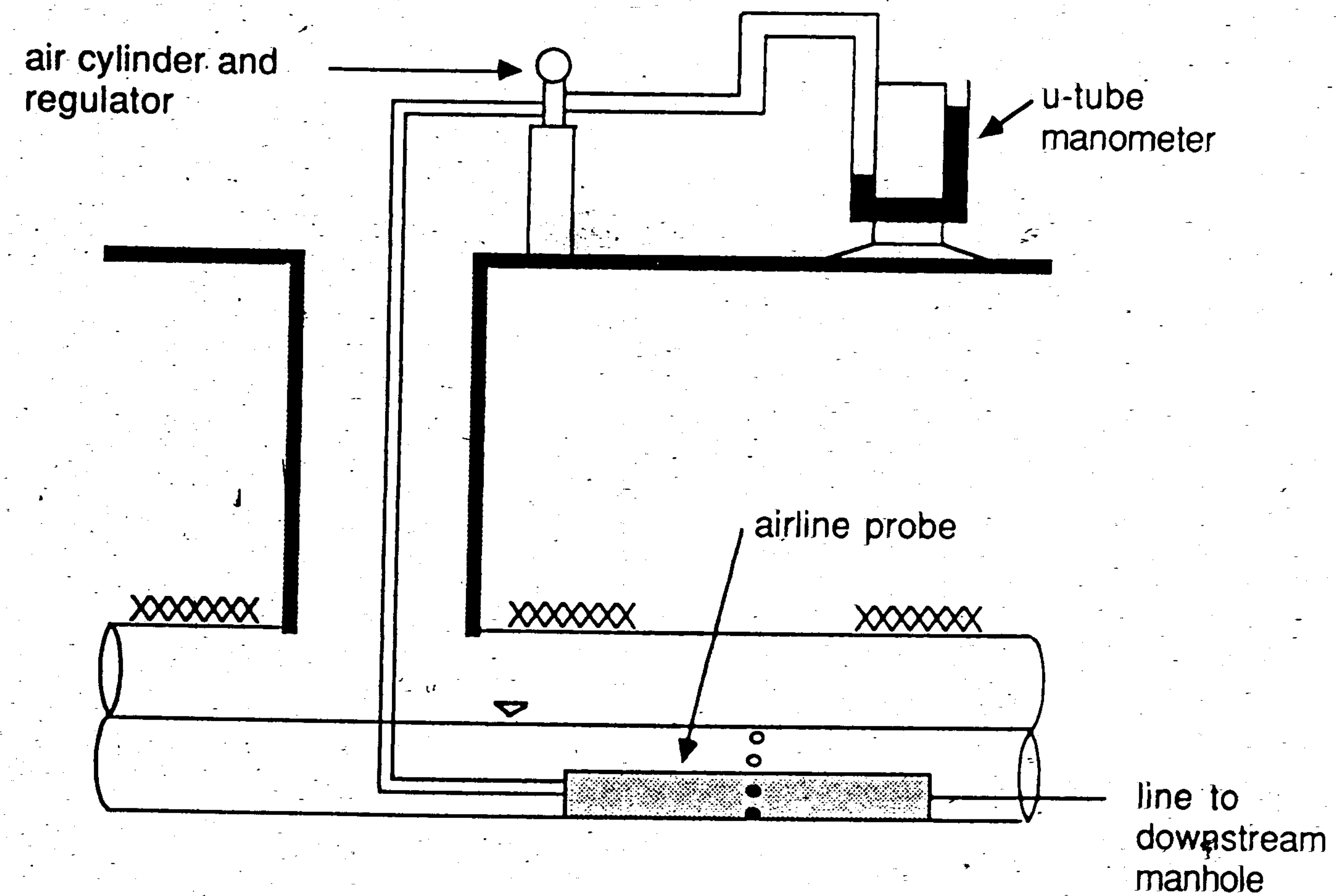

Figure 21. Airline depth measurement apparatus (NTS)

possibility airline depth measurements were taken. Airline depth measurements were taken between MH17-15 at Thorndale to establish if uniform flow was occurring before resistance measurements were taken. The results of these measurements show considerable scatter, as indicated in Figure 8, and no definite trend could be determined. The reason for this scatter is not clear. It may have been due to the unsteady nature of the flow, but attempts were made to reduce this problem by taking simultaneous measurements at the upstream manhole and the manometer. Because no clear trends could be discerned from the airline depth measurements, they were abandoned.

APPENDIX V

SENSITIVITY ANALYSIS

As mentioned in the text, a sensitivity analysis was carried out to determine what influence the measured parameters have on the hydraulic roughness calculated from the Colebrook-White equation. The analysis was carried out for typical extremes:

Parameter	Extremes	
Diameter	Large	Small
Relative depth	More than half full	Low
Roughness	High	Low
Slope	High	Low

At each extreme the variation in the hydraulic roughness estimate was determined for fixed percentage variations in the various measured parameters. The results are given in Table 16. From these results the variation in the error in hydraulic roughness around each extreme was expressed as a linear function of the errors in the measured parameters as given in Table 14 of the main body of the text.

The sensitivity analysis indicates that the roughness coefficient is most sensitive to the value of V measured, and least sensitive to the value of Q measured. For example, at a low flow depth, flat slope, and low roughness a 5% error in V gives a 310% error in the determination of k_s. On the other hand, at high roughness, flat slope, and low flow depth a 5% error in V gives a 30% error in the determination of k_s. The relative depth only has a significant influence on the error for flat slopes and low roughness. The effect of the diameter was also only significant in these circumstances, but was even less than that of the relative depth. A combination of high roughness and steep slope caused the least errors in k_s due to errors in the measured parameters.

Table 16. Sensitivity Analysis

D = 300 mm, Q = 1 ℓ/s, d/D = 0.11, S = 0.0010, n = 0.010, k_s = 0.066 mm

Parameter	% Variation in Parameter	% Variation in n	% Variation in k_s
Q	5	2.6	77.9
V	5	8.1	310.4
S	5	3.0	98.2

D = 300 mm, Q = 10 ℓ/s, d/D = 0.34, S = 0.0010, n = 0.010, k_s = 0.088 mm

Q	5	2.0	43.6
V	5	7.3	203.0
S	5	2.6	62.0

D = 300 mm, Q = 30 ℓ/s, d/D = 0.65, S = 0.0010, n = 0.010, k_s = 0.079 mm

Q	5	1.4	29.3
V	5	6.6	185.9
S	5	2.6	65.2

D = 300 mm, Q = 1 ℓ/s, d/D = 0.15, S = 0.0010, n = 0.020, k_s = 10.84 mm

Q	5	2.4	9.4
V	5	7.3	31.8
S	5	2.8	9.4

D = 300 mm, Q = 15 ℓ/s, d/D = 0.65, S = 0.0010, n = 0.020, k_s = 16.12 mm

Q	5	1.4	6.4
V	5	6.6	32.0
S	5	2.5	11.2

Table 16. Sensitivity Analysis (continued)

D = 300 mm, Q = 2 ℓ/s, d/D = 0.09, S = 0.010, n = 0.010, k_s = 0.23 mm

Parameter	% Variation in Parameter	% Variation in n	% Variation in k_s
Q	5	2.3	20.2
V	5	7.5	83.2
S	5	2.5	25.2

D = 300 mm, Q = 100 ℓ/s, d/D = 0.67, S = 0.010, n = 0.010, k_s = 0.17 mm

Q	5	1.3	12.5
V	5	6.3	83.9
S	5	2.5	29.8

D = 300 mm, Q = 2 ℓ/s, d/D = 0.12, S = 0.010, n = 0.020, k_s = 10.19 mm

Q	5	2.2	9.0
V	5	7.6	30.2
S	5	2.5	8.9

D = 300 mm, Q = 50 ℓ/s, d/D = 0.67, S = 0.010, n = 0.020, k_s = 16.30 mm

Q	5	1.3	6.0
V	5	6.3	30.4
S	5	2.5	11.2

D = 800 mm, Q = 10 ℓ/s, d/D = 0.09, S = 0.0010, n = 0.010, k_s = 0.088 mm

Q	5	2.2	53.9
V	5	7.7	222.4
S	5	2.5	66.6

Table 16. Sensitivity Analysis (continued)

D = 800 mm, Q = 400 ℓ/s, d/D = 0.64, S = 0.0010, n = 0.010, k_s = 0.039 mm

Parameter	% Variation in Parameter	% Variation in n	% Variation in k_s
Q	5	1.4	37.9
V	5	6.6	263.2
S	5	2.5	89.2

D = 800 mm, Q = 8 ℓ/s, d/D = 0.12, S = 0.0010, n = 0.020, k_s = 14.40 mm

Q	5	2.3	10.2
V	5	7.5	34.9
S	5	2.5	10.5

D = 800 mm, Q = 200 ℓ/s, d/D = 0.64, S = 0.0010, n = 0.020, k_s = 19.86 mm

Q	5	1.4	7.1
V	5	6.6	38.2
S	5	2.5	13.2

D = 800 mm, Q = 30 l/s, d/D = 0.09, S = 0.010, n = 0.010, k_s = 0.21 mm

Q	5	2.2	21.5
V	5	7.6	93.1
S	5	2.5	27.1

Table 16. Sensitivity Analysis (continued)

D = 800 mm, Q = 1,400 ℓ/s, d/D = 0.69, S = 0.010, n = 0.010, k_s = 0.091 mm

Parameter	% Variation in Parameter	% Variation in n	% Variation in k_s
Q	5	1.2	14.0
V	5	6.2	102.9
S	5	2.5	36.5

D = 800 mm, Q = 15 ℓ/s, d/D = 0.09, S = 0.010, n = 0.020, k_s = 13.40 mm

Q	5	2.2	9.5
V	5	7.6	34.0
S	5	2.5	10.0

D = 800 mm, Q = 700 ℓ/s, d/D = 0.69, S = 0.010, n = 0.020, k_s = 20.00 mm

Q	5	1.2	6.4
V	5	6.2	35.4
S	5	2.5	13.3

APPENDIX VI

RESULTS OF FIELD MEASUREMENTS

The raw data of the field tests is presented in the following tables, as follows:

(a) Column 1 - measured flow (L/s).

(b) Column 2 - measured velocity in (m/s).

(c) Column 3 - measured flow depth at one of the manholes (mm).*

(d) Column 4 - calculated average flow depth based on A = Q/V (mm).*

(e) Column 5 - ratio of depth to diameter.

(f) Column 6 - calculated hydraulic radius based on A = Q/V.

(g) Column 7 - Reynolds number = $\frac{4R\nu}{\nu}$.

(h) Column 8 - the friction factor calculated from the Darcy-Weisbach equation.

(i) Column 9 - equivalent sand grain roughness, k_s, computed from the Colebrook-White equation (mm).

(j) Column 10 - relative roughness for channel flow, where R = hydraulic radius.

(k) Column 11 - Manning n calculated from Manning's equation.

(l) Column 12 - Froude number.

The average and standard deviations are also given for all of the important variables for each field test.

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* In lines where there is sediment present the depth recorded is the actual flow depth about the sediment.

OCT 19, 1983 MH 22-22 RIVERBEND (PVC PLASTIC PIPE)

DIAM = 0.25520

LENGTH= 83.800 SLOPE=0.00340000

DEPTH OF SEDIMENTS 10.00MM

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
1.440	0.247	82.00	34.88	0.118	24.829	28471	0.109	10.888	0.1078	0.02010	0.477
1.020	0.269	40.00	24.83	0.082	18.802	20882	0.088	3.088	0.0414	0.01818	0.410
0.790	0.228	38.00	22.72	0.074	17.317	18482	0.084	4.887	0.0717	0.01704	0.538
1.810	0.272	83.00	38.01	0.117	28.083	28338	0.081	7.874	0.0785	0.01837	0.522
1.800	0.301	47.00	30.84	0.103	22.338	27811	0.088	3.438	0.0384	0.01838	0.418
1.220	0.290	43.00	28.47	0.088	18.888	23840	0.083	3.780	0.0348	0.01478	0.522
2.130	0.347	83.00	38.03	0.120	28.713	37082	0.087	2.808	0.0273	0.01484	0.887

AVERAGE	STANDARD DEVIATION
FLOW= 1.347L/S	0.438
VELOCITY=0.279M/S	0.0388
Y/D=0.100	0.0181
KS= 5.088MM	3.0419
MANNING N=0.01848	0.002084
FROUD NO =0.579	0.0884

NOV. 2, 1983 MH 26-26 RIVERBEND (PVC PLASTIC PIPE)

DIAM = 0.20120

LENGTH= 119.80000 SLOPE=0.03120000

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
1.340	1.010	0.0	17.31	0.086	11.074	48438	0.027	0.077	0.0017	0.00869	2.974
1.280	1.080	0.0	15.86	0.078	10.257	45137	0.022	0.007	0.0002	0.00787	3.251
1.340	1.090	0.0	16.44	0.082	10.538	47689	0.022	0.003	0.0001	0.00779	3.289
1.490	1.000	0.0	16.73	0.083	11.941	49575	0.029	0.144	0.0030	0.00923	2.828
1.900	1.020	0.0	21.81	0.108	13.797	58426	0.032	0.273	0.0049	0.00998	2.889
0.970	0.880	0.0	15.27	0.076	9.815	35859	0.031	0.140	0.0036	0.00920	2.782

AVERAGE	STANDARD DEVIATION
FLOW= 1.382L/S	0.307
VELOCITY=1.010M/S	0.0721
Y/D=0.087	0.0119
KS= 0.107MM	0.1018
MANNING N=0.00879	0.000848
FROUD NO =2.983	0.2801

NOV 16, 1983 MH 33-32 RIVERBEND (PVC PLASTIC PIPE)

DIAM = 0.29820

LENGTH= 83.80??? SLOPE=0.00340000

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
1.310	0.243	46.80	36.89	0.120	24.381	24607	0.119	10.793	0.1102	0.02016	0.474
1.420	0.245	47.00	36.84	0.123	23.190	27424	0.076	4.887	0.0538	0.01864	0.572
1.230	0.248	46.80	36.42	0.123	23.121	23606	0.100	6.892	0.0829	0.01908	0.488
0.830	0.222	42.09	30.38	0.101	19.272	17792	0.104	7.670	0.0898	0.01888	0.482
1.400	0.229	44.00	42.61	0.142	26.433	25130	0.134	16.316	0.1842	0.02288	0.427
0.880	0.222	41.00	34.01	0.114	21.482	19772	0.116	10.383	0.1206	0.02026	0.468
1.820	0.282	46.00	44.29	0.146	27.400	26666	0.115	13.088	0.1192	0.02102	0.461

AVERAGE	STANDARD DEVIATION
FLOW= 1.287L/S	0.273
VELOCITY=0.243M/S	0.0222
Y/D=0.126	0.0188
KS=10.247MM	3.7040
MANNING N=0.01981	0.001873
FROUD NO =0.484	0.0481

MAY 10, 1984 MH 19-17 THORNDALE INDUSTRIAL (PVC PLASTIC PIPE)

DIAM = 0.29820

LENGTH= 80.050 SLOPE=0.00275000

DEPTH OF SEDIMENT= 15.00MM

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO.	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
2.050	0.318	35.00	35.33	0.118	26.122	23774	0.086	2.718	0.0280	0.01481	0.593
3.100	0.380	38.00	45.53	0.152	32.252	32512	0.057	3.471	0.0269	0.01518	0.682
2.340	0.304	36.00	40.63	0.136	29.383	25701	0.068	4.947	0.0421	0.01641	0.534
2.020	0.283	38.00	37.14	0.124	27.234	22883	0.068	4.555	0.0416	0.01620	0.537
2.250	0.314	39.00	38.33	0.128	27.861	25288	0.061	3.590	0.0321	0.01539	0.587
1.980	0.302	37.00	35.20	0.118	26.037	22848	0.062	3.380	0.0325	0.01525	0.587

AVERAGE	STANDARD DEVIATION
FLOW= 2.285L/S	0.426
VELOCITY=0.313M/S	0.0199
Y/D=0.129	0.0131
KS= 3.778MM	0.8227
MANNING N=0.01557	0.000876
FROUD NO.=0.583	0.0237

MAY 10, 1984 MH. 17-18 THORNDALE INDUSTRIAL (PVC PLASTIC PIPE)

DIAM = 0.29820

LENGTH= 88.280 SLOPE=0.00393000

DEPTH OF SEDIMENT= 15.00MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
2.120	0.241	26.00	24.12	0.114	25.288	24843	0.087	4.093	0.0402	0.01587	0.650
1.840	0.222	27.00	21.83	0.106	23.024	21894.	0.071	4.276	0.0487	0.01616	0.634
1.920	0.226	28.00	22.81	0.109	24.417	22724	0.070	4.306	0.0441	0.01608	0.639
1.990	0.222	27.00	22.95	0.112	25.282	23080	0.0/5	5.227	0.0517	0.01675	0.615
1.660	0.202	26.00	20.75	0.102	22.220	18871.	0.076	5.186	0.0559	0.01664	0.607
1.710	0.207	26.00	21.17	0.104	22.801	20471.	0.077	5.087	0.0542	0.01675	0.611

AVERAGE	STANDARD DEVIATION
FLOW= 1.873L/S	0.174
VELOCITY=0.220M/S	0.0139
Y/D=0.108	0.0047
KS= 4.719MM	0.5119
MANNING N=0.01641	0.000421
FROUD NO.=0.626	0.0176

MAY 14, 1984 MH. 19-18 THORNDALE INDUS. (2 PVC PLAS. PIPE SECTS.)

DIAM = 0.29820

LENGTH= 118.330 SLOPE=0.00333000

DEPTH OF SEDIMENT= 15.00MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0.900	0.209	34.00	25.15	0.084	19.863	11788.	0.117	9.519	0.1215	0.02009	0.459
0.917	0.239	34.00	23.07	0.077	18.182	12399.	0.085	4.835	0.0585	0.01891	0.540
1.080	0.213	35.00	28.38	0.095	21.711	13319.	0.125	11.820	0.1361	0.02108	0.442
0.882	0.219	33.00	24.00	0.080	18.814	11867.	0.103	7.185	0.0955	0.01864	0.482
1.080	0.215	37.00	28.15	0.094	21.588	13258.	0.122	11.262	0.1304	0.02080	0.448
1.330	0.228	37.00	32.73	0.109	24.495	15873.	0.126	13.615	0.1390	0.02183	0.437

AVERAGE	STANDARD DEVIATION
FLOW= 1.028L/S	0.166
VELOCITY=0.219M/S	0.0098
Y/D=0.090	0.0120
KS= 9.705MM	3.2327
MANNING N=0.01985	0.001776
FROUD NO.=0.470	0.0394

MAY 23, 1984 MH 103-102 STONY PLAIN (PVC PLASTIC PIPE)

DIAM.= 0.25146

LENGTH= 48.47000 SLOPE=0.00180000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
0.990	0.231	30.00	35.85	0.141	22.104	15966	0.069	2.464	0.0278	0.01447	0.472
0.800	0.207	35.00	33.11	0.132	20.895	13420	0.088	3.382	0.0405	0.01845	0.438
0.880	0.215	36.00	33.90	0.135	21.155	14248	0.085	3.016	0.0356	0.01809	0.450
1.600	0.272	35.00	44.24	0.176	27.022	23025	0.052	2.201	0.0204	0.01405	0.485
1.080	0.227	34.00	37.71	0.150	23.350	16904	0.064	3.282	0.0351	0.01827	0.449
0.720	0.217	25.00	30.10	0.120	18.833	12870	0.057	1.802	0.0251	0.01359	0.483
0.590	0.193	18.00	28.20	0.112	17.802	10782	0.068	2.784	0.0388	0.01489	0.444
0.610	0.200	23.00	28.15	0.112	17.778	11137	0.063	2.302	0.0324	0.01445	0.460
0.730	0.214	23.00	30.38	0.121	18.103	12408	0.059	2.122	0.0278	0.01417	0.474
0.680	0.200	22.00	30.32	0.121	19.083	11943	0.067	2.984	0.0369	0.01514	0.443
0.580	0.189	22.00	27.29	0.109	17.262	10781	0.082	2.117	0.0307	0.01424	0.468
0.610	0.199	21.00	28.25	0.112	17.833	11117	0.064	2.390	0.0335	0.01488	0.457
0.610	0.218	23.00	32.23	0.128	20.180	13781	0.060	2.364	0.0293	0.01443	0.488

AVERAGE	STANDARD DEVIATION
FLOW= 0.819L/S	0.279
VELOCITY=0.215M/S	0.0206
Y/D=0.128	0.0190
KS= 2.557MM	0.4703
MANNING N=0.01483	0.000504
FROUD NO =0.462	0.0168

MAY 29, 1984 MH 102-101 STONY PLAIN (PVC PLASTIC PIPE)

DIAM.= 0.25146

LENGTH= 93.87000 SLOPE=0.00212000

Q(L/S)	V(M/S)	MEAS DEPTH	CAL. DEPTH	Y/D	CAL R	REY NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0.795	0.215	31.00	32.15	0.128	20.134	12841	0.072	3.749	0.0466	0.01585	0.482
0.505	0.185	25.00	26.10	0.104	16.547	9081	0.080	3.820	0.0577	0.01615	0.443
1.015	0.235	37.00	35.74	0.142	22.218	18489	0.057	3.487	0.0390	0.01548	0.478
0.730	0.213	31.00	29.92	0.118	18.821	11893	0.069	3.105	0.0413	0.01530	0.475
0.731	0.220	31.00	29.65	0.119	18.783	12256	0.065	2.536	0.0351	0.01479	0.491
0.695	0.205	31.00	30.25	0.120	19.027	11571	0.075	3.545	0.0505	0.01601	0.456
0.529	0.185	27.00	26.94	0.107	17.051	8358	0.083	4.211	0.0617	0.01649	0.436

AVERAGE	STANDARD DEVIATION
FLOW= 0.712L/S	0.171
VELOCITY=0.208M/S	0.0183
Y/D=0.120	.0128
KS= 3.549MM	0.5279
MANNING N=0.01572	0.000576
FROUD NO.=0.463	0.0201

MAY 28, 1984 MH 103-101 STONY PLAIN (2 PVC PLASTIC PIPE SECTIONS)

DIAM = 0,28148

LENGTH= 143,34000 SLOPE=0,00200000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
0,840	0,188	35,00	35,30	0,140	21,888	13281.	0,088	6,281	0,0711	0,01772	0,408
0,887	0,227	43,00	35,40	0,141	22,023	16288.	0,087	3,462	0,0393	0,01848	0,484
0,888	0,188	31,00	40,08	0,159	24,680	14168.	0,110	10,722	0,1088	0,02018	0,361
0,717	0,180	35,00	33,80	0,134	21,089	11887.	0,102	8,027	0,0881	0,01897	0,377
0,888	0,174	27,00	39,19	0,156	24,172	12843.	0,125	13,224	0,1368	0,02149	0,336
1,126	0,208	37,00	41,77	0,158	25,843	16287.	0,093	8,213	0,0801	0,01870	0,390
0,948	0,184	32,00	40,39	0,161	24,870	13973.	0,118	11,818	0,1188	0,02071	0,352

AVERAGE	STANDARD DEVIATION
FLOW= 0,918L/S	0,128
VELOCITY=0,194M/S	0,0184
Y/D=0,151	0,0123
KS= 8,818MM	3,3739
MANNING N=0,01903	0,002025
FROUD NO.=0,384	0,0423

JUNE 4, 1984 MH 49-48 DEVON (PVC PLASTIC PIPE)

DIAM.= 0,29920

LENGTH= 78,260 SLOPE=0,00148000

DEPTH OF SEDIMENT= 34,00MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
1,150	0,188	77,00	31,48	0,105	25,301	13773.	0,108	10,895	0,1057	0,02009	0,314
1,400	0,201	79,00	31,46	0,105	25,289	18770.	0,073	4,825	0,0477	0,01849	0,382
1,440	0,203	79,00	31,98	0,107	25,639	17171.	0,072	4,829	0,0471	0,01848	0,383
1,550	0,198	81,00	35,12	0,117	27,724	18110.	0,082	6,921	0,0624	0,01780	0,356
1,550	0,198	80,00	35,52	0,119	27,987	18282.	0,083	7,127	0,0637	0,01791	0,355
1,270	0,188	81,00	30,61	0,102	24,712	15327.	0,081	5,991	0,0606	0,01738	0,362
1,420	0,174	79,00	38,23	0,121	28,483	16333.	0,109	12,309	0,1081	0,02081	0,310

AVERAGE	STANDARD DEVIATION
FLOW= 1,403L/S	0,152
VELOCITY=0,190M/S	0,0147
Y/D=0,111	0,0078
KS= 7,528MM	2,8873
MANNING N=0,01811	0,001841
FROUD NO.=0,352	0,0287

JUNE 11, 1984 MH 86-85 RIVERBEND (PVC PLASTIC PIPE)

DIAM = 0.20117

LENGTH= 92.45000 SLOPE=0.01410000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY NO.	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
0.428	0.489	16.00	13.08	0.065	8.443	13626	0.039	0.223	0.0066	0.01007	1.662
0.325	0.447	13.00	11.52	0.057	7.477	11192	0.041	0.243	0.0081	0.01016	1.616
0.509	0.556	16.00	13.48	0.067	8.705	16179	0.031	0.067	0.0019	0.00805	1.856
0.442	0.490	16.00	13.33	0.066	8.611	14130	0.040	0.284	0.0078	0.01018	1.646
0.234	0.413	15.00	9.74	0.048	6.347	8774	0.041	0.173	0.0064	0.00986	1.626
0.185	0.372	10.00	8.25	0.041	5.382	6736	0.043	0.147	0.0068	0.00979	1.569

AVERAGE	STANDARD DEVIATION
FLOW= 0.350L/S	0.133
VELOCITY=0.46(M/S	0.0644
Y/D=0.057	0.0107
KS= 0.187MM	0.0730
MANNING N=0.00985	0.000422
FROUD NO =1.669	0.0644

JUNE 12, 1984 MH, 303-302 BURNEWOOD (PVC PLASTIC PIPE)

DIAM.= 0.20120

LENGTH= 67.09000 SLOPE=0.00570000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
4.530	0.439	29.00	72.51	0.360	39.825	62082.	0.092	12.798	0.0803	0.02008	0.606
5.100	0.441	29.00	78.90	0.392	42.471	68475.	0.098	15.151	0.0892	0.02084	0.580
4.530	0.445	30.00	71.79	0.357	38.518	62415.	0.089	11.857	0.0750	0.01988	0.618
4.530	0.413	25.00	75.88	0.377	41.230	60435.	0.108	17.894	0.1073	0.02182	0.555
4.340	0.431	25.00	71.21	0.354	39.273	60078	0.095	13.171	0.0838	0.02024	0.602
3.660	0.433	26.00	62.68	0.312	35.473	54514.	0.085	9.561	0.0674	0.01882	0.640
5.270	0.448	30.00	80.18	0.398	42.982	68037	0.097	15.033	0.0874	0.02077	0.581
6.510	0.427	25.00	97.38	0.484	49.249	74638.	0.121	25.624	0.1301	0.02378	0.498
4.820	0.412	24.00	77.11	0.383	41.745	61042.	0.110	18.472	0.1106	0.02205	0.549

AVERAGE	STANDARD DEVIATION
FLOW= 4.788L/S	0.790
VELOCITY=0.432M/S	0.0127
Y/D=0.360	0.0489
KS=15.484MM	4.7143
MANNING N=0.02059	0.001470
FROUD NO.=0.582	0.0446

JUNE 14, 1984 MH. 614-613 LEDUC SOUTHPARK (PVC PLASTIC PIPE)

DIAM.= 0,20117

LENGTH= 84 72000 SLOPE=0,00415000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0,282	0,186	23,00	21,06	0,105	13,246	7586.	0,160	10,601	0,1998	0,02187	0,440
0 414	0,222	20,00	21,63	0,108	13,808	10571.	0,091	4,191	0,0759	0,01870	0,561
0,556	0,176	26,00	32,59	0,162	20,088	12311.	0,206	22,790	0,2841	0,02671	0,378
0,611	0,177	25,00	33,33	0,166	20,487	12493.	0,213	24,221	0,2956	0,02723	0,372
0 687	0,183	22 00	30,61	0,152	18,832	9888.	0,263	28,937	0,3421	0,02981	0,336
0,722	0,175	24,00	37,72	0,188	22,897	13819	0,244	31,989	0,3493	0,02958	0,346
0,698	0,165	28,00	42,19	0,210	25,306	16148.	0,241	34,905	0,3448	0,03001	0,343

AVERAGE	STANDARD DEVIATION
FLOW= 0,571L/S	0,202
VELOCITY=0,179M/S	0,0215
Y/D=0,156	0,0365
KS=22,519MM	11,2937
MANNING N=0,02803	0,004995
FROUD NO.=0,399	0,0673

JUNE 18, 1984 MH. 10-10A LEDUC ROMULUS (PVC PLASTIC PIPE)

DIAM.= 0,25146

LENGTH= 74,620 SLOPE=0,00233000

DEPTH OF SEDIMENT= 22.00MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0,520	0,081	23,00	35,47	0,141	25,567	7418.	0,743	101,939	0,9585	0,05315	0,148
0,295	0,096	18,00	18,31	0,073	15,230	5132.	0,282	25,981	0,4265	0,03035	0,244
0,751	0,080	21,00	44,32	0,176	31,810	9873.	0,716	119,231	0,9370	0,05385	0,148
0,277	0,095	21,00	17,80	0,071	14,859	4853.	0,303	26,388	0,4440	0,03081	0,240
0,512	0,072	17,00	38,77	0,154	28,575	7075.	1,014	133,103	1,1644	0,06284	0,128
0,744	0,083	27,00	42,66	0,170	30,871	9922.	0,680	107,845	0,8734	0,05087	0,156
0,381	0,093	23,00	22,64	0,091	18,416	5919.	0,388	41,924	0,5691	0,03612	0,210
0,411	0,088	16,00	34,38	0,137	25,820	5926.	1,078	125,031	1,2058	0,06377	0,123

AVERAGE	STANDARD DEVIATION
FLOW= 0,484L/S	0,185
VELOCITY=0,088M/S	0,0113
Y/D=0,127	0,0426
KS=88,180MM	45,7805
MANNING N=0,04773	0,013553
FROUD NO.=0,174	0,0493

JUNE 19, 1984 MH. 9-10 LEDUC ROMULUS (PVC PLASTIC PIPE)

DIAM.= 0.25146

LENGTH= 70.980 SLOPE=0.00357000

DEPTH OF SEDIMENT= 12.00MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
0.244	0.128	20.00	14.38	0.057	11.743	8336	0.201	12.783	0.2724	0.02412	0.367
0.534	0.171	22.00	21.84	0.087	16.645	10228	0.161	13.651	0.2036	0.02286	0.408
0.480	0.152	18.00	21.18	0.084	16.388	8808	0.188	17.433	0.2684	0.02621	0.387
0.417	0.151	22.00	18.70	0.074	15.424	8266	0.190	18.843	0.2538	0.02482	0.376
0.545	0.162	21.00	23.23	0.092	17.763	10213	0.190	18.054	0.2541	0.02611	0.373
0.565	0.140	20.00	26.98	0.107	20.177	10026	0.266	34.124	0.4228	0.03163	0.300
0.295	0.087	13.00	21.43	0.085	16.843	8688	0.494	47.078	0.7087	0.04022	0.231
0.325	0.133	13.00	17.77	0.071	14.118	8883	0.224	17.888	0.3134	0.02824	0.346
0.349	0.124	15.00	14.78	0.078	14.787	7033	0.221	19.263	0.3280	0.02866	0.340

AVERAGE	STANDARD DEVIATION
FLOW= 0.415L/S	0.118
VELOCITY=0.141M/S	0.0219
Y/D=0.082	0.0141
KS=21.751MM	11.2807
MANNING N=0.02743	0.008384
FROUD NO.=0.345	0.0516

JULY 17, 1984 MH. 124-123 SPRUCE GROVE (PVC PLASTIC PIPE)

DIAM = 0.20117

LENGTH= 58.72000 SLOPE=0.00567500

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
1.010	0.308	17.00	32.08	0.160	19.772	23557	0.082	6.256	0.0791	0.01783	0.882
0.348	0.233	16.00	18.77	0.093	11.980	10745	0.098	4.182	0.0878	0.01891	0.858
0.515	0.231	18.00	24.87	0.123	15.489	13796	0.129	8.917	0.1439	0.02026	0.867
0.525	0.263	18.00	22.87	0.114	14.427	14629	0.093	4.565	0.0795	0.01897	0.872
0.581	0.277	18.00	23.66	0.118	14.894	15907	0.088	4.106	0.0685	0.01846	0.898
0.614	0.315	21.00	22.50	0.112	14.207	17255	0.084	1.983	0.0349	0.01403	0.811
0.843	0.282	22.00	31.83	0.158	19.622	22091	0.102	7.582	0.0986	0.01877	0.828

AVERAGE	STANDARD DEVIATION
FLOW= 0.646L/S	0.240
VELOCITY=0.274M/S	0.0339
Y/D=0.125	0.0247
KS= 5.375MM	2.2572
MANNING N=0.01732	0.001552
FROUD NO.=0.871	0.0742

JULY 27, 1984 MH. 216-217 YELLOWBIRD-17 AVE. (CONCRETE PIPE)

DIAM.= 0,20480

LENGTH= 54,88000 SLOPE=0,00306000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
2,860	0,356	51,00	52,21	0,172	32,023	42658.	0,081	4,188	0,0328	0,01871	0,588
3,220	0,377	56,00	53,19	0,178	32,812	48995.	0,088	3,280	0,0251	0,01498	0,628
3,200	0,365	55,00	55,22	0,181	33,632	46065.	0,081	4,216	0,0321	0,01578	0,585
2,890	0,374	54,00	50,81	0,167	31,184	43768.	0,084	2,918	0,0234	0,01468	0,637
3,280	0,364	56,00	56,08	0,184	34,086	45282	0,088	5,226	0,0383	0,01643	0,572
2,380	0,381	50,00	42,44	0,139	26,447	37612.	0,044	1,384	0,0131	0,01288	0,712
2,890	0,367	56,00	46,26	0,158	28,784	39860.	0,056	3,128	0,0283	0,01488	0,624
2,850	0,348	55,00	49,12	0,161	30,239	39489.	0,080	3,783	0,0311	0,01543	0,603
2,440	0,344	54,00	46,77	0,153	28,906	37218.	0,059	3,401	0,0294	0,01515	0,611
1,850	0,307	46,00	43,24	0,142	26,947	31044.	0,068	4,595	0,0428	0,01620	0,587

AVERAGE	STANDARD DEVIATION
FLOW= 2,778L/S	0,448
VELOCITY=0,358M/S	0,0213
Y/D=0,163	0,0182
KS= 3,618MM	1,0671
MANNING N=0,01621	0,000998
FROUD NO =0,614	0,0408

JULY 31, 1984 MH. 341-342 YELLOWBIRD-18 AVE. (CONCRETE PIPE)

DIAM.= 0,20320

LENGTH= 59,74001 SLOPE=0,00800000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0,168	0,087	24.00	19,83	0,098	12,818	4404.	0,526	37,428	0,7417	0,03850	0,286
0,178	0,093	18,00	21,91	0,108	13,860	4880.	0,523	46,900	0,8459	0,04368	0,244
0,118	0,084	16,00	21,40	0,105	13,557	3109.	1,311	72,537	1,3378	0,05311	0,168
0,164	0,088	17,00	19,34	0,095	12,316	4349	0,502	35,179	0,7141	0,03844	0,273
0,114	0,081	13,00	21,59	0,107	13,728	3034	1,429	76,870	1,3962	0,05803	0,181
0,100	0,089	18,00	18,37	0,090	11,727	2800.	0,978	53,308	1,1384	0,05313	0,196

AVERAGE	STANDARD DEVIATION
FLOW= 0,138L/S	0,030
VELOCITY=0,080M/S	0,0175
Y/D=0,101	0,0071
KS=53,869MM	17,5316
MANNING N=0,05085	0,011959
FROUD NO =0,218	0,0494

AUG 7, 1984 MH 380-344 YELLOWBIRD-107 ST (CONCRETE PIPE)

DIAM = 0.30480

LENGTH= 41.32001 SLOPE=0.00331000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
4.280	0.380	64.00	66.78	0.219	38.648	50918	0.080	9.634	0.0598	0.01864	0.831
3.430	0.330	46.00	60.98	0.200	36.768	43083	0.088	10.624	0.0722	0.01926	0.810
3.840	0.330	44.00	62.34	0.208	37.487	43818	0.089	11.288	0.0751	0.01983	0.804
3.330	0.310	42.00	62.40	0.208	37.529	41291	0.101	14.222	0.0958	0.02040	0.474
4.210	0.344	46.00	68.38	0.224	40.666	48674	0.088	12.202	0.0750	0.01978	0.801
4.070	0.343	46.00	66.91	0.220	39.917	48894	0.088	11.880	0.0730	0.01986	0.806
3.830	0.336	46.00	64.78	0.213	38.795	46639	0.088	11.347	0.0731	0.01961	0.806
4.160	0.342	47.00	66.09	0.223	40.622	49199	0.090	12.344	0.0761	0.01968	0.800
4.120	0.336	47.00	68.19	0.224	40.688	48887	0.092	12.871	0.0799	0.02010	0.483
4.480	0.362	46.00	69.96	0.230	41.807	51888	0.087	11.826	0.0712	0.01959	0.806

AVERAGE	STANDARD DEVIATION
FLOW= 3.940L/S	0.365
VELOCITY=0.339M/S	0.0136
Y/D=0.216	0.0101
KS=11.810MM	1.3003
MANNING N=0.01967	0.000666
FROUD NO =0.803	0.0143

AUG 9, 1984 MH 204A-204 LAKE DISTRICT-95 ST (CONCRETE PIPE)

DIAM = 0.38100

LENGTH= 61.17000 SLOPE=0.00027000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY. NO	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
8.880	0.153	87.00	193.22	0.507	96.106	53877	0.087	27.365	0.0712	0.02253	0.125
7.610	0.134	82.00	189.94	0.498	95.072	46505	0.112	43.508	0.1144	0.02584	0.111
8.880	0.172	86.00	173.01	0.454	88.381	55106	0.064	13.119	0.0367	0.01908	0.151
9.220	0.179	91.00	178.08	0.462	90.434	59092	0.060	11.299	0.0312	0.01848	0.156
8.690	0.161	88.00	182.06	0.478	92.487	54357	0.076	18.723	0.0633	0.02067	0.137
8.880	0.168	87.00	178.16	0.462	90.470	55463	0.066	15.232	0.0421	0.01971	0.146

AVERAGE	STANDARD DEVIATION
FLOW= 8.615L/S	0.539
VELOCITY=0.161M/S	0.0160
Y/D=0.477	0.0218
KS=21.711MM	12.1267
MANNING N=0.02104	0.002629
FROUD NO =0.137	0.0169

AUG 18, 1984 MH 100-101 ST ALBERT LAROSE DR (CONCRETE PIPE)

DIAM = 0.25400

LENGTH= 77.280 SLOPE=0.00378000

DEPTH OF SEDIMENT= 7.80MM

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
2.700	0.388	77.80	44.24	0.175	29.869	43446	0.082	3.557	0.0301	0.01528	0.867
2.140	0.248	72.80	40.18	0.158	27.220	36047	0.087	4.385	0.0403	0.01801	0.831
2.810	0.389	70.80	43.40	0.171	29.063	40788	0.083	4.155	0.0357	0.01577	0.844
2.390	0.371	65.80	41.82	0.164	28.082	39803	0.081	3.669	0.0320	0.01832	0.861
2.740	0.381	81.80	48.30	0.174	30.120	43853	0.082	4.031	0.0335	0.01584	0.861
2.810	0.388	80.80	43.48	0.171	29.114	40788	0.084	4.224	0.0384	0.01882	0.842
2.880	0.377	82.80	44.90	0.177	29.887	42478	0.083	4.133	0.0348	0.01873	0.847
2.220	0.328	87.80	42.24	0.166	24.418	38524	0.074	5.744	0.0805	0.01886	0.888
1.720	0.318	82.80	38.42	0.143	25.090	30445	0.073	4.857	0.0484	0.01884	0.808

AVERAGE	STANDARD DEVIATION
FLOW= 2.402L/S	0.327
VELOCITY=0.382M/S	0.0222
Y/D=0.167	0.0110
KS= 4.209MM	0.8807
MANNING N=0.01580	0.000848
FROUD NO.=0.839	0.0223

AUG 28, 1984 MH 17-18 ST ALBERT MCKENNEY AVE (CONCRETE PIPE)

DIAM = 0.20320

LENGTH= 84.10001 SLOPE=0.00840700

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
0.880	0.294	13.00	24.43	0.120	15.383	18468	0.131	9.080	0.1478	0.02039	0.728
1.280	0.409	17.00	31.02	0.153	19.183	28841	0.085	5.134	0.0669	0.01889	0.892
0.820	0.387	20.00	26.83	0.131	16.852	22308	0.081	5.189	0.0774	0.01723	0.887
0.820	0.327	16.00	25.84	0.131	16.855	18881	0.115	7.900	0.1185	0.01834	0.772
1.210	0.400	17.00	30.31	0.149	18.773	27412	0.087	5.259	0.0700	0.01713	0.883
0.510	0.288	18.00	21.10	0.104	13.275	13904	0.121	6.865	0.1283	0.01811	0.781
0.720	0.285	15.00	26.71	0.131	16.697	17432	0.151	12.270	0.1837	0.02215	0.674
1.040	0.385	19.00	29.08	0.143	18.071	24079	0.100	6.898	0.0928	0.01830	0.824
0.880	0.358	15.00	27.89	0.137	17.385	22720	0.100	6.435	0.0925	0.01818	0.828
0.540	0.283	13.00	22.09	0.109	13.873	14435	0.129	7.999	0.1431	0.01988	0.738

AVERAGE	STANDARD DEVIATION
FLOW= 0.865L/S	0.265
VELOCITY=0.337M/S	0.0487
Y/D=0.131	0.0161
KS= 7.280MM	2.1973
MANNING N=0.01887	0.001648
FROUD NO.=0.785	0.0736

AUG 28, 1984 MH 17-18 THORNDALE INDUSTRIAL
AUGMENTED FLOW TESTS ON PVC PLASTIC PIPE

DIAM = 0.29920

LENGTH= 88.28 SLOPE=0.00382000

DEPTH OF SEDIMENT= 12.00

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
0.820	0.284	21.00	20.11	0.067	18.880	14191	0.074	2.122	0.0488	0.01840	0.633
0.850	0.284	22.00	21.00	0.070	18.888	14808	0.079	2.778	0.0870	0.01802	0.611
0.790	0.281	21.00	20.88	0.069	18.212	13884	0.087	6.491	0.0884	0.01773	0.584

AVERAGE — STANDARD DEVIATION

AVERAGE	STANDARD DEVIATION
FLOW= 0.820L/S	0.030
VELOCITY=0.281M/S	0.0089
Y/D=0.068	0.0015
KS= 3.797MM	0.8881
MANNING N=0.01805	0.000888
FROUD NO =0.609	0.0244

DEPTH OF SEDIMENT= 1.00

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
4.260	0.484	54.00	56.30	0.188	24.630	58722	0.082	3.002	0.0217	0.01467	0.726
4.310	0.481	52.00	57.03	0.191	25.028	59016	0.083	3.241	0.0231	0.01484	0.718
4.360	0.478	53.00	57.77	0.193	25.430	59285	0.084	3.496	0.0247	0.01510	0.707

AVERAGE	STANDARD DEVIATION
FLOW= 4.310L/S	0.050
VELOCITY=0.481M/S	0.0030
Y/D=0.191	0.0025
KS= 3.246MM	0.2470
MANNING N=0.01488	0.000212
FROUD NO =0.717	0.0098

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
5.210	0.474	74.00	63.89	0.214	28.238	67898	0.082	3.444	0.0228	0.01501	0.716
5.350	0.470	74.00	64.89	0.217	28.621	68074	0.083	3.599	0.0232	0.01513	0.710
5.210	0.460	72.00	66.28	0.221	29.488	66562	0.090	5.008	0.0317	0.01616	0.666

AVERAGE	STANDARD DEVIATION
FLOW= 5.257L/S	0.081
VELOCITY=0.468M/S	0.0142
Y/D=0.217	0.0040
KS= 4.017MM	0.8616
MANNING N=0.01543	0.000628
FROUD NO =0.697	0.0271

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
12.010	0.641	105.00	93.37	0.312	52.624	127983	0.040	2.197	0.0104	0.01377	0.787
11.740	0.638	104.00	92.06	0.308	52.216	128116	0.039	2.138	0.0102	0.01371	0.781
11.870	0.635	104.00	93.22	0.312	52.782	128812	0.040	2.312	0.0110	0.01389	0.781

AVERAGE	STANDARD DEVIATION
FLOW=11.873L/S	0.135
VELOCITY=0.638M/S	0.0031
Y/D=0.310	0.0024
KS= 2.216MM	0.0886
MANNING N=0.01379	0.000092
FROUD NO.=0.783	0.0053

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
21.770	0.827	129.00	119.89	0.401	64.180	201180	0.029	0.855	0.0037	0.01216	0.681
21.770	0.830	142.00	119.57	0.400	64.051	201814	[illegible]	0.817	0.0035	[illegible]	0.688

AVERAGE	STANDARD DEVIATION
FLOW=21.770L/S	0.0
VELOCITY=0.828M/S	0.0021
Y/D=0.400	0.0008
KS= 0.836MM	0.0260
MANNING N=0.01212	0.000043
FROUD NO.=0.684	0.0033

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
38.210	0.914	172.00	175.88	0.888	82.229	288314	0.029	1.478	0.0048	0.01287	0.784
38.760	0.927	174.00	171.88	0.878	81.217	288812	0.028	1.282	0.0038	0.01288	0.787
38.220	0.930	172.00	168.82	0.867	80.881	284828	0.028	1.188	0.0037	0.01288	0.788
38.220	0.919	172.00	171.18	0.872	81.024	282871	0.030	1.228	0.0041	0.01277	0.782
38.220	0.912	176.00	173.10	0.878	81.280	281888	0.030	1.808	0.0042	0.01288	0.778

AVERAGE	STANDARD DEVIATION
FLOW=38.888L/S	0.667
VELOCITY=0.921M/S	0.0076
Y/D=0.876	0.0080
KS= 1.322MM	0.1128
MANNING N=0.01278	0.000188
FROUD NO =0.781	0.0128

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
30.280	0.870	182.00	166.25	0.888	76.601	268284	0.030	1.320	0.0044	0.01278	0.818
30.280	0.882	168.00	166.62	0.888	76.747	266186	0.031	1.627	0.0048	0.01280	0.802
30.280	0.886	180.00	180.28	0.802	78.007	243313	0.032	1.832	0.0081	0.01303	0.788

AVERAGE	STANDARD DEVIATION
FLOW=30.280L/S	0.000
VELOCITY=0.883M/S	0.0070
Y/D=0.888	0.0032
KS= 1.630MM	0.1091
MANNING N=0.01288	0.000140
FROUD NO =0.804	0.0084

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
10.120	0.712	118.00	78.80	0.288	66.772	118838	0.027	0.683	0.0028	0.01110	0.874
10.870	0.704	120.00	78.84	0.288	68.284	122880	0.028	0.842	0.0038	0.01144	0.843
10.870	0.707	121.00	79.60	0.288	68.188	122797	0.028	0.812	0.0033	0.01141	0.848

AVERAGE	STANDARD DEVIATION
FLOW=10.620L/S	0.280
VELOCITY=0.708M/S	0.0040
Y/D=0.283	0.0087
KS= 0.882MM	0.0787
MANNING N=0.01133	0.000201
FROUD NO =0.888	0.0188

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
3.120	0.381	59.00	81.10	0.171	31.289	44873	0.063	6.428	0.0383	0.01882	0.863
3.200	0.390	60.00	82.10	0.174	31.883	45881	0.065	6.783	0.0373	0.01815	0.855

AVERAGE	STANDARD DEVIATION
FLOW= 3.180L/S	0.057
VELOCITY=0.390M/S	0.0007
Y/D=0.172	0.0024
KS= 4.585MM	0.2321
MANNING N=0.01804	0.000182
FROUD NO =0.859	0.0059

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
1.080	0.283	28.00	32.37	0.108	20.478	18571	0.091	5.322	0.0772	0.01784	0.585
1.280	0.287	29.00	33.70	0.113	21.288	21080	0.080	4.878	0.0585	0.01677	0.604
1.280	0.283	29.00	34.78	0.118	21.801	21374	0.084	5.780	0.0880	0.01734	0.588

AVERAGE	STANDARD DEVIATION
FLOW= 1.207L/S	0.112
VELOCITY=0.278M/S	0.0129
Y/D=0.112	0.0040
KS= 5.553MM	0.5774
MANNING N=0.01732	0.000527
FROUD NO.=0.585	0.0135

AUG 30, 1988 MH 103-102 STORM PLAIN
AUGMENTED FLOW TESTS ON PVC PLASTIC PIPE

DIAM = 0.25400

LENGTH= 48.47 SLOPE= 0.0180000

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
25.800	0.862	181.00	186.78	0.735	75.818	172276	0.025	0.823	0.0021	0.01163	0.888
25.800	0.865	180.00	183.88	0.732	75.434	173662	0.026	0.895	0.0029	0.01188	0.900
26.000	0.833	180.00	193.82	0.771	76.220	168808	0.027	0.835	0.0027	0.01208	0.888
25.700	0.848	180.00	187.28	0.749	75.780	171742	0.026	0.887	0.0022	0.01172	0.888
25.000	0.847	178.00	182.88	0.725	75.301	170662	0.026	0.884	0.0022	0.01188	0.888

AVERAGE	STANDARD DEVIATION
FLOW=25.660L/S	0.358
VELOCITY=0.857M/S	0.0088
Y/D=0.742	0.0178
KS= 0.879MM	0.0827
MANNING N=0.01172	0.000188
FROUD NO =0.888	0.0172

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
15.300	0.888	124.00	141.78	0.564	67.542	127214	0.030	1.094	0.0040	0.01248	0.830
15.900	0.887	128.00	140.84	0.559	67.218	124718	0.031	1.188	0.0044	0.01259	0.826
16.100	0.896	122.00	135.19	0.534	65.729	121283	0.030	1.094	0.0042	0.01248	0.828

AVERAGE	STANDARD DEVIATION
FLOW=15.767L/S	0.811
VELOCITY=0.890M/S	0.0053
Y/D=0.552	0.0129
KS= 1.125MM	0.0533
MANNING N=0.01250	0.000061
FROUD NO =0.828	0.0060

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
8.730	0.444	71.00	105.08	0.418	55.821	80640	0.040	2.318	0.0104	0.01392	0.503
9.110	0.461	73.00	107.24	0.428	56.441	83427	0.038	2.228	0.0099	0.01384	0.505
8.390	0.440	72.00	102.88	0.408	54.664	66328	0.040	2.298	0.0104	0.01390	0.506

AVERAGE	STANDARD DEVIATION
FLOW= 8.743L/S	0.360
VELOCITY=0.448M/S	0.0086
Y/D=0.418	0.0081
KS= 2.281MM	0.0408
MANNING N=0.01389	0.000041
FROUD NO =0.505	0.0012

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
6.710	0.386	50.00	98.70	0.388	53.519	72312	0.058	5.818	0.0272	0.01648	0.428
6.710	0.382	50.00	100.52	0.400	53.848	71880	0.058	6.273	0.0291	0.01671	0.421
6.530	0.382	51.00	100.59	0.400	53.870	70003	0.061	7.195	0.0334	0.01719	0.410

AVERAGE	STANDARD DEVIATION
FLOW= 6.650L/S	0.104
VELOCITY=0.380M/S	0.0072
Y/D=0.399	0.0020
KS= 6.428MM	0.7030
MANNING N=0.01679	0.000371
FROUD NO =0.420	0.0094

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
0.716	0.128	26.00	43.19	0.172	26.441	12502	0.235	35.438	0.3351	0.02988	0.232
0.663	0.122	23.00	41.88	0.167	25.706	11768	0.244	35.871	0.3488	0.03029	0.228
0.955	0.153	27.00	46.10	0.183	26.049	16104	0.189	24.432	0.2176	0.02880	0.273

AVERAGE	STANDARD DEVIATION
FLOW= 0.778L/S	0.156
VELOCITY=0.134M/S	0.0159
Y/D=0.174	0.0088
KS=31.947MM	6.5129
MANNING N=0.02859	0.002599
FROUD NO =0.248	0.0244

SEPT 10, 1984 MH. 811-812 LEDUC ROMULUS
AUGMENTED FLOW TESTS ON PVC PLASTIC PIPE

DIAM = 0.25146

LENGTH= 40.00 ,SLOPE=0.00384000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
0.730	0.258	53.00	26.98	0.107	17.084	15207	0.073	3.249	0.0478	0.01848	0.800
0.810	0.248	53.00	24.51	0.097	15.551	13349	0.072	2.881	0.0462	0.01818	0.805
0.840	0.242	48.00	25.84	0.102	16.212	13711	0.077	3.484	0.0534	0.01878	0.886

AVERAGE	STANDARD DEVIATION
FLOW= 0.800L/S	0.082
VELOCITY=0.247M/S	0.0064
Y/D=0.102	0.0048
KS= 3.108MM	0.2852
MANNING N=0.01848	0.000288
FROUD NO =0.897	0.0103

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
2.480	0.381	74.00	46.78	0.186	28.404	38807	0.082	2.386	0.0211	0.01417	0.882
2.340	0.386	74.00	45.16	0.180	27.838	38853	0.081	2.288	0.0207	0.01404	0.898
2.200	0.383	72.00	43.82	0.173	26.821	35443	0.080	2.098	0.0197	0.01388	0.704

AVERAGE	STANDARD DEVIATION
FLOW= 2.343L/S	0.148
VELOCITY=0.387M/S	0.0040
Y/D=0.180	0.0064
KS= 2.258MM	0.1818
MANNING N=0.01402	0.000180
FROUD NO =0.897	0.0081

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
8.450	0.884	108.00	82.72	0.328	48.328	104483	0.029	0.882	0.0036	0.01188	0.884
8.550	0.883	107.00	83.48	0.332	48.852	108028	0.029	0.888	0.0037	0.01183	0.888
8.450	0.885	106.00	83.88	0.332	48.898	103858	0.030	0.768	0.0041	0.01178	0.887

AVERAGE	STANDARD DEVIATION
FLOW= 8.483L/S	0.058
VELOCITY=0.881M/S	0.0049
Y/D=0.331	0.0018
KS= 0.708MM	0.0527
MANNING N=0.01186	0.000113
FROUD NO.=0.887	0.0084

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
17.380	0.842	142.00	109.07	0.434	57.122	158877	0.022	0.287	0.0012	0.01048	0.934
18.120	0.844	141.00	112.33	0.447	58.317	162381	0.023	0.288	0.0013	0.01080	0.920
17.620	0.844	142.00	109.98	0.437	57.453	159578	0.022	0.271	0.0012	0.01050	0.932
18.120	0.837	143.00	113.08	0.450	58.578	161748	0.023	0.333	0.0014	0.01072	0.908
19.220	0.839	143.00	118.08	0.470	60.338	187007	0.024	0.390	0.0016	0.01091	0.887

AVERAGE	STANDARD DEVIATION
FLOW=18.094L/S	0.708
VELOCITY=0.841M/S	0.0031
Y/D=0.447	0.0140
KS= 0.312MM	0.0511
MANNING N=0.01084	0.000178
FROUD NO.=0.918	0.0183

OCT 19, 1984 MH 103-102 STONY PLAIN
AUGMENTED FLOW TESTS ON PVC PLASTIC PIPE

DIAM = 0.25146

LENGTH= 46.47 SLOPE=0.00180000

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
8.610	0.287	80.00	88.19	0.275	40.001	81487	0.036	1.122	0.0070	0.01280	0.870
8.040	0.287	82.00	86.19	0.282	36.830	68248	0.036	1.124	0.0073	0.01281	0.868
8.840	0.298	82.00	70.80	0.282	40.828	82268	0.037	1.277	0.0076	0.01274	0.860

AVERAGE	STANDARD DEVIATION
FLOW=8.220L/S	0.280
VELOCITY=0.283M/S	0.0052
Y/D=0.272	0.0058
KS= 1.175MM	0.0866
MANNING N=0.01268	0.000124
FROUD NO =0.868	0.0054

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
14.310	0.552	105.00	130.06	0.517	64.216	114950	0.030	1.005	0.0038	0.01233	0.546
11.860	0.521	106.00	116.27	0.470	60.400	102048	0.031	1.144	0.0047	0.01264	0.550
12.840	0.526	110.00	124.44	0.495	62.452	106926	0.032	1.216	0.0048	0.01268	0.540

AVERAGE	STANDARD DEVIATION
FLOW=13.070L/S	1.180
VELOCITY=0.534M/S	0.0163
Y/D=0.494	0.0235
KS= 1.122MM	0.1081
MANNING N=0.01250	0.000164
FROUD NO =0.545	0.0055

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
26.380	0.850	170.00	186.53	0.750	75.857	162523	0.026	0.587	0.0019	0.01162	0.462
27.580	0.840	177.00	203.53	0.809	76.516	166974	0.026	0.772	0.0025	0.01195	0.436
26.080	0.848	175.00	185.86	0.755	75.867	159799	0.026	0.670	0.0022	0.01174	0.479
26.200	0.844	178.00	207.26	0.824	76.486	159822	0.026	0.722	0.0024	0.01187	0.430
26.200	0.845	178.00	208.91	0.823	76.503	160182	0.026	0.723	0.0024	0.01185	0.432
26.380	0.841	170.00	194.05	0.772	76.233	158626	0.026	0.747	0.0025	0.01190	0.464

AVERAGE	STANDARD DEVIATION
FLOW=27.127L/S	0.976
VELOCITY=0.846M/S	0.0073
Y/D=0.789	0.0340
KS= 0.702MM	0.0744
MANNING N=0.01181	0.000155
FROUD NO =0.455	0.0266

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
13.470	0.508	101.00	132.43	0.527	[illegible]	108863	0.036	1.894	0.0073	0.01349	0.499
12.830	0.517	106.00	124.13	0.494	62.352	104533	0.037	1.403	0.0056	0.01290	0.530
12.370	0.516	105.00	122.32	0.488	61.759	103329	0.033	1.361	0.0055	0.01285	0.533

AVERAGE	STANDARD DEVIATION
FLOW=12.823L/S	0.575
VELOCITY=0.514M/S	0.0049
Y/D=0.502	0.0214
KS= 1.553MM	0.2865
MANNING N=0.01308	0.000257
FROUD NO.=0.521	0.0188

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
10.580	0.488	88.00	113.18	0.450	58.520	92736	0.035	1.573	0.0067	0.01312	0.529
10.870	0.488	81.00	115.58	0.460	59.480	94084	0.035	1.576	0.0070	0.01324	0.523
12.210	0.487	81.00	126.59	0.504	63.168	99725	0.038	2.209	0.0087	0.01362	0.482

AVERAGE	STANDARD DEVIATION
FLOW=11.220L/S	0.870
VELOCITY=0.488M/S	0.0008
Y/D=0.471	0.0267
KS= 1.819MM	0.3410
MANNING N=0.01333	0.000272
FROUD NO =0.515	0.0157

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
8.470	0.408	52.00	78.21	0.315	44.725	89015.	0.038	1.550	0.0087	0.01310	0.544
8.550	0.393	48.00	82.25	0.327	48.118	88802.	0.042	2.231	0.0121	0.01388	0.512
8.400	0.392	49.00	80.79	0.321	46.482	87810.	0.042	2.134	0.0117	0.01378	0.517

AVERAGE	STANDARD DEVIATION
FLOW= 8.473L/S	0.075
VELOCITY=0.398M/S	0.0080
Y/D=0.321	0.0061
KS= 1.972MM	0.3881
MANNING N=0.01359	0.000424
FROUD NO.=0.524	0.0168

Q(L/S)	V(M/S)	MEAS DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
1.840	0.294	28.00	48.78	0.198	30.083	28391	0.061	3.822	0.0318	0.01553	0.482
1.720	0.282	26.00	47.75	0.190	28.980	24274	0.060	3.478	0.0300	0.01527	0.459
1.890	0.290	31.00	48.89	0.194	29.458	28397	0.053	2.623	0.0223	0.01445	0.465

AVERAGE	STANDARD DEVIATION
FLOW= 1.817L/S	0.087
VELOCITY=0.289M/S	0.0089
Y/D=0.194	0.0041
KS= 3.308MM	0.5170
MANNING N=0.01508	0.000553
FROUD NO.=0.469	0.0178

SEPT 24, 1984 MH. R6-R7 124 AVE & 43 ST EDMONTON
AUGMENTED FLOW TESTS ON CONCRETE PIPE

DIAM.= 0.30480

LENGTH= 107.25 SLOPE=0.00308620

DEPTH OF SEDIMENT= 9.50

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
4.250	0.331	128.50	62.84	0.206	41.361	44849	0.091	12.982	0.0788	0.02008	0.476
4.560	0.336	127.50	65.76	0.216	42.923	47083	0.093	13.414	0.0806	0.02032	0.471
4.560	0.336	128.50	65.76	0.216	42.923	47083	0.093	13.414	0.0806	0.02032	0.471

AVERAGE	STANDARD DEVIATION
FLOW= 4.457L/S	0.179
VELOCITY=0.334M/S	0.0022
Y/D=0.216	0.0066
KS=12.940MM	0.4746
MANNING N=0.02025	0.000144
FROUD NO.=0.473	0.0029

DEPTH OF SEDIMENT= 9.50

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY. NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
10.590	0.474	151.50	98.06	0.322	57.985	89949	0.052	8.089	0.0349	0.01755	0.846
10.590	0.477	153.50	97.56	0.320	57.742	90185	0.051	7.754	0.0336	0.01740	0.851
10.940	0.481	153.50	99.44	0.326	58.561	92231	0.051	7.817	0.0334	0.01742	0.850

AVERAGE	STANDARD DEVIATION
FLOW=10.707L/S	0.202
VELOCITY=0.477M/S	0.0035
Y/D=0.323	0.0032
KS= 7.887MM	0.1780
MANNING N=0.01746	0.000084
FROUD NO.=0.848	0.0028

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY NO.	FRIC FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
24.770	0.874	208.00	153.28	0.503	78.478	168782	0.041	3.489	0.0114	0.01486	0.820
24.420	0.882	201.00	150.18	0.493	78.487	168571	0.039	3.066	0.0102	0.01455	0.835
25.210	0.886	203.00	153.27	0.503	78.477	171784	0.039	3.141	0.0103	0.01459	0.831
24.340	0.878	204.00	150.31	0.493	78.530	167826	0.040	3.176	0.0106	0.01462	0.832
23.930	0.877	204.00	148.67	0.488	74.995	166245	0.040	3.142	0.0105	0.01458	0.835

AVERAGE	STANDARD DEVIATION
FLOW=24.534L/S	0.482
VELOCITY=0.880M/S	0.0045
Y/D=0.496	0.0087
KS= 3.207MM	0.1811
MANNING N=0.01464	0.000120
FROUD NO.=0.830	0.0053

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
15.510	0.568	180.00	122.09	0.401	85.364	121888	0.049	4.981	0.0191	0.01567	0.600
15.020	0.562	158.00	120.15	0.394	84.590	118858	0.050	5.064	0.0198	0.01581	0.598
14.700	0.560	159.00	118.55	0.388	83.946	117255	0.049	4.964	0.0194	0.01586	0.602

AVERAGE	STANDARD DEVIATION
FLOW=15.077L/S	0.408
VELOCITY=0.563M/S	0.0042
Y/D=0.395	0.0058
KS= 5.003MM	0.0482
MANNING N=0.01568	0.000027
FROUD NO.=0.600	0.0013

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
4.960	0.374	120.00	72.37	0.237	42.745	52348	0.074	8.598	0.0509	0.01816	0.626
4.960	0.370	114.00	72.82	0.239	43.027	52128	0.075	9.302	0.0540	0.01844	0.620
4.790	0.373	118.00	70.74	0.232	41.909	51185	0.073	8.262	0.0493	0.01797	0.633

AVERAGE	STANDARD DEVIATION
FLOW= 4.903L/S	0.098
VELOCITY=0.372M/S	0.0021
Y/D=0.236	0.0037
KS= 8.754MM	0.5219
MANNING N=0.01819	0.000235

FROUD NO.=0.527 0.0064

Q(L/S)	V(M/S)	MEAS DEPTH	CAL. DEPTH	Y/D	CAL. R	REY NO.	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
1.830	0.182	50.00	52.97	0.174	22.368	20362	0.213	38.414	0.2886	0.02940	0.320
1.820	0.182	77.00	52.74	0.173	22.282	20283	0.212	38.074	0.2851	0.02932	0.320
1.870	0.184	83.00	55.48	0.182	23.778	20381	0.242	46.821	0.3488	0.03155	0.299

AVERAGE	STANDARD DEVIATION
FLOW= 1.840L/S	0.026
VELOCITY=0.183M/S	0.0046
Y/D=0.176	0.0050
KS=41.104MM	4.8839
MANNING N=0.03008	0.001284
FROUD NO =0.313	0.0122

OCT. 1, 1984 MH. A14-A13 69 AVE. & 42 ST. EDMONTON
AUGMENTED FLOW TESTS ON CONCRETE PIPE

DIAM.= 0.30480

LENGTH= 122.21 SLOPE=0.00484390

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL. R	REY NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
3.400	0.478	42.00	47.00	0.154	28.027	44233	0.048	2.104	0.0181	0.01381	0.844
3.280	0.471	43.00	46.18	0.152	28.572	43069	0.048	2.097	0.0183	0.01381	0.842
3.240	0.472	42.00	45.73	0.150	28.312	42768	0.048	2.000	0.0177	0.01370	0.848

AVERAGE	STANDARD DEVIATION
FLOW= 3.307L/S	0.083
VELOCITY=0.473M/S	0.0026
Y/D=0.152	0.0021
KS= 2.067MM	0.0580
MANNING N=0.01377	0.000066
FROUD NO =0.845	0.0033

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
8.610	0.883	82.00	69.82	0.229	41.438	90569	0.034	1.003	0.0060	0.01220	0.883
8.610	0.883	83.00	69.82	0.229	41.438	90569	0.034	1.003	0.0060	0.01220	0.883
8.480	0.883	83.00	69.08	0.227	41.046	89728	0.033	0.960	0.0058	0.01213	0.888

AVERAGE	STANDARD DEVIATION
FLOW= 8.567L/S	0.078
VELOCITY=0.883M/S	0.0000
Y/D=0.228	0.0014
KS= 0.988MM	0.0245
MANNING N=0.01218	0.000044
FROUD NO =0.885	0.0033

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
15.810	0.827	81.00	93.10	0.305	52.676	139951	0.029	0.808	0.0038	0.01186	1.016
15.690	0.835	81.00	97.86	0.321	55.075	147176	0.030	0.913	0.0041	0.01207	1.000
15.230	0.828	81.00	95.66	0.314	54.066	143274	0.030	0.889	0.0041	0.01202	1.004

AVERAGE	STANDARD DEVIATION
FLOW=15.243L/S	0.640
VELOCITY=0.830M/S	0.0044
Y/D=0.313	0.0079
KS= 0.869MM	0.0586
MANNING N=0.01198	0.000110
FROUD NO =1.008	0.0098

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
30.080	0.989	111.00	132.43	0.434	69.324	219421	0.027	0.789	0.0028	0.01188	0.988
28.090	0.991	112.00	125.56	0.412	66.722	211813	0.026	0.639	0.0024	0.01165	1.028
28.350	0.991	113.00	128.43	0.418	67.080	212884	0.026	0.666	0.0024	0.01169	1.025
29.370	0.997	113.00	129.26	0.424	68.139	217413	0.026	0.676	0.0025	0.01168	1.016
26.930	0.992	113.00	121.56	0.399	65.153	206843	0.025	0.559	0.0021	0.01135	1.050
30.830	0.995	113.00	134.32	0.441	70.016	222980	0.027	0.791	0.0028	0.01188	0.983

AVERAGE	STANDARD DEVIATION
FLOW=28.942L/S	1.426
VELOCITY=0.992M/S	0.0029
Y/D=0.421	0.0154
KS= 0.685MM	0.0905
MANNING N=0.01165	0.000201
FROUD NO.=1.016	0.0218

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO
17.130	0.843	84.00	98.21	0.322	55.222	148884	0.030	0.862	0.0039	0.01197	1.006
16.890	0.840	82.00	97.48	0.320	54.883	147540	0.030	0.859	0.0039	0.01197	1.008
16.230	0.838	83.00	94.85	0.311	53.687	143984	0.029	0.786	0.0037	0.01182	1.021

AVERAGE	STANDARD DEVIATION
FLOW=16.750L/S	0.466
VELOCITY=0.840M/S	0.0025
Y/D=0.318	0.0058
KS= 0.835MM	0.0434
MANNING N=0.01192	0.000085
FROUD NO =1.013	0.0075

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
11,880	0,752	72,00	80,21	0,283	48,742	112482.	0,031	0,893	0,0046	0,01201	1,004
11,880	0,751	70,00	80,38	0,284	48,780	112433.	0,032	0,805	0,0048	0,01203	1,002
11,880	0,749	71,00	80,54	0,284	48,858	112314.	0,032	0,829	0,0050	0,01206	0,998

AVERAGE — STANDARD DEVIATION

FLOW=11,880L/S — 0,0

VELOCITY=0,751M/S — 0,0015

Y/D=0,284 — 0,0004

KS=0,842MM — 0,0166

MANNING N=0,01204 — 0,000024

FROUD NO.=1,001 — 0,0028

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
8,120	0,670	64,00	67,91	0,222	40,441	85716.	0,034	1,024	0,0063	0,01224	0,979
8,540	0,669	63,00	70,44	0,231	41,768	89389.	0,035	1,195	0,0072	0,01252	0,858
8,280	0,670	63,00	68,88	0,226	40,831	87765.	0,035	1,082	0,0066	0,01234	0,872

AVERAGE — STANDARD DEVIATION

FLOW= 8,313L/S — 0,212

VELOCITY=0,670M/S — 0,0006

Y/D=0,227 — 0,0042

KS= 1,100MM — 0,0868

MANNING N=0,01237 — 0,000143

FROUD NO.=0,870 — 0,0103

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO.	FRIC. FAC.	KS(MM)	KS/4R	MANNING N	FROUD NO.
2,890	0,320	32,00	61,24	0,168	31,427	32185.	0,117	15,351	0,1221	0,02166	0,842
2,870	0,321	33,00	60,86	0,167	31,212	32065.	0,115	14,905	0,1194	0,02148	0,846
2,830	0,320	33,00	60,42	0,165	30,988	31712.	0,116	14,743	0,1190	0,02146	0,847

AVERAGE — STANDARD DEVIATION

FLOW= 2,863L/S — 0,031

VELOCITY=0,320M/S — 0,0005

Y/D=0,167 — 0,0014

KS=15,000MM — 0,3150

MANNING N=0,02153 — 0,000112

FROUD NO.=0,845 — 0,0025

OCT. 5, 1984 MH 315-317 YELLOWBIRD -17 AVE.
AUGMENTED FLOW TESTS ON CONCRETE PIPE

DIAM.= 0.30480

LENGTH= 54.86 SLOPE=0.00308000

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
9.120	0.815	89.00	88.92	0.292	80.813	85890	0.048	3.282	0.0180	0.01479	0.850
9.800	0.821	100.00	91.80	0.300	82.129	82074	0.048	3.338	0.0180	0.01482	0.848
9.250	0.801	99.00	91.63	0.301	82.191	88848	0.050	4.150	0.0189	0.01542	0.822

AVERAGE STANDARD DEVIATION

FLOW= 9.223L/S 0.248

VELOCITY=0.812M/S 0.0103

Y/D=0.298 0.0050

KS= 3.590MM 0.4983

MANNING N=0.01500 0.000388

FROUD NO.=0.840 0.0188

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
16.190	0.820	132.00	118.08	0.387	83.799	131878	0.040	2.884	0.0106	0.01424	0.868
17.070	0.823	130.00	122.40	0.402	85.488	136796	0.041	2.918	0.0111	0.01442	0.867
16.820	0.816	123.00	121.00	0.397	84.931	133128	0.041	3.013	0.0118	0.01451	0.864

AVERAGE STANDARD DEVIATION

FLOW=16.827L/S 0.440

VELOCITY=0.820M/S 0.0038

Y/D=0.395 0.0072

KS= 2.875MM 0.1838

MANNING N=0.01439 0.000137

FROUD NO =0.869 0.0072

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO	FRIC FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
23.390	0.695	158.00	143.12	0.470	73.131	167787	0.036	2.348	0.0080	0.01392	0.867
24.770	0.702	153.00	148.47	0.487	74.926	173811	0.037	2.442	0.0081	0.01401	0.858
26.310	0.724	156.00	151.93	0.498	76.050	181734	0.036	2.136	0.0070	0.01371	0.865
26.310	0.697	160.00	156.55	0.514	77.498	178287	0.038	2.935	0.0095	0.01442	0.832
26.870	0.703	160.00	158.11	0.519	77.973	180924	0.038	2.886	0.0092	0.01438	0.834
26.310	0.711	169.00	154.11	0.506	76.740	180090	0.036	2.481	0.0081	0.01408	0.851

AVERAGE STANDARD DEVIATION

FLOW=25.650L/S 1.317

VELOCITY=0.708M/S 0.0107

Y/D=0.499 0.0182

KS= 2.535MM 0.3059

MANNING N=0.01406 0.000270

FROUD NO.=0.852 0.0182

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO.
14.350	0.549	118.00	118.17	0.388	83.794	117183	0.051	5.347	0.0210	0.01609	0.581
14.030	0.554	114.00	115.43	0.379	82.672	118151	0.049	4.775	0.0180	0.01575	0.604
12.760	0.560	114.00	106.77	0.350	59.016	110558	0.046	3.588	0.0152	0.01497	0.639

AVERAGE STANDARD DEVIATION

FLOW=13.713L/S 0.841

VELOCITY=0.554M/S 0.0055

Y/D=0.372 0.0155

KS= 4.570MM 0.8974

MANNING N=0.01560 0.000572

FROUD NO.=0.611 0.0247

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL DEPTH	Y/D	CAL R	REY NO.	FRIC. FAC	KS(MM)	KS/4R	MANNING N	FROUD NO
7.060	0.425	75.00	84.83	0.279	49.003	71614	0.055	7.525	0.0384	0.01743	0.550
6.530	0.435	80.00	79.87	0.262	46.523	69582	0.059	5.651	0.0304	0.01646	0.582
7.130	0.434	79.00	84.26	0.276	48.676	72845	0.052	6.672	0.0343	0.01699	0.564

AVERAGE STANDARD DEVIATION

FLOW= 6.840L/S 0.271

VELOCITY=0.431M/S 0.0055

Y/D=0.272 0.0080

KS= 6.616MM 0.8363

MANNING N=0.01596 0.000491

FROUD NO.=0.565 0.0161

Q(L/S)	V(M/S)	MEAS. DEPTH	CAL. DEPTH	Y/D	CAL. R	REY. NO	FRIC. FAC.	KS(MM)	KS/AR	MANNING N	FROUD NO
5,220	0,282	80,00	77,83	0,256	45,680	57705	0,064	11,556	0,0586	0,01948	0,481
4,620	0,269	72,00	66,85	0,220	39,829	54358	0,062	5,726	0,0288	0,01862	0,573
6,010	0,290	88,00	79,64	0,261	46,363	62264	0,072	9,234	0,0498	0,01831	0,533

AVERAGE	STANDARD DEVIATION
FLOW= 5,287L/S	0,646
VELOCITY=0,280M/S	0,0155
Y/D=0,246	0,0226
KS= 8,972MM	3,1241
MANNING N=0,01814	0,001446
FROUD NO.=0,529	0,0410