Hydraulics Research
Wallingford

SELF-CLEANSING CONDITIONS FOR SEWERS CARRYING SEDIMENT

Summary Report

Report SR 222
January 1990

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This report describes work funded by the Department of the Environment under Research Contract No DGR PECD 7/6/82. It is published on behalf of the Department of the Environment, but any opinions expressed in this report are not necessarily those of the funding Department. The work was carried out in the River Engineering Department of Hydraulics Research, Wallingford headed by Dr W R White. The nominated project officers were Dr R P Thorogood for DoE and Dr W R White for HR.

ABSTRACT

An experimental study was made of the factors governing the deposition of non-cohesive sediment in a 300mm diameter concrete pipe using 0.72mm sand. Results were compared with previous HR data for the limit of deposition in 158mm and 77mm diameter smooth pipes. This indicated that limiting sediment concentrations in the concrete pipe were approximately half those expected in a smooth pipe of equal diameter. A formula was developed which can be used to estimate minimum velocities needed to prevent deposition of non-cohesive sediments in sewers.

Tests were also carried out with small depths of sediment deposition. These showed that a mean sediment depth of 1% of the pipe diameter enables a flow to transport significantly more sediment than at the limit of deposition with effectively no increase in head loss. Self-cleansing sewers designed for a 1% sediment depth could therefore be laid at flatter minimum gradients than those designed according to a "no-deposit" criterion.

Full details of the study are given in Hydraulics Research Report SR 221.

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1 INTRODUCTION

It has long been recognised that sediment deposits in sewers cause loss of flow capacity and can lead to surcharging and sometimes surface flooding. The problems were often considered to be localised and were usually dealt with by means of routine maintenance. However, two recent developments have demonstrated that the adverse effects of sediments in sewers are more serious than previously believed.

Firstly, the increased use of closed-circuit television equipment has shown that large lengths of sewerage systems contain significant deposits. A survey carried out for a CIRIA (1987) research project suggested that up to 25,000km of sewers and drains in the UK may be affected. Even though many such deposits may not be large enough to cause regular surcharging or flooding, they will still reduce the maximum flow capacity of a system and prevent it coping with the flood event for which it was designed.

The second development is the greater emphasis now placed on environmental aspects such as water quality. Stormwater sewerage systems, either separate or combined, are responsible for a significant proportion of the pollution that enters estuaries, rivers and watercourses, particularly in urban areas. Research has shown that many of the pollutants such as those responsible for the biological and chemical oxygen demand become closely associated with the sediment particles in sewers. Thus, sediments discharged directly from separate storm water sewers or from storm sewer overflows in combined systems will cause pollution in the receiving waters. In order to be able to study methods of improving water quality, it is therefore important to understand how sediment is transported through a sewerage system. The build-up of deposits near storm sewer overflows can also cause them to operate more frequently than necessary and thereby produce additional pollution.

Experimental research on sediment movement in sewers has been carried out at Hydraulics Research (HR) since 1975, under two studies funded by the Department of the Environment (DoE). The first study between 1975 and 1982 was concerned principally with developing an improved criterion for the design of self-cleansing sewers. Experiments were made using 77mm and 158mm diameter smooth plastic pipes, and showed how the flow velocity needed to prevent the formation of deposits depended on factors such as the sediment concentration, particle size and pipe diameter. Results of this study were described in reports by May (1975, 1982).

The second study, which is summarised in this report, forms part of the River Basin Management (RBM) programme. This is a co-ordinated programme of research into the effects of sewers on rivers, and covers field work, laboratory studies and the development of computational models. Individual projects are being carried out by the Water Research Centre (WRc), universities and HR, with funding provided by the Regional Water Authorities (and their successor organisations), the Science and Engineering Research Council and the Construction Industry Directorate of DoE.

A major component of the programme is MOSQITO, a computational water quality model for sewers, which is being developed at HR (with DoE funding) for use by the UK water industry. In order to be able to predict variations in water quality in sewers, it is necessary to determine rates of sediment deposition and erosion. The experimental study on sediment movement described in this report therefore has two functions : it extends the scope of the 1975-1982 work on self-cleansing sewers and secondly provides information necessary for the development of MOSQITO.

2 SCOPE OF STUDY

The principal objective of the present study is to aid the development of improved guidelines for the design of self-cleansing sewers carrying sediment.

Current practice for the design of self-cleansing sewers is to ensure that either the flow velocity or the shear stress produced by the flow exceeds a certain limiting value. Typical minimum values are in the range 0.75m/s to 1.0m/s for velocity and 1 N/m^2 to 4 N/m^2 for shear stress. Such limits are usually linked with a requirement that they be achieved at a given depth of flow (eg with the pipe half full) or with a given frequency (eg once a day on average for a combined sewer). These conditions lead to values of minimum gradient below which gravity sewers should not be laid if they are intended to be self-cleansing. A survey of various guidelines for self-cleansing sewers is contained in Appendix G of CIRIA (1987).

Recent laboratory studies, including the work carried out at HR under the first DoE contract, showed that self-cleansing conditions cannot be defined simply in terms of a fixed value of velocity or shear stress but need to take account of the rate of sediment entering the system, the size and density of the sediment, and the diameter of the pipe. Various formulae which include these extra factors have been developed, but they were mostly based on experiments carried out with non-cohesive sediments in smooth pipes of small diameter. Sediments in separate storm water sewers usually remain non-cohesive, but in combined systems they may become coated with biological slimes and greases. Crabtree (1988) classified sewer sediments into five broad categories. Type A material corresponds to the coarser sediment which forms bed deposits in combined sewers; analysis showed that it typically consists of granular sand and gravel with an organic content of about 10%. Rheological tests carried out by Williams et al (1989) on four Type A samples from sewers in Cardiff indicated that the material was cohesive, so results from laboratory studies with non-cohesive sediments may need to be applied with caution to combined systems. However, until the behaviour of non-cohesive sediments is understood properly, it will be difficult to take correct account of cohesive effects.

As mentioned, most studies on self-cleansing conditions have been carried out with smooth pipes of small diameter. Unfortunately, the resulting formulae give widely-differing predictions when extrapolated to pipes significantly larger than those originally tested. The first part of the new study described in this report was, therefore, designed to investigate self-cleansing conditions in concrete pipes of 300mm and 450mm diameter, which are more typical of those used in many sewerage systems. The results are compared with data and equations from previous studies in order to identify more accurately the effects of pipe size and texture.

Although earlier studies have disagreed on the precise flow conditions needed to prevent sediment deposition, most predict that the required flow velocity increases with increasing pipe size. The implication is that the minimum gradients of large sewers (eg having diameters > 0.5m) should be steeper than those specified at present. A change in design guidelines based on recent research could, therefore, significantly increase the costs of new sewerage schemes by requiring pipes to be laid at greater depths; more pumping would also be needed. However, there is a possibility that the criterion usually adopted for "self-cleansing" conditions - namely, no formation of stationary sediment deposits - may be more severe than is actually necessary. If small depths of sediment deposit are permitted under design conditions, it may be possible to reduce the values of minimum flow velocity; this in turn would allow the use of somewhat flatter pipe gradients. It can also be argued that the criterion of no sediment deposition is a fiction because some sediment will always remain in a sewer after a storm and will usually form a stationary deposit until the next storm occurs.

Whether or not a relaxation in the self-cleansing criterion is justified depends on the answers to two questions. Firstly, if sediment deposits are allowed to form, will they remain small or will they grow in size until ultimately the pipe surcharges or becomes blocked? Secondly, will the additional hydraulic resistance due to the deposits be large enough to reduce the hydraulic capacity of the sewer significantly? The second part of the study summarised in this report was carried out to answer these questions and provide guidance on suitable design criteria for sewers carrying sediment.

3 EXPERIMENTS AND ANALYSIS

The experiments were carried out using a 20m length of spun concrete pipe installed in an existing 2.44m wide tilting flume. The mean internal diameter of the pipe was 298.8mm with a standard deviation of 2.9mm.

Adding sediment to the flow at the upstream end of the test pipe and collecting it at the downstream end would have required large quantities of dry sediment and would have made it difficult to continue tests for long periods. Separate systems were therefore provided for recirculating the water and the wet sediment. The sediment was collected in a hopper at the downstream end of the test pipe and returned, with a small proportion of the water, to the upstream end by means of a slurry pump.

To permit continuous tests, it was necessary to be able to measure the rate of sediment transport without disturbing conditions in the test pipe. This was made possible by the development of a new infra-red device for measuring sediment concentration in the slurry pipe. The instrument consisted of an infra-red source and a detector positioned vertically and diametrically opposite each other on the outside of a section of transparent pipe. Sand particles interrupting the light beam reduced its strength and gave an electrical signal which was found to be linearly proportional to sediment concentration over most of its range. The sensitivity of the instrument could be reduced by increasing the flow velocity in the slurry pipe, or vice versa provided the velocity was sufficient to prevent deposition. The output from the detector was connected to a counter so that the signal could be averaged over any period from 1 second to 9999 seconds.

Preliminary tests were carried out to determine the hydraulic resistance of the 299mm concrete pipe when carrying clear water. A total of 94 measurements was obtained with depths of flow varying between 1/4-full and pipe-full. The overall value of hydraulic roughness in the Colebrook-White formula was found to be k_s = 0.15mm. The results showed that the roughness increased with flow depth up to a maximum of k_s = 0.30mm when the pipe was flowing 3/4-full, and then decreased to k_s = 0.12mm when the pipe was flowing full.

Tests to determine conditions at the limit of sediment deposition were carried out with sand having a mean size of 0.72mm and a specific gravity of 2.62. Below the limit, the flow transported the sediment along the invert of the pipe as separate particles (flume traction); beyond the limit, the sediment formed either slow-moving dunes separated by lengths of clear pipe (at lower flow velocities) or a continuous deposited bed (at higher flow velocities). A total of 48 tests was made with average flow depths equal to 0.37, 0.51, 0.74 and 1.0 times the pipe diameter; flow velocities varied between 0.5m/s and 1.5m/s and volumetric sediment concentrations between 0.3ppm and 440ppm.

The new data for the 299mm diameter concrete pipe were compared with previous HR results for smooth 77mm and 158mm diameter pipes (see May (1982)) and with predictions given by formulae due to Macke (1982), May (1982), Mayerle (1988) and Mayerle & Nalluri (1989). The comparisons showed that limiting sediment concentrations in the concrete pipe were approximately half those expected in a smooth pipe of the same diameter. The following best-fit equation to all the HR data for the 77mm, 158mm and 299mm pipes was established

$$C_v = 2.11\mathrm{x}10^{-2}\ (y/D)^{0.36}\ (A/D^2)^{-1}\ (d/R)^{0.6}$$

$$.[1 - (V_t/V_L)]^4\ \left[\frac{V_L^2}{g\ (s-1)\ D}\right]^{3/2} \qquad (1)$$

where C_v = volumetric sediment concentration

y = flow depth

D = pipe diameter

A = cross-sectional area of flow

d = mean sediment size

R = hydraulic radius of flow

V_t = mean flow velocity in pipe at threshold of movement for individual sediment particle

V_L = mean flow velocity in pipe at limit of deposition when transporting sediment concentration C_v

g = acceleration due to gravity

s = specific gravity of sediment

For smooth pipes, the threshold velocity should be calculated from the following equation due to Novak & Nalluri (1975)

$$V_{ts} = 0.61 \; [g \; (s-1) \; d]^{1/2} \; (d/R)^{-0.27} \qquad (2)$$

An equivalent formula is not available for non-smooth commercial pipes. In the present study, it was found that the effective value of the threshold velocity in the spun concrete pipe was

$$V_t = 4 \; V_{ts}/3 \qquad (3)$$

where V_{ts} is calculated from Equation (2). Further work is needed to establish a general relationship for threshold velocities in commercial pipes. Equation (1) is not valid if the sediment is being transported in suspension at the limit of deposition, and it is therefore recommended that it should not be applied for sand sizes less than about 0.4mm to 0.5mm.

A separate series of 44 tests was carried out in the 299mm diameter concrete pipe to investigate the effect of small amounts of sediment deposition on the rate of sediment transport and on the hydraulic resistance. In most cases, the 0.72mm sand formed a series of separated dunes which moved slowly along the pipe. In each test, the volume of deposited material was measured, and a mean sediment depth, y_s, calculated assuming this volume to be distributed uniformly along the pipe. Measurements were made with the pipe flowing full and half-full, and with values of y_s/D mainly in the range 0.1% to 4%.

Comparison with the previous results showed that rates of sediment transport with small amounts of deposition were significantly higher than those measured at the limit of deposition. As an example, going from the limit of deposition to a bed depth of $y_s/D = 1\%$ doubled the sediment concentration when the 299mm pipe was flowing half-full at a velocity of 1.2m/s; at a flow velocity of 0.6m/s, the concentration was increased by a factor of 7. The measurements of head loss showed that deposition began to produce a measurable increase in resistance when the sediment depth reached $y_s/D = 3\%$ (giving $k_s = 0.54$mm, compared with $k_s = 0.15$mm for clear water). However, below a value of about $y_s/D = 1\%$, the effect on resistance was not significant.

These results suggest that a mean deposited depth of $y_s/D = 1\%$ could provide a suitable criterion for the design of self-cleansing sewers. This would allow minimum velocities and gradients for pipes to be somewhat reduced compared with those required by the "no-deposit" criterion. An equation for reliably predicting sediment transport in pipes with deposited beds is not yet available, but an interim solution might be to assume an approximate doubling of sediment concentration relative to that at the limit of deposition. The minimum flow velocity V_m required to limit sediment depths to 1% of pipe diameter could then be estimated from the following modified version of Equation (1)

$$C_v = 4.0\times10^{-2}\,(y/D)^{0.36}\,(A/D^2)^{-1}\,(d/R)^{0.6}\,.\,[1-(V_t/V_m)]^4\left[\frac{V_m^2}{g\,(s-1)\,D}\right]^{3/2} \qquad (4)$$

These suggestions are based on a limited number of tests, and should be reviewed when more experimental data become available.

A full description of the experimental measurements and the data analysis is given in Hydraulics Research Report SR 221 (see May et al (1989)).

4 CONCLUSIONS

(1) Tests have been carried out to study the limit of deposition in a 299mm diameter concrete pipe using 0.72mm sand, flow velocities between 0.5m/s and 1.5m/s, volumetric sediment concentrations between 0.3 ppm and 440 ppm, and proportional depths of flow between 3/8-full and pipe-full.

(2) The results were compared with previous HR data for 158mm and 77mm diameter smooth pipes and with the predictions of several equations for the limit of deposition. For a given velocity and depth of flow, the limiting sediment concentration in the concrete pipe was found to be typically half that expected in a smooth pipe of the same diameter.

(3) The best-fit equation to the HR data for the three pipe sizes is given by Equation (1). For smooth pipes, the threshold velocity should be calculated from Equation (2). For the 299mm concrete pipe, the threshold velocity was found to be 33% higher than the equivalent smooth-pipe value. Equation (1) should not be applied for sand sizes less than about 0.4mm to 0.5mm.

(4) Tests were also carried out in the 299mm concrete pipe to measure hydraulic resistance and rates of sediment transport for various small depths of bed deposit. The results showed that the sediment transporting capacity of the flow increased significantly as the mean sediment depth increased. The effect of the deposits on hydraulic resistance did not become significant

until the mean sediment depth reached about 3% of the pipe diameter.

(5) Based on these findings, it is suggested that a mean deposit depth equal to 1% of the pipe diameter could provide a satisfactory criterion for the design of "self-cleansing" sewers. It would allow a worthwhile reduction in minimum velocities and gradients, particularly for larger pipes, compared with the previous "no-deposit" criterion, and would not result in any significant reduction in hydraulic capacity. Equation (4) provides a possible method of determining minimum velocities for a 1% deposit depth, and should give conservative results.

(6) These suggestions should be reviewed as more experimental data become available on sediment transport in pipes with deposited beds.

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