First evaluation of new design concepts for self-cleaning distribution networks

M. van den Boomen, A. van Mazijk and R. H. S. Beuken

ABSTRACT

Ten years of joint research on the nature and causes of discoloured water has resulted in guidelines for the implementation of self-cleaning distribution networks. These were first introduced in the Netherlands in 1999. The self-cleaning concept has been monitored right from its beginnings. Based on this evaluation, significant observations were found on the contribution of changes in flow velocity to the self-cleaning characteristics of pipes. These observations will be further investigated and will form the basis for the concept of dynamic sediment transportation modelling.

Key words | discoloration, self cleaning distribution networks, sediment transport

M. van den Boomen (corresponding author)
R. H. S. Beuken
Kiwa Water Research,
Groningenhaven 7, PO Box 1072,
3430 BB Nieuwegein,
The Netherlands
Tel: +31 (0)30 6069 756
Fax: +31 (0)30 6061 165
E-mail: *martine.van.den.boomen@kiwa.nl*
E-mail: *ralph.beuken@kiwa.nl*

A. van Mazijk
Delft University of Technology,
Faculty of Civil Engineering & Geosciences,
Section of Water Management,
Stevinweg 1, PO Box 5048,
2600 GA Delft,
The Netherlands
Tel: +31 (0)15 278 54 77
Fax: +31 (0)15 278 59 15
E-mail: *a.van.mazijk@CiTG.TUDelft.nl*

INTRODUCTION

Discoloured water is a generally recognised water quality problem in water distribution systems. In 1990 about 1,200 water quality complaints were registered yearly by a representative Dutch water utility which serves approximately 530,000 connections. Today this number has decreased to approximately 250 per year. The decrease of complaints is ascribed to improved water quality management and the results of 10 years of joint research on the nature and causes of discoloured water. Discoloured water, caused by long-term accumulation and formation of sediment in drinking water networks, can basically be prevented in three stages by:

- sufficient water treatment at the plant;
- removing sediment adequately through pipe flushing;
- creating hydraulic conditions which prevent long-term settling of sediment.

A great deal of research has been conducted internationally on the first two measures to prevent discoloured water (van den Hoven & Vreeburg 1992; van den Hoven *et al.* 1994). This research has delivered satisfactory results and practices for the Netherlands (Schaap *et al.* 2002; AWWA 2002). This article will focus on the third measure. Hydraulic processes are a key factor to consider when dealing with discoloured water problems; it is a factor that has received little attention until quite recently. Creating unfavourable conditions for sediment accumulation and growth by designing distribution networks with self-cleaning characteristics can prevent discoloration.

CONCEPT OF SELF-CLEANING DISTRIBUTION NETWORKS

The basic difference compared with the traditional way of designing distribution networks is that the self-cleaning networks are designed as branched systems instead of looped systems (Figure 1). In addition, the diameters of the self-cleaning networks are designed on a once a day flow velocity of 0.4 m s^{-1} based on household peak demand. The three upper oval shapes in Figure 1b

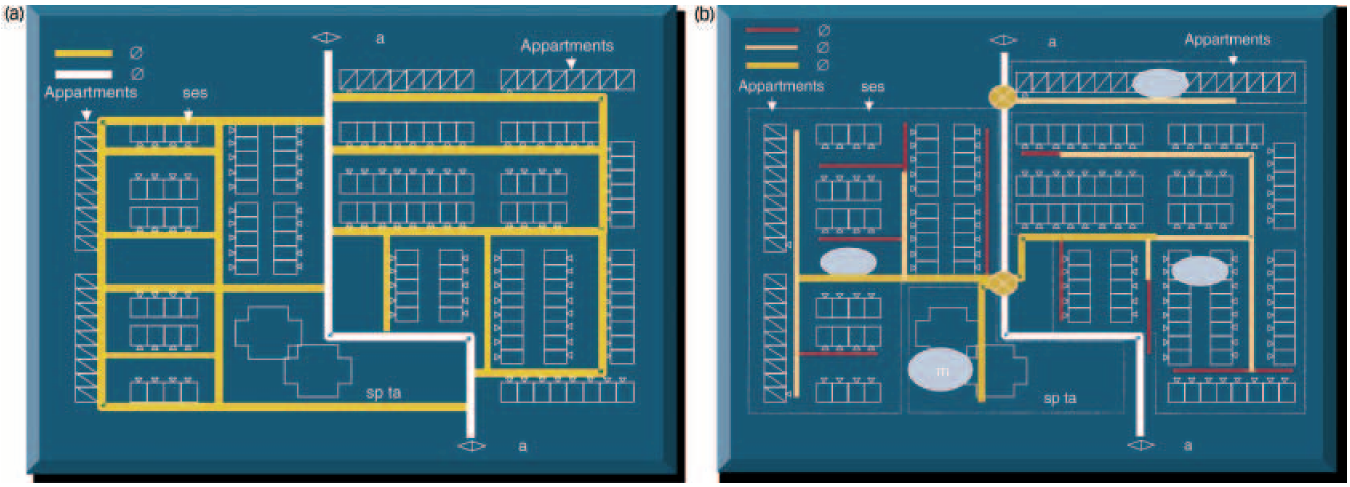

Figure 1 | Concepts of distribution networks (a) looped system; (b) self-cleaning system.

represent the total number of households connected to the branches. The size of branched sections will generally range between 10 and 120 households. The pipe diameter depends on the number of house connections downstream and consequently the diameter will become smaller with a decrease in the number of house connections. The oval shape at the bottom of Figure 1b represents the demanded capacity at a hospital. In this example the capacity is determined by fire flow requirements.

The advantages of self-cleaning distribution networks are:

- no stagnant water;
- short residence times;
- improved water quality;
- a proven reduction of up to 30% on material costs.

Possible drawbacks such as the provision of firefighting water and security of supply in case of a pipe burst have been successfully eliminated. The new networks are designed in close co-operation with fire brigades in order to ensure sufficient firefighting water. Just as looped systems, the branched networks fully comply with the Dutch standards of supply guarantee.

A risk assessment has shown that the networks are resistant to significant changes in demand without adversely affecting the required working pressures. Manufacturers have developed the necessary appurtenances for installing hydrants on pipes with smaller diameters.

In order to evaluate the self-cleaning concept, the performances of the networks since they were first installed in 1999 have been monitored as part of the joint research programme. The monitoring and evaluation have been carried out on three levels:

A pipe test rig was built to evaluate the constant self-cleaning velocity of 0.4 m s^{-1} under laboratory conditions.

Turbidity and volume passage were measured in self-cleaning branched networks to evaluate the design guidelines and to assess the self-cleaning potential of those networks.

Based on the results of the laboratory pipe tests and the field measurements, theoretical research was carried out. This has led to new conclusions on the causes of the self-cleaning effects in distribution networks. The influence of the dynamics of demand has proved to be significant.

LABORATORY EXPERIMENTS

A pipe test rig was built to evaluate the constant self-cleaning velocity of 0.4 m s^{-1} and to see whether this

Table 1 | Sediment samples used in the laboratory rig test

Material	Grain size (μm)	Density (kg m^{-3})	Modelling of
Sand	180–250	2,650	Sand
Iron oxide	45–90	3,140	Iron oxide
Iron oxide	180–250	3,140	Iron oxide
Flour	63–90	1,320	Organic matter
Flour	150–180	1,320	Organic matter

value had to be adjusted. The value of 0.4 m s^{-1} was originally based on practical experience and corresponds to the range of values found within the Shields diagram (Shields 1936) for re-suspension of non-cohesive material.

In the period 1993 to 1995, many sediment samples were taken from distribution networks in the Netherlands and their compositions were analysed. The results of the analyses were used in the development of the pipe test rig, and drinking water sediment was modelled using the standardised fractions shown in Table 1.

The reason for using standardised fractions and not real samples from the distribution networks is based on the large variations of fractions that were found in real sample compositions. What real samples have in common is the existence of certain fractions. Standardised fractions have the advantages of being uniform, with an accurately reproducible composition.

Standardised fractions were inserted into a full range of pipe diameters under a wide range of steady flow conditions. Once a sediment fraction was inserted, the flow velocity of the water was very slowly increased until flow conditions were approximately steady. Using a video camera, the flow velocities at which the first movements of particles occurred and at which all particles were moving were recorded.

The results, presented in Figure 2, show that all sediment fractions were transported at flow velocities within the range of 0.07–0.29 m s^{-1} under steady flow conditions. Another observation during the tests was the formation of sediment islands and dunes on the bottom of

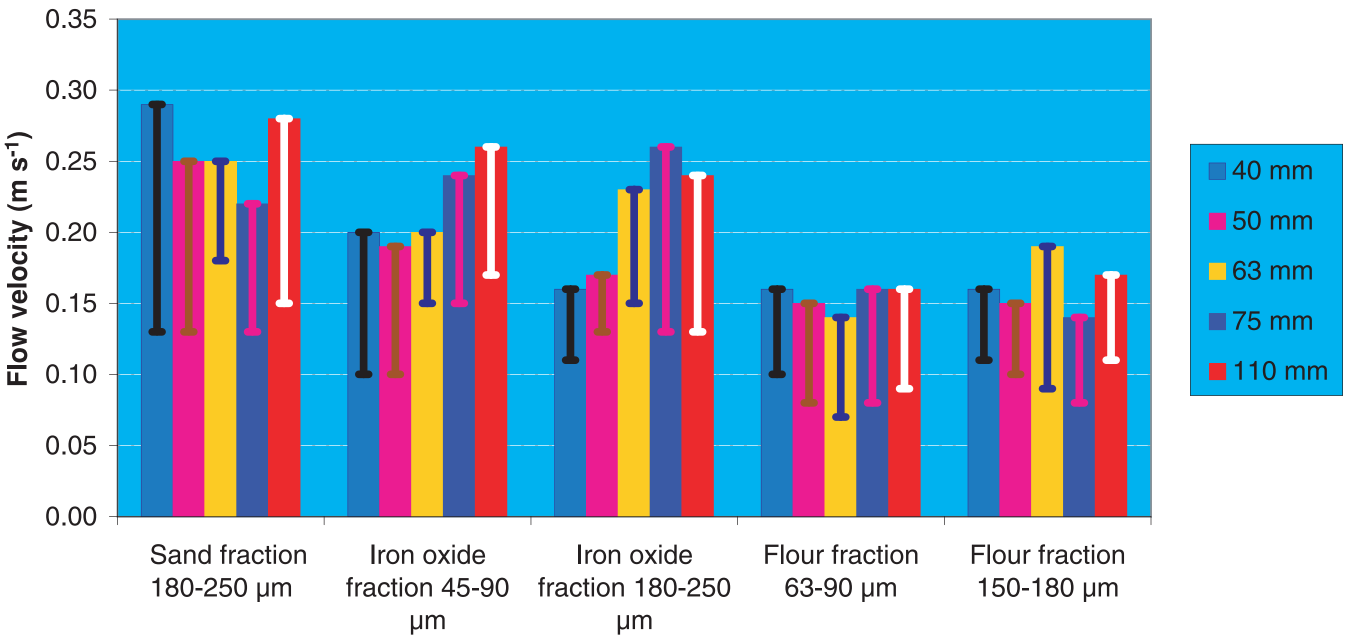

Figure 2 | First and last movement of particles of standardised sediment fractions under steady flow conditions and in different PVC pipe diameters.

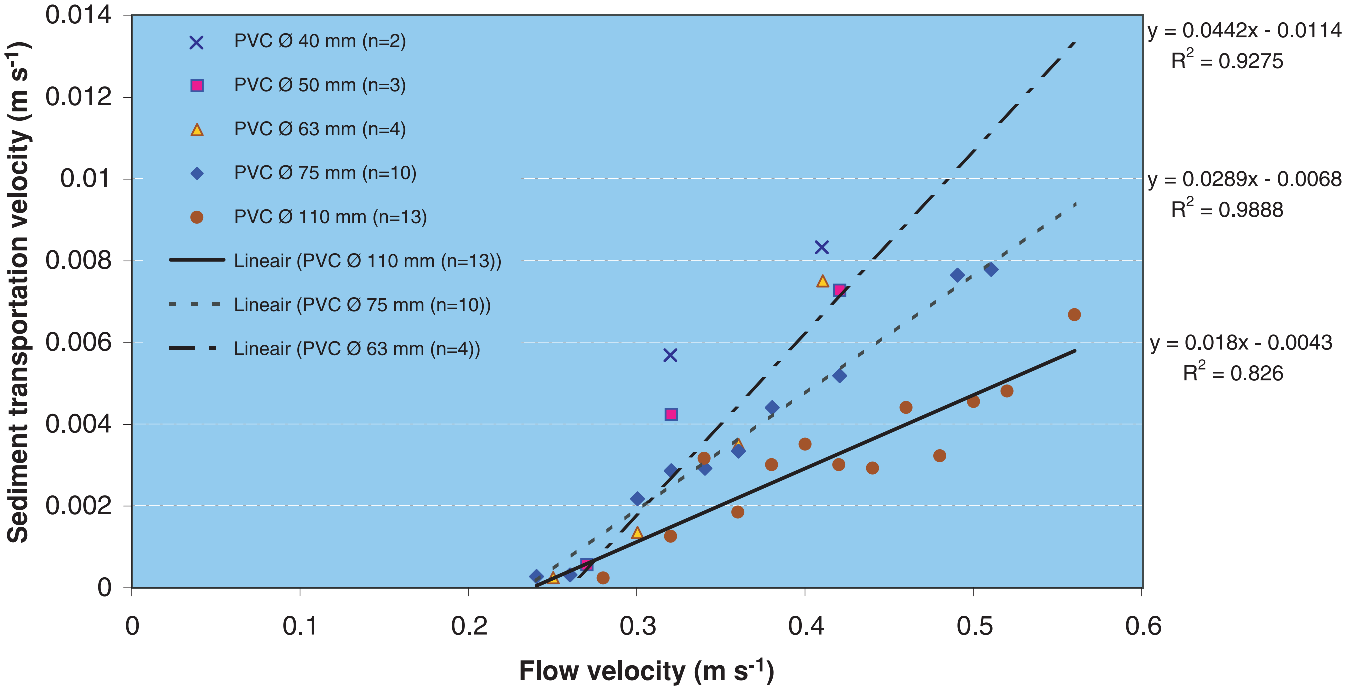

Figure 3 | Sediment transportation velocities in relation to flow velocities and pipe diameter.

the pipes. The sediment particles group themselves in a form with the lowest resistance against the shear stress of the flow. It was noticed that an individual particle starts moving at lower flow velocities than a group of non-cohesive particles. Consequently accumulated sediment requires larger flow velocities than the individual particles to be removed.

The video shots also showed that with flow velocities less than 0.35 m s^{-1}, the sediment fractions were mainly transported along the bottom half of the pipe. Between the heavier and lighter sediment fractions no difference was noticed in the velocity of first particle movement in relation to the pipe diameter (Figure 3). However, as soon as all sediment particles were moving, it was clearly noticed that in smaller pipe diameters lower flow velocities were required for sediment transportation than in larger pipe diameters (Figure 3).

Although these results indicate that a flow velocity of 0.3 m s^{-1} could be sufficient for self-cleaning, the value of 0.4 m s^{-1} is recommended at this moment in time. The reason for this recommendation is based on the observation that the sediment will be transported mainly along the bottom half of the pipes. Obstacles in pipes or slopes will form a risk factor and will require larger flow velocities than 0.3 m s^{-1}.

Further research is strongly recommended. The experiments conducted in the pipe test rig were carried out with non-cohesive material during short experiment times. In reality, sediment will coagulate in time, which will affect its behaviour under different hydraulic conditions. In addition, the effect of the accumulation of sediment near obstacles and upward slopes needs to be investigated. Another important factor concerns the steady flow conditions, which are rarely found in distribution networks, as will be illustrated in the next paragraph.

FIELD MEASUREMENTS

The field measurements concern the turbidity and the flow in a recently constructed self-cleaning distribution network of a Dutch Water Company (PWN). In a single

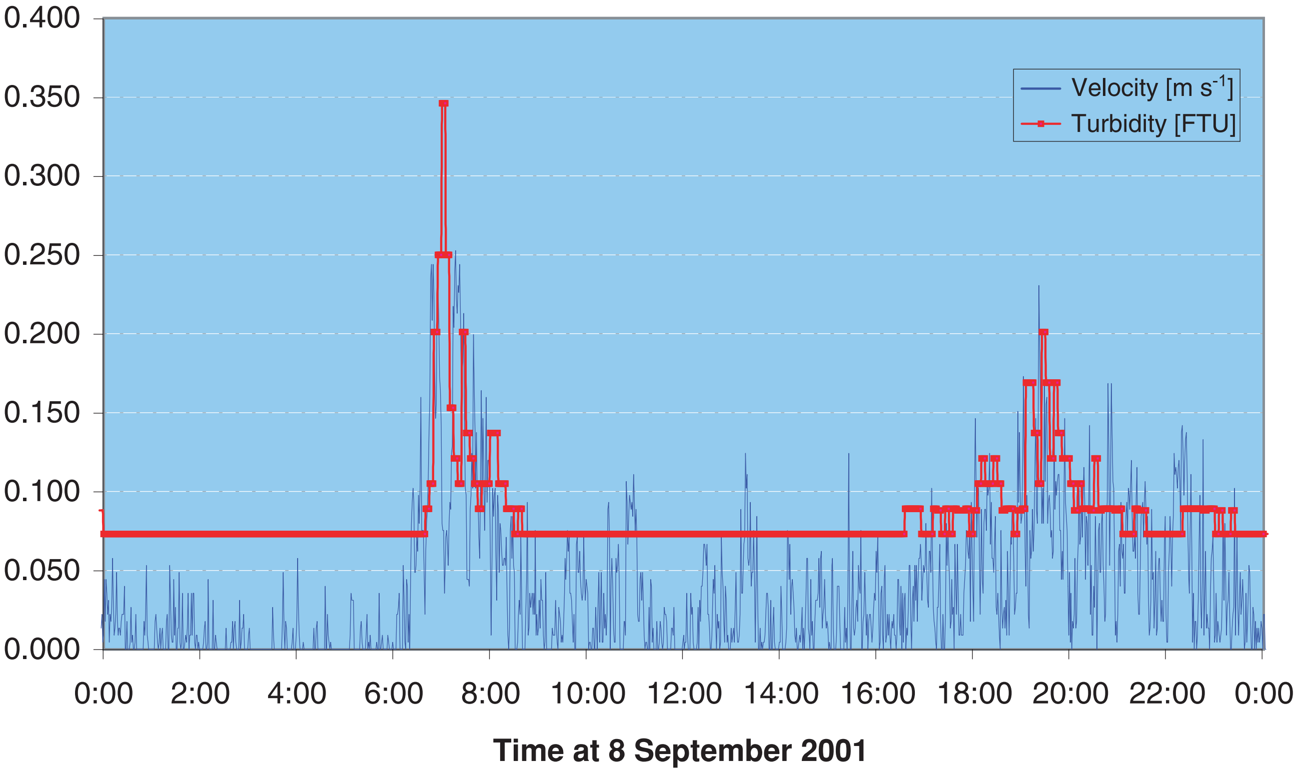

Figure 4 | Turbidity and flow measurements in a branch of a self-cleaning distribution network with a downstream water consumption of 46 households.

branch of the self-cleaning network, three monitoring points were installed with a downstream water consumption of 7, 19 and 46 house connections. At these monitoring points, the turbidity was measured continuously with plotted values every five minutes and the flow was measured digitally with plotted values of passed volume per 2 min. These kinds of detailed measurement are thought to be unique to the Netherlands.

Figure 4 shows a representative view of an average day with a downstream consumption of 46 houses. The passed volume per 2 min is translated into the flow velocity in metres per second. It is assumed that the estimation error of this experiment is acceptable considering a downstream demand of 46 households. Additional, more accurate measurement will be carried out in the near future as part of the joint research programme.

Some interesting observations can be deduced from Figure 4.

1. A velocity of 0.4 m s^{-1} is not reached during the day.
2. The demand pattern has no characteristics of steady flow. The demand pattern is composed of strong fluctuations in velocities.
3. Turbidity only follows the morning and evening demand peaks.
4. The demand peaks during 20:30 and 22:30 do not result in a change of turbidity.

Based on the first observation, it is noticed that the criterion for self-cleaning, a velocity of 0.4 m s^{-1}, is not achieved. Two reasons exist: (1) the design formula used to calculate household peak demand is believed to over estimate real household peak demand; and (2) water companies still tend to apply larger diameters than the guidelines advise. However, based on observations 3 and 4, there are some indications that the pipe may be self-cleaning after all. Based on the pattern that turbidity follows flow peaks with a certain height and density, it would be expected that, if sediment was present, a turbidity increase would have also been noticed at 20:30

and 22:30, and not only during the morning and early evening peaks.

Analysis of the complete datasets reveals the same patterns between changes of flow velocities and turbidity. Therefore a strong indication exists that, on this level of network design, the fluctuations of the flow velocity cause sediment removal and not the flow velocity itself, i.e. the steady flow. Historically, all computations for network design are based on steady flow conditions. To further underpin the field observations, a theoretical research study was carried out.

THEORETICAL RESEARCH

Field measurements indicated that the dynamics of demand pattern have a strong influence on turbidity or sediment removal. The objective of the theoretical research was to assess the contribution of changes in flow velocity to the self-cleaning characteristics of drinking water pipes.

Until now, knowledge about the real forces on and the behaviour of non-cohesive drinking water sediment has been very limited and case specific. A hypothesis was developed, which builds on what we do know: (1) the shear stress on the pipe wall, (2) the practical results and (3) the pipe test rig results, which shows that satisfactory cleaning results are obtained with a steady flow velocity of 0.4 m s^{-1}. The hypothesis defines the shear stress on a PVC 110 mm pipe wall at a steady flow velocity of 0.4 m s^{-1} to be at least equal to the critical shear stress required for re-suspension of drinking water sediment. Based on this hypothesis, the basic equation for transport of water under pressure, including the dynamic term, can be transformed into Equation 1.

$$\tau = -\rho \cdot R \cdot \left(\frac{\partial u}{\partial t} + g \cdot \frac{\partial \varphi}{\partial x} \right) \quad (1)$$

where:

τ = shear stress on the pipe wall (N m^{-2})
ρ = density of the water/sediment composition (≈1,000 kg m^{-3}) (kg m^{-3})
R = hydraulic radius ($= \frac{1}{4} \cdot D$) (m)
D = internal diameter of the pipe (m)
g = gravitation constant (m s^{-2})
u = average flow velocity of the water (m s^{-1})
φ = piezometric level (m)
$\partial u/\partial t$ = velocity gradient or 'the acceleration term' (m s^{-2})
$\partial \varphi/\partial x$ = energy gradient or 'the velocity term' (m m^{-1})

Equation 1 shows that the shear stress on the pipe wall, which hypothetically is set as a measure for the shear stress on the sediment, depends on the internal pipe diameter, the velocity term ($\partial \varphi/\partial x$) and the changes in flow velocity ($\partial u/\partial t$). The shear stress for re-suspension of sediment is calculated based on a constant flow velocity of 0.4 m s^{-1} as $\tau = 0.47$ N m^{-2}. With this boundary constraint, all relationships between velocity and changes in velocity can be calculated for different PVC pipe diameters from Equation 1 (van den Boomen & van Mazijk 2002). The results are presented in Figure 5.

Figure 5 shows for example, that a steady velocity of 0.10 m s^{-1} in a 110 mm PVC pipe needs an additional change of flow velocity of 0.0166 m s^{-2} to meet the criterion for self cleaning ($\tau = 0.47$ N m^{-2}). This change of flow velocity would already occur when the flow accelerates from 0.10 m s^{-1} to 0.12 m s^{-1} in 1 sec. The theoretical approach fully supports the field observations. Even with low flow velocities, small changes of flow will be enough to ensure a self-cleaning effect. Looking back at the demand pattern of Figure 4 it is concluded that the changes in flow velocity at this level of network design are the dominant cause for sediment removal. This implies that with small constant velocities, the dynamic forces may not be ignored in water quality modelling and network design.

PULSE FLUSHING

The results of the theoretical research on the dynamics of flow have already been applied in the concept of pulse flushing. The same principle as for removing daily drinking water sediment from the network is valid for removing

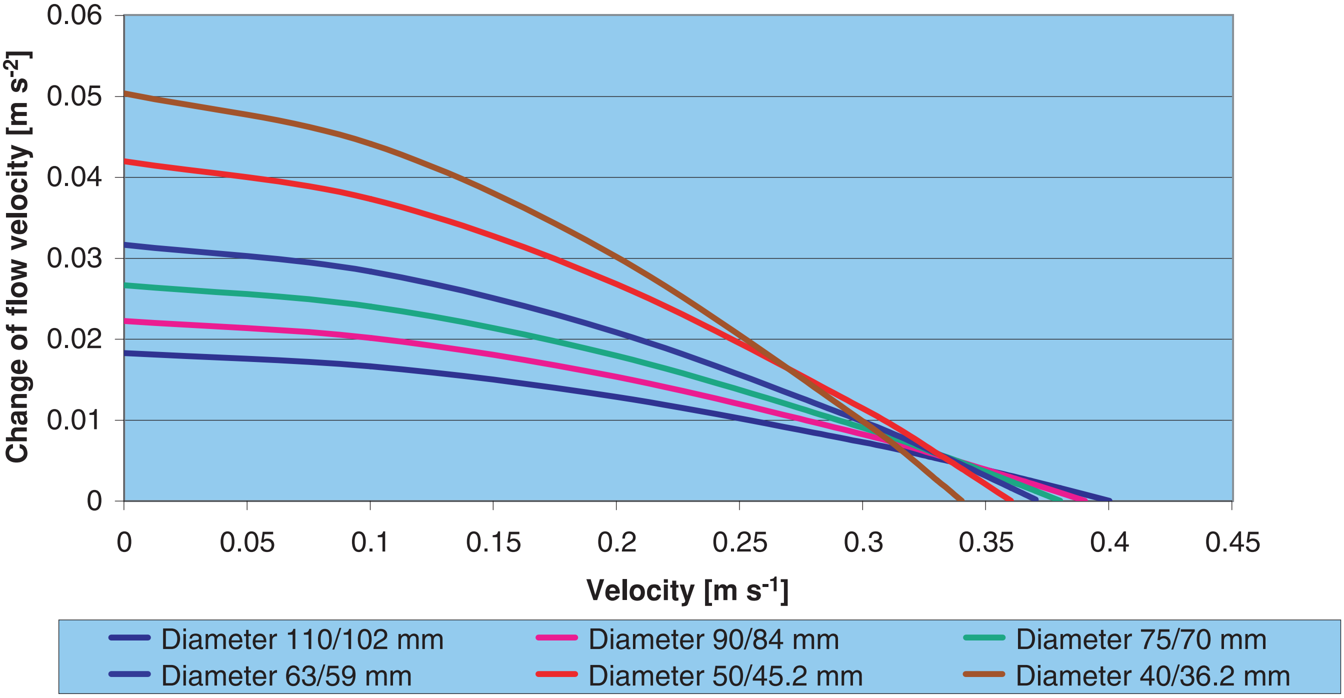

Figure 5 | Combinations of flow velocites and changes of flow velocites which reach a self-cleaning shear stress for PVC drinking water pipes.

accumulated sediment. From practical experience, satisfactory cleaning results are obtained with unidirectional flushing and a steady flushing velocity of 1.5 m s^{-1}. The calculated shear stress at a flow velocity of 1.5 m s^{-1} forms the starting point for the calculation of pulse patterns with steady final velocities less than 1.5 m s^{-1}. Generating a pulse pattern in the field allows water companies to flush pipes using a lower final velocity than the conventional flushing velocity practice. The technique of pulse flushing is applicable in those areas where the conventional flushing velocity cannot be met. The first test results are very promising and the technique should become more common practice in the future.

CONCLUSIONS AND PRACTICAL RESULTS

A first evaluation of the self-cleaning distribution concept resulted in the following conclusions:

- The guidelines presented in this paper for the design of self-cleaning distribution networks can achieve satisfactory results in practice.
- A constant flow velocity of 0.4 m s^{-1} is adequate to remove daily drinking water sediment from the network.
- Field measurements show that, in practice, velocities of 0.4 m s^{-1} will not occur in branched networks for two reasons: (1) the design formula for household peak demand overestimates the real demand; and (2) water companies tend to install larger diameter pipes than those recommended by the guidelines.
- Field measurements and theoretical research show that the self-cleaning characteristics of present networks are probably attributable to fluctuations in flow velocities.
- Theoretical research indicates that, although lower flow velocities occur in practice, the fluctuations in flow velocities result in a self-cleaning effect.

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- Dynamics of flow cannot be neglected when dealing with small flow velocity and sediment transportation.
- The results of this research are also applicable to pulse flushing.

FOLLOW UP

The findings of this research will be used to adjust the design criteria for self-cleaning distribution networks. A tool to facilitate the manual computations has been developed. New field measurements of flow per second and turbidity will be carried out in a branched network, designed on a self-cleaning velocity. These results will be used to determine actual changes of flow velocity so that the shear stress and turbidity can be linked and the self-cleaning characteristics can be quantified.

A second research path will lead to the dynamic modelling of sediment transportation for drinking water networks. Such a model will assist planning engineers with a self-cleaning design in branched networks and in looped networks, reveal hot spots for discoloration and support maintenance staff with just-in-time flushing signals.

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