The reliability of sediment transport predictions in sewers: influence of hydraulic and morphological uncertainties

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ABSTRACT

The paper is focussed on the concept of defining the "predictability" of sediment transport. Engineers are faced with a number of sediment transport formulas derived from different tests and described as suitable for application in sewers. Bed and suspended load formulas vary in form and performance, generally depending on the data sets that were used to calibrated them. As different sediment types have been tested no single, generally valid formula has been established so far. Formulas are distributed in the scientific literature and are often reported without the information necessary to define their range of potential applicability. Therefore, this paper along with analysing the formulas available, will also comment on the assumptions used in their development as well as the reliability of their underlying data to aid engineers in the selection of the most appropriate sediment transport formulae to correspond with the environment in which they are working.

Key words | in-sewer sediment transport, prediction, transport formula, shear stress

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NOMENCLATURE

A	cross-sectional flow area	g	acceleration due to gravity
B	water surface width	k_s	equivalent roughness height for bed
C_v	volumetric sediment concentration	R	hydraulic radius
D	water depth	R_b	hydraulic radius related to the bed
D_*	dimensionless particle size $= [(s-1)g/\nu^2]^{1/3}d_{50}$	S	water surface slope
d_x	sieve size for which x % of the sample by weight passes	s	specific gravity of sediment particles
E	erosion rate	U	cross-section mean flow velocity
F_b	mobility parameter based on total shear stress	u_*	bed grain shear velocity $= (\tau_b/\rho)^{0.5}$
F_g	grain mobility parameter in May's equations	W_b	bed width
F_s	effective mobility parameter in May's equations	q_b	bed load transport
G_{gr}	sediment transport parameter in Ackers and White equations	q_s	suspended load transport
		θ_b	non-dimensional bed shear stress $= \tau_b/(g(\rho_s - \rho)d_{50})$
F_{gr}	sediment mobility parameter in Ackers and White equations	Φ	non-dimensional total transport $= (q_b + q_s)/\rho_s[gd_{50}^3(\rho_s - \rho)/\rho]^{0.5}$
A_{gr}	F_{gr} at threshold of movement	λ_g	grain Darcy-Weissbach friction factor

λ_b	Darcy-Weissbach friction factor for bed
η	transport parameter in May's equations
ϕ	transition factor
ρ	density of water
ρ_s	sand density
τ_b	bed shear stress
τ_{cr}	critical shear stress
ν	kinematic viscosity of water
μ	bed form factor
$'$	related to grain
$''$	related to bed form

INTRODUCTION

Sediment transport in sewer pipes is extremely complex involving interactions between turbulent flows and rough and potentially mobile boundaries. The difficulty in mathematically describing the sediment and flow interaction in sewers is magnified by the wide variety of sediments that enter and form deposits in the inverts of sewer pipes, by the wide variation of flow conditions, ranging from sub- to supercritical, and in the unsteadiness of the flows as discharges vary during each day and especially during rain events.

Sediment deposition can have an impact on system performance by increasing hydraulic roughness thus reducing flow capacity and eroded sediment can be released to the environment via overflows. These reasons have encouraged the development and use of tools to model in-sewer sediment transport so as to assess their impact on flow capacity and water quality. Currently several prediction tools are available to predict the sediment movement in sewers. In fact, engineers can use a number of sediment transport formulas each derived from different laboratory and field tests, and described as suitable by software providers for application in sewers. Bed and suspended load formulas vary in performance, depending on the underlying assumptions and on the data sets that they are calibrated with, and the mathematical form of the relationships. Moreover, these formulas are often reported without the information necessary to objectively define their conditions of potential applicability.

The purpose of the paper is therefore to gather information on some of the existing sediment transport predictors used for in-sewer sediment transport prediction; along with providing advice on laboratory and field testing techniques so that users are able to assess the reliability of their underlying data. This advice will aid engineers in the selection of the most appropriate sediment transport formulae to correspond with the environment in which they are working.

SEDIMENT BEDS AND THEIR MOBILITY

It is widely acknowledged that the sewer sediment transport processes are complex in all phases: settling, sedimentation, erosion and transport. This is due to the nature of sewer systems, sediment inputs vary, both in terms of quality and quantity, and discharge rates also vary at both long and short time scales. (Ashley & Verbanck 1996; Ashley et al. 2004).

Commonly distinction is made between different modes of solid particle transport and their origin: bed load occurs close to the bed and consists mainly of relatively coarse particles. Finer particles, often largely organic in character, are transported in suspension as suspended load. Solids may originate from the bed as well as from boundary input from catchment surfaces. The mode of transport of solid particles in a sewer pipe can also be determined by the varying flow conditions during periods of dry and wet weather (Wotherspoon 1994). During storm events deposited sediment can be mobilised and move both in suspension and as bed load, during dry weather conditions the suspended matter can settle, to form a near-bed highly organic layer and can mix with existing granular fractions. The inclusion of such sediment can result in the development of cohesive strength in sewer deposit and thus change significantly its erosion behaviour (Tait et al. 2003a; Banasiak et al. 2005). The terms cohesive-like or partly cohesive sediments have been used to describe deposits that display a greater resistance to erosion than it would be anticipated in regard to purely granular deposits of similar grain size (Crabtree 1989).

MODELLING OF IN-SEWER SEDIMENT TRANSPORT

As different sediment deposits can be encountered in real sewer pipes, ranging from non-cohesive to partially cohesive, and with a wide range of grain sizes, different modelling approaches have been considered. Transport of real sewer sediments and also artificial surrogates, which have been selected to provide a wide range of suitable physical particle characteristics, have been investigated in a number of laboratory and field studies. The earliest laboratory studies used homogeneous, single sized granular sediments, following experience from fluvial studies (e.g. Alvarez-Hermandez 1990; Perruisquia 1991; Kleijwegt 1992), but attempted to quantify the possible effects of circular flow sections. The next generation of laboratory studies started to use sediment mixtures to mimic the behaviour of heterogeneous sewer sediment deposits (Skipworth 1996; Tait *et al.* 1998; De Sutter 2000; De Sutter *et al.* 2000). These experimental studies concentrated on the assessment of the impact of the cohesive-like properties within heterogeneous sediment deposits on mobility, primarily on the determination of the threshold of motion and transport functions in respect of grain size composition and their physical properties and particular flow parameters. Artificial surrogates were preferred as they permitted more reproducible results with a clearer quantification of individual parameters on the erosion strength than real sewer sediments. However, more recent tests with real sewer sediments have indicated that the use of surrogates can lead to unrepresentative predictions in respect to the behaviour of real sewer sediments (Tait *et al.* 2003a). This becomes true especially when the biochemical properties are important as these are not well reproduced when surrogates are used (Banasiak *et al.* 2005).

Many studies have tried to deliver a predictive tool for sediment transport in sewers. Various relationships have been developed or adapted from fluvial hydraulics. They can be divided into groups in terms of the sediment type and conceptual approach. The most common of these are the relationships in terms of excess shear stress ($\tau_b - \tau_{cr}$) or ($\theta_b - \theta_{cr}$), others use formula based on stream power concepts and then based on computer optimization of observed data using various dimensionless parameters, e.g. Ackers & White (1973).

NON-COHESIVE SEDIMENTS AND Φ-θ MODEL

The most widely used in the prediction of the amount of sediment transported as bed load is a relation of the "excess shear stress" type.

$$\Phi = f(\theta_b - \theta_{cr}) \tag{1}$$

A relation typical of the form of Equation 1 is the well known Meyer-Peter & Müller (1948) formula

$$\Phi = 8(\mu\theta_b - 0.047)^{1.5} \tag{2}$$

The use of such relationships can be problematic, first, it may be difficult to select a representative diameter d for a mixture, second is the determination of a deposit's threshold of motion. Finally is the accurate determination of the shear stress acting on the bed. This becomes particularly complicated in the case of ripple or dune formation and arises because only a fraction of the total bed shear stress τ_b is responsible for the transport of sediment.

Bed shear stress

Einstein & Barbarossa (1952) introduced a concept that stated that the spatially-averaged total bed shear stress τ_b is a linear superposition of the shear stress acting on the bed surface (called the effective or grain shear stress, τ_b') and the shear stress induced by the form drag τ_b'' caused by any bed forms.

$$\tau_b = \tau_b' + \tau_b'' \tag{3}$$

For a flat bed the grain shear stress is equal to the total shear stress. The determination of each component is problematic. For fully turbulent and rough flows with a flat bed the bed shear stress may be determined from (Kleijwegt 1992)

$$\frac{U}{u_*} = \frac{1}{\kappa}\log\left(\frac{12R_b}{k_s}\right) \tag{4}$$

where U – mean flow velocity, u_* – shear velocity ($= (\tau_b/\rho)^{0.5}$), R_b – hydraulic radius of the bed, k_s – roughness height. For sewer pipes both the wall and bed roughness needs to be accounted for. Kleijwegt (1992) proposed a method to calculate the bed shear stress

assuming that the mean flow velocities in narrow channels in the zones related to the bed and walls are not equal, i.e. the velocities near the wall are higher than those near the rougher bed. This method requires an iterative solution using predetermined roughness values for the wall boundaries. The determination of the effective roughness height k_s for the sediment deposit, despite long extensive research efforts, remains one of the major uncertainties. It is typically related to the characteristic size of the sediment, $k_s = cd_x$. There are indications that for uniform materials k_s can be as low as d_{50} or $2d_{50}$ (Nikuradse 1933; Song et al. 1998; Coleman et al. 2003; Banasiak & Verhoeven 2008), but the multiplication factor c is likely to increase with the sediment gradation. The influence of k_s on the shear stress according to the Equation 4 is shown in Figure 1; larger k_s results in higher effective bed shear stress.

Recent studies have also attempted to quantify the influence of bed forms on the shear stress partitioning in sewer pipes. Ota & Nalluri (2003) performed tests in an artificially roughened 305-mm-diam uPVC pipe and in a 225 mm concrete pipe with loose deposited beds consisting of uniform sediments with a d_{50} range of 0.6–5.7 mm. Ota & Nalluri (2003) obtained the following relationship between θ_b and θ'_b with validity restricted to the range of dunes.

$$\theta_b = 18\theta_b'^{1.87} \tag{5}$$

From a study performed in a 390 mm semi-circular flume using a fine sand ($d_{50} = 0.19$ mm) and a wider flow conditions range (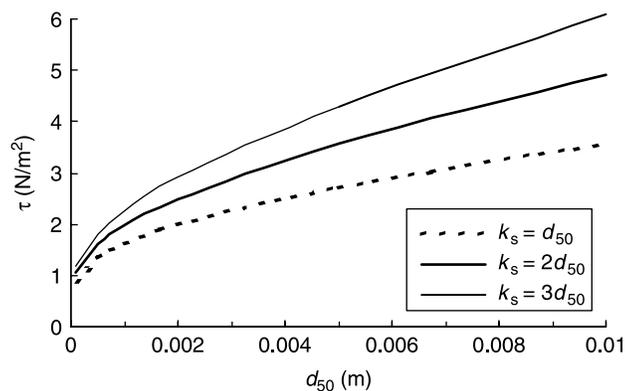reaching transition regime for bed forms, i.e. washed out dunes) Banasiak & Verhoeven (2006) obtained another relationship

$$\theta_b = 0.184 \ln \theta'_b - 0.65 \tag{6}$$

Figure 2 presents both Equations 5 and 6 as well as the relationship by Engelund (1967); the later is based on different data sets on flow resistance in open channels. The formulas for pipes and open channels appear to be different for lower and higher shear stresses. The first deviation can be explained by the lower threshold for motion observed in pipes. The deviation for higher shear stresses means that in the pipe conditions the shear stress related to bed form is significantly smaller, thus higher than expected levels of grain shear stress are encountered. This reflects on scale effects; in open channels the form component can reach even 90% of total shear stress, however in pipes this level is not achieved.

SELECTED TRANSPORT FORMULAS

Ota & Nalluri (2003)

Ota & Nalluri (2003) uses θ'_b to relate the bed load transport in the form of dimensionless transport parameter Φ

$$\Phi = 16.5(\theta'_b - 0.036)^{1.67} \tag{7}$$

Ota & Nalluri (2003) further revised the sediment transport relationship using data by May (1993), and

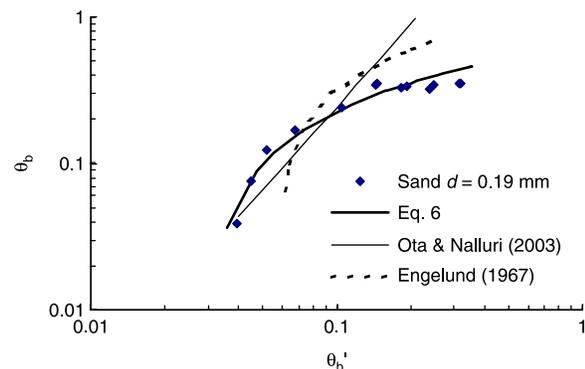Figure 1 | The influence of k_s on the shear stress using Equation 4.

Figure 2 | Total bed and grain dimensionless shear stress.

concluded that Equation (7) may be modified to the form

$$\Phi = 24(\theta_b' - 0.036)^{1.67} \tag{8}$$

Importantly, Ota & Nalluri (2003) use lower values for the threshold of motion compared to the Shields criterion. Several studies have also indicated, that the beginning of motion in circular cross-sections appears at average shear stresses 20–25% lower compared to that from rectangular flumes (Alvarez-Hermandez 1990; Berlamont et al. 2003; De Sutter et al. 2003; Banasiak & Verhoeven 2004).

May (1993)

May (1993) suggested a two stage calculation procedure. First, the roughness of the sediment bed is determined given the dimensions of the pipe, flow area, and the estimated roughness of the pipe walls and this is then used to find the overall hydraulic resistance of the pipe. The predicted flow conditions are then used to calculate the rate of sediment transport

$$C_v = \eta\left(\frac{W_b}{D}\right)\left(\frac{D^2}{A}\right)\left(\frac{\phi\lambda_g U^2}{8g(s-1)D}\right) \tag{9}$$

May's formula determines the grain friction factor λ_g from Colebrook-White

$$\frac{1}{\sqrt{\lambda_g}} = -2\log_{10}\left[\frac{d_{50}}{12R} + \frac{0.6275\nu}{UR\sqrt{\lambda_g}}\right] \tag{10}$$

which applies to both transition and fully turbulent flows and also suggests to use d_{50} as the sediment grain size. The effective mobility of the sediment is defined by the parameters

$$F_s = F_g\sqrt{\phi} \tag{11}$$

$$F_g = \sqrt{\frac{\lambda_g U^2}{8g(s-1)d_{50}}} \tag{12}$$

The value of the transport parameter η depends on F_s and is obtained by selecting the appropriate equation depending on the value of F_s.

Ackers (1984)

The formula of Ackers was based on the original Ackers-White sediment transport equations for alluvial channels (Ackers & White 1973), but some of the definitions of the parameters were revised so as to make them applicable to pipes and culverts as well as to open channels. The rate of sediment movement is expressed in terms of the non-dimensional transport parameter G_{gr}

$$G_{gr} = \left(\frac{C_v R}{d_{50}}\right)\left(\frac{A}{W_e R}\right)^{1-n}\left(\frac{u_*}{U}\right)^n \tag{13}$$

G_{gr} depends on the mobility of the sediment given by the parameter:

$$F_{gr} = \frac{u_*^n}{\sqrt{g(s-1)d_{50}}}\frac{U^{1-n}}{\left[\sqrt{32}\log_{10}\frac{12R}{d_{50}}\right]^{1-n}} \tag{14}$$

The transport and mobility parameters are linked by the equation

$$G_{gr} = H\left(\frac{F_{gr} - A_{gr}}{A_{gr}}\right)^m \tag{15}$$

where A_{gr} is the value of the mobility parameter at the threshold of movement. The various coefficients A_{gr}, H, m, n in the above equations are empirically related to the dimensionless grain size D_{gr}. This was further revised by Ackers (1991) in which the coefficients were combined and the flow resistance term was replaced by a power law term, The original formula for A_{gr}, H, m, n was recalibrated by HR Wallingford (1990). However, there was little change except for the finer sediments.

Nalluri & Alvarez (1992):

$$C_v = 0.0185 \lambda_b^{0.59} \left(\frac{d_{50}}{R_b}\right)^{0.419} \left(\frac{U^2}{g(s-1)R_b}\right)^{1.56} \qquad (16)$$

Perrusquia & Nalluri (1995):

$$\Phi = 0.0143 \theta_b'^{2.2} D_*^{0.38} \left(\frac{d_{50}}{D}\right)^{-1.11} \left(\frac{B}{D}\right)^{0.78} \qquad (17)$$

$$\Phi = \frac{C_v U R}{\sqrt{g(s-1)d_{50}^3}} \qquad (18)$$

Arthur *et al.* (1996):

$$\Phi = 0.00228 \theta_b'^{2.2} D_*^{0.38} \left(\frac{d_{50}}{D}\right)^{-1.11} \left(\frac{B}{D}\right)^{0.78} \qquad (19)$$

All the above models were based on measurements under uniform flow conditions. Table 1 comprises basic details of the experimental set-up used for the data collection for each models above.

Effect of graded sediments

Uniform sediments are rarely to be found in real sewer systems. The transport rates of sediment mixtures have been found to be higher than those for uniform materials with the same d_{50}. For instance, according to Ota & Nalluri (2003) the sediment transport may increase by some 50% when the geometric standard deviation of the (noncohesive) mixture

reaches 2.5. When dealing with heterogeneous granular sediments, a correction can be introduced for each grain size classes for the critical shear stresses (e.g. Wang 1997) or for the mobility number in the Ackers and White formula (De Sutter *et al.* 2003). This can provide significant improvement in predictive performance. The influence of gradation is that larger particles have lower movement thresholds compared to the Shields criterion due to the effects of protrusion, while fines movement can be reduced due to hiding effect. This effect has been widely recognised in rivers with the use of empirical hiding function (Sutherland 1992), however, De Sutter *et al.* (2003) indicated that the fluvial based functions required further adjustment before they could be applied in pipes. The effect of gradation tends to loose importance for larger shear stresses with high transport rates.

Formulas performance

All of the presented formulas have been assessed for the reliability of their predictions (Perrusquia 1991; May *et al.* 1996; Nalluri *et al.* 1997; De Sutter *et al.* 2003; McIllhatton *et al.* 2005). In many cases their predictions showed distinct differences with the measured values. In fact, the formulas were tested against different data, originating from both laboratory and field experiences, for different types of materials, including organic and cohesive like deposits. It was obvious that formulas developed for noncohesive sediments, when applied to partially cohesive deposits will not perform well.

Therefore, it is more appropriate to make an inter-comparison of these formulas and to apply them to the conditions similar to which they are derived. Here, data from a concrete laboratory pipe (diameter 390 mm) and a

Table 1 | Summary of evaluation of sediment transport formulas

Researcher	Pipe diameter (mm)	Grain sizes (mm)	Pipe material
Ota & Nalluri (2003)	255, 305	0.78, 0.9, 1.41, 2.83	Concrete, PVC
May (1993)	300, 450	0.47, 0.72	Concrete
Ackers & White (1973), HR Wallingford (1990)	N/A	0.04 to 28 mm – 2098 observations	N/A
Nalluri & Alvarez (1992)	154	0.53, 0.89, 0.89, 1.7, 2.9	PVC
Perrusquia & Nalluri (1995)	154, 225, 450	0.73, 0.9, 1.0, 2.5	PVC
Arthur *et al.* (1996)	154, 225, 450	0.73, 0.9, 1.0, 2.5	PVC

single uniform sand with $d_{50} = 0.19$ mm has been used to test the selected formulas that have been developed using single sized sediment. Figure 3 presents the selected methods and shows that they provide sediment transport predictions ranging in one order of magnitude. Certainly, such a situation is often not highlighted to engineers selecting a formula to predict in-sewer sediment transport.

However, this level of spread is not apparent in all formula, there is a better agreement between test data and the predictions from the equation by Ota & Nalluri (2003). This equation performs best and for a wide range of measured transport rates. The method of May (1993), which is recommended for use by Ackers et al. (1996), delivers significantly underestimated predictions. This is surprising, especially since May's (1993) data on the sediment transport were reevaluated by Ota & Nalluri (2003) to obtain Equation 8. This observation suggests that the method of May needs revision. Similarly, significant underestimation is noted for the methods of Nalluri & Alvarez (1992) and Arthur et al. (1996). However, the method of Perrusquia & Nalluri (1995) overpredicts substantially the sediment transport rate. The formula of Ackers (1991) underpredicts the transport for low values, and overpredicts it for larger shear stresses.

Possible reasons for the discrepancies between these models and experimental results may be:

Figure 3 | Measured and calculated transport rates for a noncohesive sediment.

- errors in estimating the magnitude of τ_b and τ_b'
- errors in estimating the threshold of motion;
- the number of calibration parameters and the accuracy of the data used to calibrate them.

This analysis indicates the superiority of the Φ-θ model against the formulas obtained by the analysis of large data to obtain several calibration parameters.

INFLUENCE OF COHESION

Values of the critical shear stresses for in-sewer deposits have been reported to range from 0.7 to 3.3 N/m² (Ristenpart & Uhl 1993). Sewer sediments can therefore exhibit thresholds of motion significantly higher than anticipated. The degree of cohesion depends on sediment input (fines, organic fraction) and time available for consolidation. Several studies have attempted to quantify the influence of fines and organic material on the deposit erosion strength (e.g. Williams et al. 1989; Alvarez-Hermandez 1990; Torfs 1995; Skipworth 1996; Tait et al. 1998; Camuffo et al. 2003). The cohesive strength depends both on the character of the fines and the size of granular fraction. Table 2 provides limits for the development of partly cohesive properties of a sediment bed. Furthermore, organic matter may change the cohesion strength of the deposit in two different ways due to particle bonding and weakening due to biochemical reactions (Vollertsen & Hvitved-Jacobsen 2000; Banasiak et al. 2005).

When the cohesion is low the sediments are termed as partly cohesive. This is because once eroded, partly cohesive sediments are reported to behave as noncohesive sediments and have bedforms on the eroding surface (Mitchener & Torfs 1996). However, these bedforms appear to be different from those in granular sediments. Such bed features result in the elutriation of fine particles from the top layer and the formation of a granular layer over the underlaying substrate. This has been observed in the field (Verbanck 1990) and in the laboratory using artificial mixtures (Alvarez-Hermandez 1990; Torfs 1995; Rushforth 2000) or real sewer sediment (Tait et al. 2003a; Banasiak et al. 2005). The active layer is much thinner compared to noncohesive sediment with resulting suppressed bedforms - quasi 2D ripples only few millimetres in height (Banasiak & Verhoeven 2008).
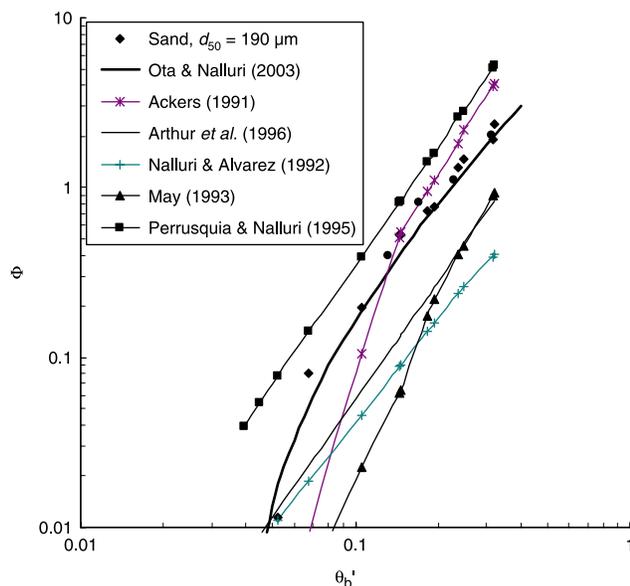
Table 2 | Percentage by weight of fines (less than 0.063 mm) at which the mode of erosion changes from granular to cohesive-type behaviour, for different cohesive materials mixed with sand

Cohesive material	Sand size (mm)	% of fines at transition in erosion mode	Source
Laponite clay	0.12–4.1	5–15	Alvarez-Hermandez (1990)
Kaolinite	0.23	3	Torfs (1995)
Montmorillonite	0.23	7–13	Torfs (1995)
Kaolinite	0.19	3–6	Banasiak & Verhoeven (2007)

Importantly, the bed load layer, if moving continuously has an inhibiting role for the progressive erosion of the underlying deposit. The development of the granular top layer may therefore contribute to the recession of the suspended solid transport in sewers following the 'first foul flush' (Rushforth et al. 2003).

With increasing cohesive strength the sediment bed becomes cohesive. This is when deposits are so strong that bed forms will be inhibited and erosion will follow patterns of surface or mass erosion, as described by Parchure & Mehta (1985). The mobility of cohesive sediments is also modeled in terms of excess bed shear stress, however, instead of transport rate, erosion rate is used (Mehta et al. 1989)

$$E = A(\tau_b - \tau_{cr})^n \tag{20}$$

Kuijper et al. (1989) proposed a nondimensional shear stress expression

$$E = M\left(\frac{\tau_b - \tau_{cr}}{\tau_{cr}}\right)^b \tag{21}$$

The erosion rate constants A, n, M and b are characterising parameters for the erosion behaviour of a homogeneous sediment with a constant concentration gradient and constant critical shear stress τ_{cr}. Equation 21 is highly sensitive to the value of τ_{cr}. Different studies have shown that the critical shear stresses and characteristic parameters vary widely and are sediment specific (Table 3). Another difficulty in establishing a formula of the form of Equations 20–21 is a typical significant scatter of experimental results with low correlation between the erosion parameters. This is especially true for in-sewer sediments displaying high heterogeneity. The accuracy in determining the bed shear stress and erosion rates may be also a source of uncertainties. Additionally, the influence of sediment type and bed preparation procedure in laboratory tests also plays an important role.

This type of relationship has been modified by changing the values of τ_{cr} with depth (Skipworth et al. 1999) and has been applied to field data with some success (Tait et al. 2003b). However, high quality data is required as the values of τ_{cr} and the other calibration factors are variable. In the case of partly cohesive sediments it is not clear what form of erosion or transport models are applicable. Different studies dealing with partly cohesive sediments have used either the erosion rate or transport rate to be related with the bed shear stress. The latter prevailed because of the observed granular bed load. The mode of erosion/transport dependent on the degree of

Table 3 | Critical shear stress for cohesive type sewer sediments

Sediment type	d_{50} (mm)	% < 63 μm / % organic material	τ_{cr}	Source
Paris	0.35–0.5	5 / 4–8	0.43–0.67	De Sutter (2000)
Ghent	0.035	70 / 8–10	1.0–1.5	De Sutter (2000)
Dendermonde	0.17	16 / 8	0.2–0.9*	Banasiak et al. (2005)

*depending on the deposition time.

cohesion may indeed be the indication towards an appropriate way of modelling partly cohesive sediments. As the bed load layer may be responsible for limiting erosion. A sensible approach may be to combine the transport rate for granular material with a model to predict the erosion rates for the washed-out fines combined with a deposit model to track the changing composition of the deposit. Certainly, more research is needed on the of partly cohesive sediment behaviour and how such deposits are formed, develop erosive strength and are then modified during erosion. It may be noted that this subject has recently become one of the major themes in the coastal sediment research (NCK 2005).

CONCLUSIONS

Hydraulic prediction of 1D pipe flows has become standard engineering practice, the major uncertainties in sewer performance are now related to sediment transport. Sediment transport is complex due to two factors: the variability of the nature of the sediment and the processes that control the sediment interaction with turbulent flow. The characteristics of in-sewer sediments have already been well described and classified, however, more effort is needed to enhance our understanding on the specific processes governing their mobility when located in heterogenous invert deposits.

Although the prediction of 1D pipe hydraulics is standard the prediction of the division between grain and bedform components of boundary shear stress is not. It is clear that partition methods developed for wide open channels do not perform well in pipes, as at higher and lower shear stress levels a higher than anticipated proportion is associated with the grains. Use of traditional boundary shear stress methods may therefore under estimate the threshold of motion and thus transport.

Many transport capacity equations have been calibrated with only a limited number of data. Even when comparing formulae against similar data to their calibration set some formulae still produce errors up to one order of magnitude. To predict the transport rates of granular sediments the formula of Ota and Nalluri can be recommended. Comparison with independent data suggested that the formulas

of Ackers (1991) and May (1993) may need modification to improve in performance. However, it is difficult to state about a clear superiority of any particular formula mentioned over the wide range of sediment characteristics found in sewers.

It has been demonstrated that the erosion and transport behaviour of a sediment can be sensitive to even minor changes in deposit composition. This is particularly true when attempting to estimate the threshold of erosion. Mixed granular deposits appear to be more mobile than expected. Empirical functions have been developed to adjust erosion thresholds in existing transport equations and these can provide significantly improved predictions.

Cohesive sediment deposits can be created with the sediment types found in sewers. A reliable prediction of their mobility should be based on experimental testing since no reliable analytical solutions are currently available. Some work has used both erosion rate and transport rate formulations however the empirical calibration parameters vary widely and predictions are sensitive to these values. The threshold of erosion is a key parameter and this appears to depend on the sediment characteristics and the duration of deposit formation. There is no accepted method to predict the development of strength of partially cohesive deposits with time or composition. It is not clear whether erosion or transport rate equations are applicable for partially cohesive deposits. It may be appropriate to use an erosion model for fine sediments and a transport rate approach for coarser granular sediment and then to combine the two approaches by modelling the change in deposit properties.

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