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Implications of bed load transport mode on flushing flows in smooth pipe

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IMPLICATIONS OF BED LOAD TRANSPORT MODE ON FLUSHING FLOWS IN A SMOOTH PIPE

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A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, Johannesburg, in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

Johannesburg, 2015

DECLARATION

I declare that this research report is my own unaided work. It is being submitted to the Degree of Master of Science to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

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…….. day of ……, ……

ABSTRACT

The mode of sediment transport in a pipe has important practical implications on self-cleansing flows. The two transport modes; discrete and collective, have significantly different active storage volumes and effective grain velocities. Excessive deposition of sediment occurs in the collective mode and can have harmful environmental, social and economic effects. The conditions under which each mode prevails and their effects on self-cleansing flows are explored in this research report through laboratory investigation. Supply-limited bed load experiments were performed resulting in two new relationships being developed to predict the prevailing transport mode and the volume of active storage in a pipe under certain conditions. A recommended methodology and a set of new design graphs are presented as amendments to the current design guidelines for the prediction of flushing flows rates and corresponding minimum durations for sediment transport in a smooth pipe.

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LIST OF SYMBOLS

A	flow area
A_{gr}	mobility parameter at threshold of movement
C_{AS}	active storage concentration
C_v	volumetric sediment concentration
C_3	empirical coefficient
d	pipe diameter
d_{50}	mean grain size
D	flow depth
D_{gr}	dimensionless grain size
F_s	effective mobility of sediment particles
F_g	grain mobility
F_{gr}	mobility parameter
g	acceleration due to gravity
H	empirical coefficient
J	empirical coefficient
k	conversion factor for Manning equation
k	rigid bed roughness without sediment
k_s	rigid bed roughness with sediment
K	empirical coefficient
m	empirical coefficient
m_{AS}	mass of active storage per metre length
m_T	total mass of active storage
n	empirical coefficient
n	Manning's roughness coefficient
P_b	bed perimeter
P_w	wall perimeter
q^*	dimensionless bed load parameter
q_{crit}	critical sediment supply rate
q_s	unit width sediment supply rate

q_{sbm} threshold sediment transport rate

Q flow rate

Q_s total bed load rate

R hydraulic radius

R_c^* particle Reynolds number

R_* shear Reynolds number

S bed slope

S_s sediment specific gravity

SF shape factor

t time

u_* shear velocity

U mean flow velocity

v_L limiting flow velocity without deposition

v_p grain velocity

v_t threshold velocity required to initiate movement

V_s volume of active storage

V_w volume of water

w particle settling velocity

W_e effective bed width

α empirical coefficient

β empirical coefficient

γ empirical coefficient

γ specific weight of water

γ_s specific weight of sediment

δ empirical coefficient

Δ effective specific density

ϵ empirical coefficient

η transport parameter

θ dimensionless bed shear stress

θ transition coefficient for particle Reynolds number

κ von Karman constant

λ	Darcy-Weisbach friction factor
λ_b	bed friction factor
λ_c	composite friction factor
λ_g	grain friction factor
λ_w	clean pipe friction factor
μ	absolute viscosity of water
ν	kinematic viscosity of water
ρ	density of water
ρ_s	density of sediment
σ	sorting of sediment mixture
τ_0	bed shear stress
τ_b	critical bed shear stress
τ^*	Shields stress parameter
χ	dimensionless grain velocity

1 INTRODUCTION

1.1 Self-cleansing design criteria

Sediment deposition in pipes can cause a significant reduction in transport capacity by blocking off part of the pipe's flow area and by increasing the hydraulic roughness experienced by the flow. A build up of sediment may also act as a store for pollutants. These effects are highly undesirable in a transport system as they may incur significant costs. Current design practices require the specification of a minimum velocity in order to achieve no deposition within a stormwater or sewage transportation system, however recent studies have suggested that this approach may be insufficient as it does not consider the effects of the sediment characteristics, the sediment concentration or the hydraulic behaviour of the flow.

James et al. (2011) distinguished between two modes of bed load transport, namely discrete and collective movement modes. Discrete movement occurs as sediment particles travel individually along the bed of the channel. In this case the average velocity at which the sediment moves is the velocity of the individual particles. Collective movement occurs when sediment particles collide and form bed forms along the bed of the channel. The average velocity of this situation is the velocity of the bed forms, which is significantly lower than that of discrete movement owing to the time in which sediment particles remain stationary in the bed form. The mode of bed load transport, by this theory, has an impact on the effectiveness of the minimum velocity criterion.

The mode of bed load transport in a pipe has important practical implications on self-cleansing flows. Sediment that is deposited in the collective mode, remains stationary for some time allowing the sediment to consolidate, thus requiring a higher velocity and boundary shear stress to be moved. Cases in which collective movement is the mode of transport require a much greater volume of sediment to be moved. Using the same minimum velocity criterion as is also used for discrete movement will require a significantly longer duration to achieve a clean pipe. In

order to achieve efficient self-cleansing pipes, criteria must be derived considering the incipient motion threshold velocity of the sediment as well as the volume of active storage required to be transported which can be determined through analysis of the existing transport mode. Adjusting design conditions to achieve discrete movement of the bed forms would significantly reduce the duration required to achieve a clean pipe.

This research report is related to the development of a criterion to define the mode of bed load transport in a smooth pipe in terms of the sediment properties and the hydraulic conditions of the flow; the determination of the quantity of active sediment storage within a water transport system for a given sediment transport rate; an analysis of the effect of duration on the self-cleansing velocity requirements and a methodology to determine the required flushing duration.

1.2 Purpose of the research

This research report contributes to the development of the theory describing sediment transport in urban drainage systems and the improvement of current design criteria for self-cleansing flows.

A threshold relationship describes the transition between discrete bed load movement to collective movement as a function of the sediment transport rate, flow conditions and sediment properties. The direct measurement of the threshold point is difficult to identify owing to the formation and breaking up of bed forms in the transition stages. The active storage is used to indirectly determine the critical sediment transport rate. This research report presents the development of a new equation describing the threshold relationship applicable to both partially full pipe flow and flat rectangular channel flow.

The quantity of active storage present in a pipe at a given time influences the time required to achieve a clean pipe. Active storage travelling discretely travels at the speed of the individual grains however active storage travelling in bed forms takes significantly longer to be flushed out. This research report presents an active storage prediction methodology for a given sediment supply rate and a

preliminary methodology to estimate the required flushing duration. Recommended amendments to the existing design practices are presented.

1.3 Research objectives

The aim of this research is to:

- develop a criterion describing the threshold between the two modes of bed load transport,
- develop a relationship enabling the quantity of active storage in a pipe to be predicted,
- determine the effects of the transport mode on the required flushing duration, and
- provide a set of amendments to the current design guidelines for self-cleansing flows.

These objectives are addressed by conducting a literature review, experimental investigation and data analyses.

1.4 Layout of the research report

The layout of the research report follows the following chapters:

1. **Introduction.** This chapter provides a brief overview of the problem investigated and the purpose for doing so.
2. **Background.** This chapter includes a review of the current design criteria for self-cleansing flows, approaches and procedures for better defining self-cleansing criteria.
3. **Experimental investigation of modes of bed load transport.** This chapter describes and provides the results of the experimental investigation into the transport mode threshold relationship. An equation is proposed allowing for the mode of bed load transport to be predicted based on sediment and hydraulic properties.

4. **Active storage prediction.** This chapter includes the analysis and manipulation of data to propose a dimensionless relationship enabling the prediction of the active storage volume in a pipe for a given scenario.
5. **Flushing durations for self-cleansing flows.** This chapter describes the importance of considering time effects on self-cleansing design criteria through experimental observations. Preliminary methods to determine the required flushing duration are investigated and presented.
6. **Design guideline amendments**. This chapter presents the recommended amendments to the current design methodologies for self-cleansing flows based on the results of this study. Numerical examples demonstrating the recommended methodologies are presented.
7. **Conclusions and recommendations.** This chapter presents the conclusions of the project and recommendations for further research. The project has proposed methods for predicting the mode of bed load transport and the quantity of active storage in a drainage pipe. Recommendations regarding the optimisation of current design criteria are provided based on experimental investigation. The applicability of the proposed methods is limited to the sediments tested; further investigation into the possible correlation of other sediments types and sizes would be valuable.

2 BACKGROUND

2.1 Introduction

The primary purpose of this research is to identify a criterion describing the modes of bed load transport in a smooth pipe. The need for this research is owing to the insufficiency in the current design practices for defining self-cleansing flows in urban drainage systems. Designing pipes to have sufficient transport capacities is important in preventing large sediment deposits as well as predicting the position and movement of pollutants within the system (De Sutter et al., 2003).

The timeous re-entrainment of sediment deposits is particularly important when considering self-cleansing criteria as the longer a sediment deposit remains stationary, the higher the velocity and boundary shear stresses required to dislodge it. The sediment deposit gains strength through consolidation and through the development of cohesion caused by the presence of organic materials in the deposits (Arthur et al. 1999). This theory lends itself to the hypothesis that higher velocities would be required to move sediment in bed forms than are required for sediment being transported discretely.

Traditionally, it was considered necessary to achieve a self-cleansing or 'clean' pipe. This refers to a pipe in which no deposition is permitted. To achieve this, a minimum velocity criterion was specified. A general value of minimum velocity was specified in order to ensure that a certain velocity would be achieved at least once a day in order to fully flush out the sediment deposits. Pipe gradient conditions were also specified as an alternative criterion for self-cleansing. Although this approach has had a degree of success, it does not adequately represent the sediment transport in pipes, as it does not consider the characteristics of the sediment, the sediment concentration or the hydraulic behaviour of the flow. Further criteria are required to be investigated in order to better define self-cleansing conditions.

2.2 General minimum velocity criteria

The general minimum velocity criterion was first defined as the requirement that a certain velocity should be reached at a certain flow depth with a certain frequency of occurrence. It was considered adequate to flush out all sediment deposits in any sewer system.

For example, the British Standard BS EN 752-4 (British Standards Institution, 1998) recommends that in order for a pipe to be self-cleansing, a minimum velocity of 0.7m/s in small diameter drains and sewers (less than 300mm) must be achieved at least once a day or that a gradient of 1:pipe diameter must be specified (De Sutter et al. 2003). This method was apparently successful in some cases.

SANRAL (2013) suggest a self-cleansing velocity of 0.7m/s for full pipe flow in a storm drain to prevent sediment deposition and 0.6m/s for a flow depth of 25% of the pipe diameter. Alternatively, this method may be used to determine the minimum required slopes at which the stormwater pipes should be set to achieve no deposition using resistance equations such as the Manning or Darcy-Weisbach equations. This is the method currently being used in practice however does not consider individual cases with varying pipe characteristics, sediment concentrations or hydraulic conditions.

Nalluri and Ab. Ghani (1996) reviewed the minimum velocity criterion and concluded that this method overdesigns the slope for small diameters (<500mm) and under designs the slope for large diameters (>1000mm). This indicates the need for further studies.

2.3 Transport rate prediction formulae

In acknowledging that the general minimum velocity criterion was inadequate, the concept of limited depth deposition was introduced and developed by Ackers (1991) and May (1993). Achieving self-cleansing conditions was considered to be impractical so Ackers (1991) and May (1993) proposed the concept of allowing for a limited sediment build-up providing that no long-term build-up occurs. This concept has the advantage of allowing a higher sediment transport capacity owing

to the increase in effective width of the sewer caused by the deposited bed (Nalluri et al. 1997).

Ackers (1991)

Ackers (1991) acknowledged the shortcomings of the minimum velocity approach and proposed the inclusion of sediment properties and differential sediment input at varying points in the system.

This approach requires estimation of the local sediment transport capacity. In this methodology it is required to determine the rate of sediment transport, which is expressed as the non-dimensional transport parameter (G_{gr}). Ackers (1991) proposes the following transport relationships:

$$G_{gr} = \left(\frac{C_v R}{d_{50}}\right)\left(\frac{A}{W_e R}\right)^{1-n}\left(\frac{u_*}{U}\right)^n \quad (2\text{-}1)$$

G_{gr} is dependent on the sediment mobility, which is expressed as the flow mobility parameter F_{gr}.

$$F_{gr} = \frac{{u_*}^n}{\sqrt{g(S_s-1)d_{50}}}\frac{U^{n-1}}{\left(\sqrt{32}\log_{10}\frac{12R}{d_{50}}\right)^{1-n}} \quad (2\text{-}2)$$

$$u_* = \sqrt{gRS} \quad (2\text{-}3)$$

The transport and mobility parameters are related by the following equation:

$$G_{gr} = H\left(\frac{F_{gr}-A_{gr}}{A_{gr}}\right)^m \quad (2\text{-}4)$$

where:

A	=	flow area
C_v	=	volumetric sediment concentration
d_{50}	=	mean grain size
R	=	hydraulic radius
u_*	=	shear velocity
g	=	acceleration due to gravity

S = bed slope

S_s = relative density of sediment

U = mean flow velocity

W_e = effective bed width

A_{gr} is defined as the mobility parameter (F_{gr}) at the threshold of movement.

A_{gr}, H, m and n are empirical coefficients that are related to the dimensionless grain size (D_{gr}) by the following equations:

$$D_{gr} = d_{50}\left[\frac{(S_s-1)g}{\nu^2}\right]^{1/3} \quad (2\text{-}5)$$

$$A_{gr} = 0.14 + \frac{0.23}{\sqrt{D_{gr}}} \quad (2\text{-}6)$$

$$m = 1.34 + \frac{9.66}{D_{gr}} \quad (2\text{-}7)$$

$$\log_{10} H = 2.86 \log_{10} D_{gr} - \left(\log_{10} D_{gr}\right)^2 - 3.53 \quad (2\text{-}8)$$

n is a transition parameter varying from 1 for fine material to 0 for coarse material. ν is the kinematic viscosity of water.

De Sutter et al. (2003) performed experiments in order to validate the performance of the method proposed by Ackers (1991). The results of this showed a poor correlation between experimental results and those predicted by the method of Ackers (1991). Nalluri et al. (1997) appraised the method of Ackers (1991) and concluded that this method overestimated the sediment transport capacity.

May (1993)

The methodology proposed by May (1993) requires the sediment bed roughness to be determined using the pipe dimensions, the flow area and an estimation of the pipe wall roughness. This is then used to determine the overall hydraulic

resistance of the pipe, which with the flow conditions is used to calculate the sediment transport rate.

The flow resistance is dependent on the grain mobility (F_g) and the Froude number. The grain mobility is defined as:

$$F_g = \sqrt{\frac{\lambda_g U^2}{8g(S_s-1)d_{50}}} \tag{2-9}$$

The grain friction factor (λ_g) can be determined by iterative application of the Colebrook-White resistance equation as a function of the mean grain size (d_{50}), the hydraulic radius (R) and the mean flow velocity (U).

The bed friction factor (λ_b) can be obtained from the mobility parameter based on total bed shear stress (F_b), which is obtained as a function of F_g depending on the values of F_g and the Froude number.

$$F_b = \sqrt{\frac{\lambda_b U^2}{8g(S_s-1)d_{50}}} \tag{2-10}$$

The composite roughness of the pipe can be obtained by:

$$\lambda_c = \frac{P_w\lambda_w+P_b\lambda_b}{P_w+P_b} \tag{2-11}$$

where:

λ_c = composite friction factor

P_w = wall perimeter

P_b = bed perimeter

λ_w = clean-pipe friction factor obtained from the Colebrook-White equation

The particle Reynolds number ($R_c{}^*$) is required in order to calculate the sediment transport rate. It is defined as:

$$R_c{}^* = \sqrt{\frac{\lambda_c}{8}}\left(\frac{U\, d_{50}}{v}\right) \tag{2-12}$$

The effective mobility of the sediment particles is defined by:

$$F_s = F_g\sqrt{\theta} \tag{2-13}$$

where θ is the transition factor for particle Reynolds number.

The volumetric sediment concentration is given by:

$$C_v = \eta\left(\frac{P_b}{d}\right)\left(\frac{d^2}{A}\right)\left(\frac{\theta\lambda_g U^2}{8g(S_s-1)d}\right) \tag{2-14}$$

where d is the pipe diameter.

The transport parameter (η) is dependent on the effective mobility of the sediment particles (F_s) and can be obtained by selecting the appropriate equation depending on the value of F_s.

The equations proposed by May (1993) are empirically based and are therefore only applicable where the assumptions made in their derivation hold true. May's (1993) formulae use a single representative grain size in determining the effective mobility of all the particles in the deposit. This method would not be appropriate to use for sediment in which the particle distribution is wide (De Sutter et al. 2003).

De Sutter et al. (2003) performed a series of experiments in order to validate the equations proposed by May (1993) to determine transport capacity. The results showed that the method performed poorly, possibly due to its empiricism. Nalluri et al. (1997), in their appraisal of the method presented by May (1993) conclude that the method overestimates the sediment transport capacity.

The comparisons made by De Sutter et al. (2003) suggest that using transport rate prediction formulae that are based on empirical relationships calibrated using uniform sediment size are unsuitable in predicting the movement and transport of sediment deposits in sewer pipe systems and therefore should not be used as a design method.

2.4 Alternative methods

Camp's criterion

Almedeij and Almohsen (2010) acknowledge that the common practice minimum velocity of 0.7m/s under pipe full capacity is insufficient in achieving a clean pipe for all cases. The Camp's criterion is used to determine the self-cleansing velocity based on properties such as pipe diameter, slope, roughness, sediment particle size and specific weight of sediment allowing for a broader application.

The Camp's criterion is derived from the Manning equation and rearranged as:

$$U = \frac{k}{n} R^{1/S_s} [\tau^* (S_s - 1) d_{50}]^{1/2} \quad (2\text{-}15)$$

$$\tau^* = \frac{\tau_0}{(\gamma_s - \gamma) d_{50}} \quad (2\text{-}16)$$

where:

k	=	conversion factor (1 for metric, 1.49 for English units)
n	=	Manning's roughness coefficient
R	=	hydraulic radius
τ^*	=	Shields stress parameter
τ_0	=	bed shear stress
d_{50}	=	sediment particle size
S_s	=	sediment specific gravity
γ	=	specific weight of water
γ_s	=	specific weight of sediment

The application of the Camp's criterion requires setting the values of τ^* and d_{50} for a given pipe size and type of material. This will produce a U value; a velocity to be used in design to avoid sediment deposition (Almedeij and Almohsen, 2010).

The Camp's criterion has the advantage of being simple and easy to use, however, it is noted that the criterion does not consider the effect of sediment concentration (Almedeij and Almohsen, 2010).

CIRIA Methodology

Arthur et al. (1999) acknowledge that existing methodologies used to design sewers to control sediment problems are inadequate. The Construction Industry Research and Information Association (CIRIA) proposed a design approach in order to reduce sewer sedimentation problems. Existing methods do not consider the range of materials in sewer sediments or the geometric characteristics of the actual sewers which are properties shown to have an effect on sediment transportation and deposition.

The method proposes the following definition of self-cleansing: "An efficient self-cleansing sewer is one having a sediment transporting capacity that is sufficient to maintain a balance between the amounts of deposition and erosion, with a time-averaged depth of sediment deposit that minimizes the combined costs of construction, operation, and maintenance." (quoted from May et al., 1996).

Arthur et al. (1999) suggest that there are three main modes of transport of sediment through a sewer system. These are:

1. Suspension
2. Bed load (transport along or near the bed)
3. Semi-permanent deposits

Although not considered as part of the CIRIA methodology, Arthur et al. (1999) acknowledge that bed forms can be considered as an additional mode of transport as their movement along a sewer invert is different to normal bed load.

Most self-cleansing criteria are based purely on the bed load aspect of sewer sedimentation owing to the effects of sediment deposits and bed forms on flow

conditions within a sewer system and the effects of pollutants deposited in a sewer system.

CIRIA proposed a formalised procedure taking into account sediment present both as deposits and in suspension. This procedure allows for a self-cleansing velocity to be obtained specific to each pipe length under consideration (Arthur et al., 1999). The self-cleansing velocity is required to meet the following criteria:

1. Criterion 1: Suspended sediments

A minimum concentration of suspended sediment is required to be transported. Relationships by Macke (1982) and Ackers (1991) are proposed in order to determine this.

Macke's (1982) equation is recommended as the normal design method.

$$C_v = \frac{\lambda^3 {v_L}^5}{30.4\,(S_s-1)w^{1.5}A} \tag{2-17}$$

where:

C_v	=	volumetric sediment concentration
v_L	=	limiting flow velocity without deposition
S_s	=	sediment specific gravity
A	=	flow area
w	=	particle settling velocity
λ	=	Darcy-Weisbach friction factor

Equation 2-17 is valid for $\tau \geq 1.07\text{N/m}^2$ (Butler et al., 2003).

In the case of sediment being transported over a sediment bed, the Ackers (1991) equation is recommended.

$$C_v = J\left(W_e\frac{R}{A}\right)^{\alpha}\left(\frac{d_{50}}{R}\right)^{\beta}\lambda^{\gamma}\left(\frac{v}{\sqrt{g(S_s-1)R}} - K\lambda^{\delta}\left(\frac{d_{50}}{R}\right)^{\epsilon}\right)^{m} \tag{2-18}$$

where:

W_e	=	effective width of sediment bed

d_{50} = mean particle size

R = hydraulic radius

g = acceleration due to gravity

λ = Darcy-Weisbach friction factor

v = kinematic viscosity of water

$J,\ K,\ m,\ \alpha,\ \beta,\ \gamma, \delta$ and ϵ are empirical coefficients and are dependent on the dimensionless grain size (D_{gr}) and the mobility parameter at incipient motion (A_{gr}).

2. Criterion 2: Bed load transport

A velocity must be determined at which the coarse, inorganic, granular material can be transported. This rate must be effective in limiting the deposition depth to a specific proportion of the pipe diameter or to completely flush out all sediments without deposition.

No bed load equations for limit of deposition cases were adequate in predicting the behaviour of sediment transport. A new equation was developed fitting the laboratory data over the full range of limit of deposition data.

$$C_v = 3.03x10^{-2}\left(\frac{D^2}{A}\right)\left(\frac{d_{50}}{D}\right)^{0.6}\left(1-\frac{v_t}{v_L}\right)^4\left(\frac{{v_L}^2}{g(S_s-1)D}\right)^{1.5} \quad (2\text{-}19)$$

$$v_t = 0.125\sqrt{g(S_s-1)d_{50}}\left(\frac{D}{d_{50}}\right)^{0.47} \quad (2\text{-}20)$$

where:

v_t = threshold velocity required to initiate movement

D = flow depth

d_{50} = mean grain size

S_s = relative density of sediment

For the case of bed load transport in circular pipes with deposited beds, the equation by May (1993) was found to fit laboratory data with the best agreement and is recommended in this case.

$$C_v = \eta \left(\frac{W_e}{D}\right)\left(\frac{D^2}{A}\right)\left(\frac{\theta \lambda_g v^2}{8g(S_s-1)D}\right) \quad (2\text{-}21)$$

where:

W_e	=	effective bed width
λ_g	=	grain friction factor
θ	=	transition coefficient for particle Reynolds number
η	=	sediment transport parameter

3. Criterion 3: Entrainment from cohesive-like in-sewer deposits

A flow velocity must be specified that causes a shear stress effective in removing inorganic deposited sediment that may have developed additional cohesive strength. Usually, this criterion is defined as a bed shear stress of greater than 2.0 N/m^2 assuming a cohesive sediment particle size of 1.00mm and a Colebrook-White roughness value of k_b = 1.2mm (Arthur et al., 1999).

The full-bore velocity (v_f) can be determined from equation 2-22.

$$v_f = \sqrt{\frac{8\tau_b}{\rho \lambda_b}} \quad (2\text{-}22)$$

$$\lambda_b \approx \frac{1}{4\left[\log_{10}\left(\frac{k_b}{3.7D}\right)\right]^2} \quad (2\text{-}23)$$

where:

τ_b	=	critical bed shear stress
λ_b	=	bed friction factor

Arthur et al. (1999) provide Figure 2-1 as a schematic of the CIRIA design procedure.

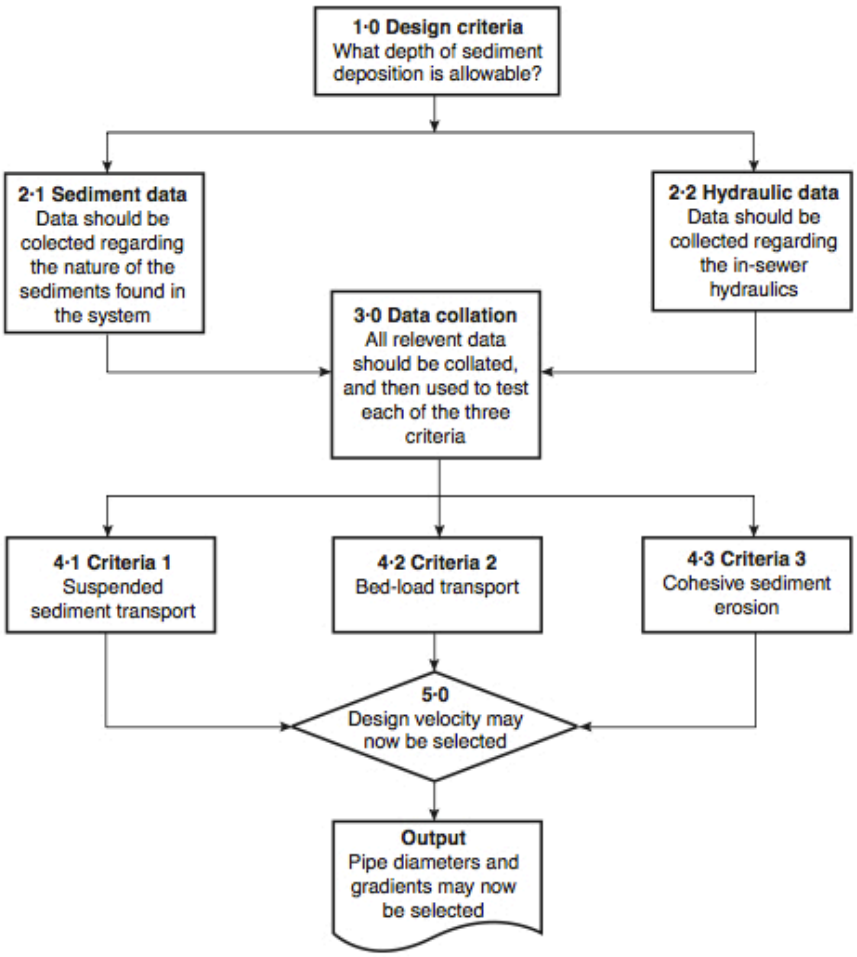


Figure 2-1 Schematic application of CIRIA design procedure (Arthur et al., 1999)

Arthur et al. (1999) provide Figure 2-2 to illustrate how the design of different pipe sizes is governed by different criteria in the CIRIA design procedure.

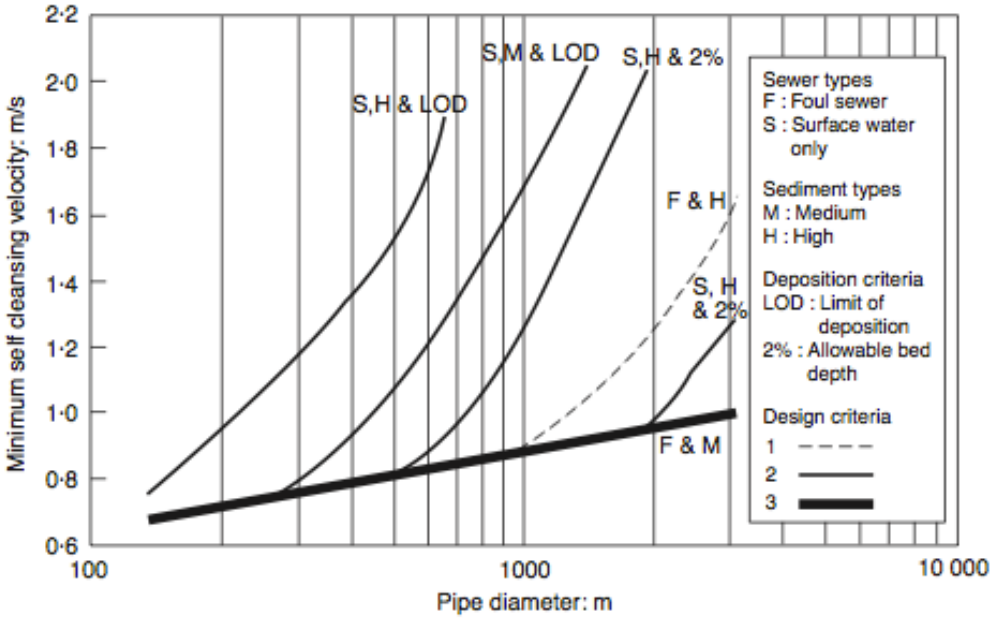


Figure 2-2 Minimum required flow for different sediment transport criteria (Arthur et al., 1999)

The introduction of the CIRIA methodology indicates the importance of various factors that were previously excluded in design considerations such as boundary shear stress, hydraulic roughness and pipe characteristics. It should be noted that the CIRIA method does not consider the very complex physical processes occurring during erosion and the transportation of sewer sediments. It is an improvement on previous design criteria but requires further improvement for acceptable usage.

2.5 Other research

A number of other researchers have proposed different equations in order to determine the required velocity to achieve self-cleansing conditions.

Ab. Ghani (1993) proposes the following:

For clean pipes:

$$\frac{U}{\sqrt{gd_{50}(S_s-1)}} = 3.08{C_v}^{0.21}{D_{gr}}^{-0.09}\left(\frac{R}{d_{50}}\right)^{0.53}{\lambda_g}^{-0.21} \quad (2\text{-}24)$$

$$\lambda_g = 1.13\lambda^{0.98}{C_v}^{0.02}{D_{gr}}^{0.01} \quad (2\text{-}25)$$

For pipes with deposited loose beds:

$$\frac{U}{\sqrt{gd_{50}(S_s-1)}} = 1.18{C_v}^{0.16}\left(\frac{W_e}{D}\right)^{-0.18}\left(\frac{d_{50}}{d}\right)^{-0.34}{\lambda_g}^{-0.31} \quad (2\text{-}26)$$

$$\lambda_g = 0.0014{C_v}^{-0.04}\left(\frac{W_e}{D}\right)^{0.34}\left(\frac{R}{d_{50}}\right)^{0.24}{D_{gr}}^{0.54} \quad (2\text{-}27)$$

Meyerle et al. (1991) reviewed existing criteria for self-cleansing flow velocities for rectangular channels and circular cross section channels with both rigid smooth and rough beds. It was concluded that no existing equation adequately described the behaviour of the flow over all flow regimes and new equations were proposed. Meyerle et al. (1991) proposed two methods for the determination of the self-cleansing flow velocity for smooth circular cross section channels. The first method is a simple equation, independent of the channel roughness, allowing a quick calculation of the average self-cleansing velocity. The second method is more complex and comprises two equations, dependent on the roughness of the channel, solved iteratively. The first method's equation is presented in Equation 2-28 and the second method's equations are presented in Equations 2-29 and 2-30.

$$\frac{U}{\sqrt{gd_{50}(S_s-1)}} = 4.32\ {C_v}^{0.23}(d_{50}(R)^{-0.68}) \quad (2\text{-}28)$$

$$\frac{U}{\sqrt{gd_{50}(S_s-1)}} = 14.43{D_{gr}}^{-0.14}{C_v}^{0.18}\left(\frac{d_{50}}{R}\right)^{-0.56}{\lambda_g}^{0.18} \quad (2\text{-}29)$$

$$\frac{k_s-k}{R} = 0.0130\ {D_{gr}}^{0.24}{C_v}^{0.40} \quad (2\text{-}30)$$

where:

k = rigid bed roughness without sediment

k_s = rigid bed roughness with sediment

The methods proposed by Meyerle et al. (1991) are an extension and refinement of previous results.

It can be seen that there is an agreement amongst most researchers regarding the parameters on which the critical self-cleansing velocity depends. This gives a basis for further research.

2.6 Modes of bed load movement

The importance of considering the bed load transport in determining self-cleansing flows has been acknowledged and taken into account by various researchers; however, the modes of bed load transport are not commonly considered. In the case of supply-limited transport over rigid beds, there are two modes of bed load movement: discrete and collective movement. Discrete movement refers to the transport of sediment as individual grains. These grains move separately and continuously. Collective movement refers to the transport of sediment in bed forms. The movement of bed forms occurs as grains join the deposit, remain at rest for some time and then leave the bed form. Owing to the rest periods experienced in the bed forms, the average velocity of collective movement is significantly lower than that of discrete movement. Active storage refers to the volume of sediment in transport. In discrete movement, the active storage is the total volume of all the moving sediment grains. In collective movement, the active storage is the sum of the moving and the temporarily at rest grains (James et al. 2011).

The build-up of sediment deposits is largely influenced by the mode of bed load movement owing to the difference in average velocities of each mode. Self-cleansing flows must satisfy the incipient motion condition and consider the amount of active storage (which is indicative of the mode of movement).

James et al. (2011) performed laboratory experiments to determine the conditions under which discrete and collective movement occur and to establish a criterion for the determination of the transport mode for the case of a smooth, rigid, flat bed. The investigation focused on the following relationship:

$$\frac{u_*}{w} = f(q^*, R_*) \tag{2-31}$$

where:

$$q^* = \frac{q_s}{\rho_s \sqrt{(S_s - 1) g {d_{50}}^3}} \tag{2-32}$$

$$R_* = \frac{\rho u_* d_{50}}{\mu} \tag{2-33}$$

and

u_*/w	=	movability number
R_*	=	shear Reynolds number
u_*	=	shear velocity
w	=	particle settling velocity
ρ	=	density of water
ρ_s	=	density of sediment
q^*	=	dimensionless bed load parameter
q_s	=	unit width sediment supply rate
S_s	=	relative density of sediment
g	=	acceleration due to gravity
d_{50}	=	mean grain size

The relationship between the active storage and sediment transport rate was used to indirectly determine the threshold value between discrete and collective movement. The preliminary threshold relationship is defined as:

$$\left(\frac{u_*}{w}\right)_{bm} = 0.0738 \ln\left(\frac{q^*}{R_*}\right) + 0.721 \tag{2-34}$$

The subscript 'bm' refers to the boundary between the two modes of movement.

Equation 2-34, in conjunction with incipient motion criteria can be used to better describe supply-limited sediment movement in rigid boundary channels (James et al. 2011).

2.7 Conclusion

The mode of bed load transport in smooth rectangular channels has been considered by James et al. (2011). A threshold relationship between the discrete and collective modes has been developed and is presented as equation 2-34. This equation is applicable to open channels with a flat bed however sewer and stormwater drainage systems consist mainly of pipes. The primary focus of this research is to expand on the research by James et al. (2011) by considering the modes of bed load transport through a pipe.

3 EXPERIMENTAL INVESTIGATION OF MODES OF BED LOAD TRANSPORT

3.1 Introduction

The mode of bed load transport has important practical implications on self-cleansing flows. At the recession of a flow event, sediment is deposited on the channel bed and depending on the flow conditions, transported by the flow. For the discrete mode, the effective velocity of the sediment is the velocity of the individual grains. In the collective mode, particle collisions and temporarily at rest particles reduce the effective velocity. This suggests that to achieve a self-cleansing condition for each mode requires different criteria. This indicates the need for this study.

As discussed in Chapter 2, James et al. (2011) developed an equation defining the threshold of movement between the two modes of flow for a smooth, flat, rigid surface. This chapter presents the results of the experimental investigation performed to develop a threshold relationship for a smooth pipe and to determine whether any correlation exists between the flat surface data and the pipe data. The laboratory investigation was conducted in the hydraulics laboratory in the School of Civil and Environmental Engineering at the University of the Witwatersrand.

Laboratory experiments were performed in Flume B. A 300mm diameter, 11.4m long PVC pipe was halved and placed into the 380mm wide tilting flume and secured using wooden spacers. The sediment feeder was positioned at the upstream end of the tilting portion of the flume. The experimental set-up is shown in Figures 3-1 to 3-6.

Five series of experiments were conducted with varying hydraulic conditions and sediment properties. The results of these experiments are presented in this chapter.

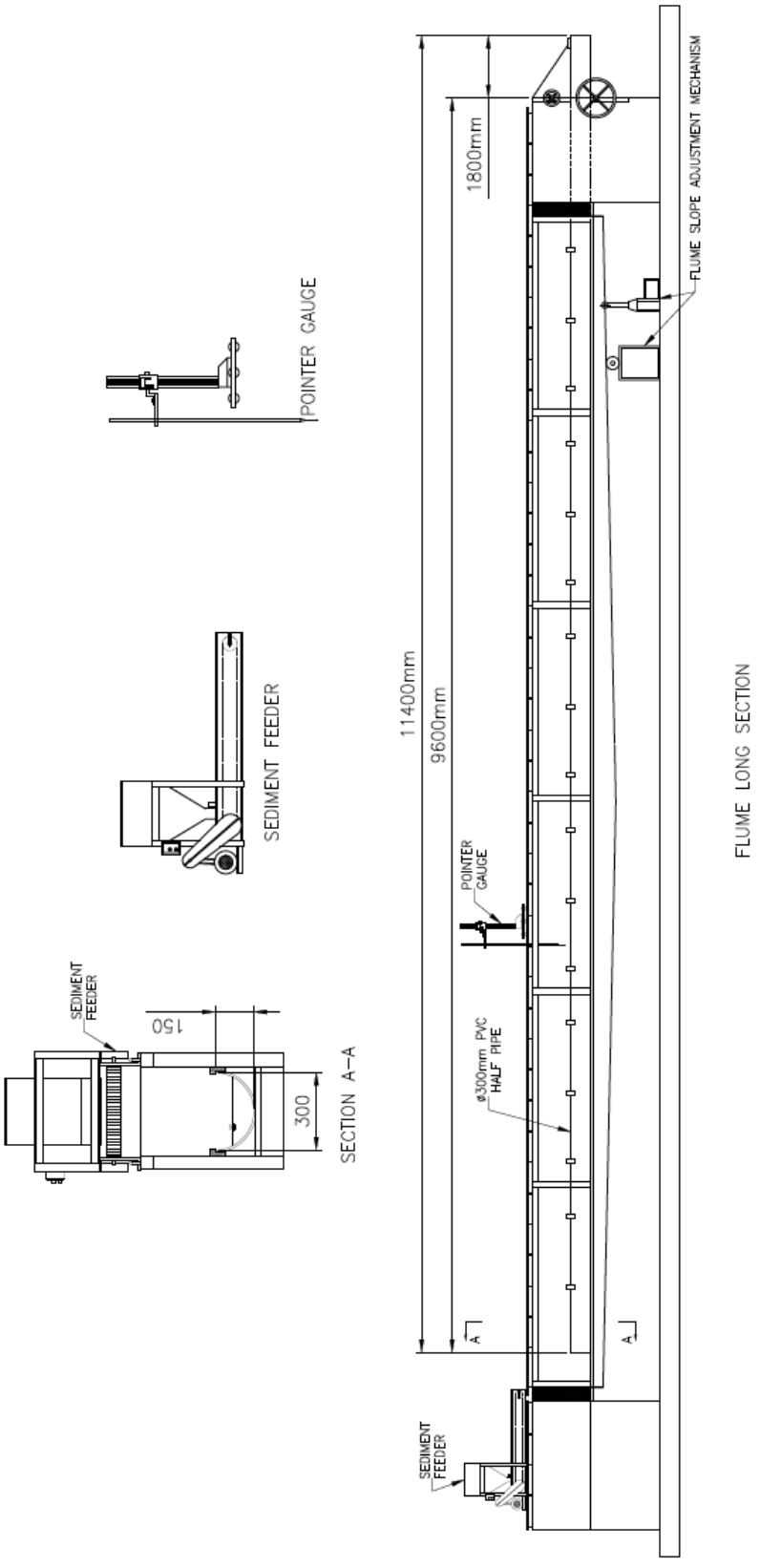


Figure 3-1 Diagram of experimental set up (not to scale)



Figure 3-2 PVC pipe positioned in flume



Figure 3-3 Side view of sediment feeder mounted on flume



Figure 3-4 Front view of sediment feeder mounted on flume



Figure 3-5 Flume outlet



Figure 3-6 Electronic pointer gauge

3.2 Active storage

The transition between discrete and collective modes of sediment transport are not easily observed owing to the formation and breaking up of bed forms during the transition stage. The difference in mass of active storage present in the flume for each mode allows for the transition value to be determined indirectly. Data were collected in order for a plot of active storage against sediment supply rate to be obtained. The trends for each mode are distinctly different and their intersection denotes the threshold between them.

Each experiment was carried out in the 11.4m long, 300mm diameter PVC half pipe in a tilting flume. For each series, combinations of discharge and bed slope were selected to achieve uniform flow through the pipe. The flow depth (D) was

confirmed to be uniform by measuring water surface levels using an electronic pointer gauge at regular intervals along the pipe. The discharge (Q) was measured using a turbine flow meter in the supply pipe and the slope adjusted using a hydraulic jack at the downstream end of the tilting flume. Sediment was supplied at the upstream end of the test section from a hopper feeding a conveyor belt.

Two different sediment densities were tested; quartz (ρ_s = 2650kg/m^3) and coal (ρ_s = 1550kg/m^3). Four grades of quartz and one grade of coal were tested, all angular and well-sorted. The median grain size of each grade was determined through sieve analyses (see Appendix A for raw data) and the sediment settling velocity (w) was determined from Graf and Acaroglu (1968). The series of experiments are listed in Table 3.1.

Table 3-1 Experimental conditions for modes of bed load transport laboratory investigation

Series	Sediment type	d_{50} (mm)	w (m/s)	S	Q (l/s)	D (mm)	q_{sbm} (g/s)
A1	Quartz	2.00	0.201	0.0030	18.6	48.8	4.40
A2	Quartz	2.00	0.201	0.0040	15.5	40.7	5.10
A3	Quartz	2.00	0.201	0.0034	10.8	36.3	3.60
B1	Quartz	0.85	0.105	0.0040	13.9	39.5	3.90
B2	Quartz	0.85	0.105	0.0032	19.9	48.3	3.30
B3	Quartz	0.85	0.105	0.0028	24.8	60.9	3.70
C1	Coal	0.92	0.052	0.0010	15.5	57.6	3.20
C2	Coal	0.92	0.052	0.0012	17.1	59.9	3.40
D1	Quartz	0.70	0.093	0.0027	24.8	57.5	2.97
D2	Quartz	0.70	0.093	0.0030	21.7	52.5	3.56
E1	Quartz	1.40	0.130	0.0031	18.6	47.7	4.80
E2	Quartz	1.40	0.130	0.0035	10.8	35.4	3.60

For each experiment, the sediment was supplied at a constant rate until an equilibrium condition was reached. This was confirmed by ensuring that the rate of sediment mass being supplied to the flume was the same as that exiting the flume, which were measured by collecting the sediment at the input and exit sections for predefined durations and weighing the sediment. Once it was confirmed that an equilibrium condition was reached, the sediment supply was abruptly stopped and the sediment remaining in the flume was collected and

weighed. This is the deposited active storage. The active storage was plotted against the sediment supply rate for each series of experiments. The results are plotted for each series in Figure 3-7 to 3-11.

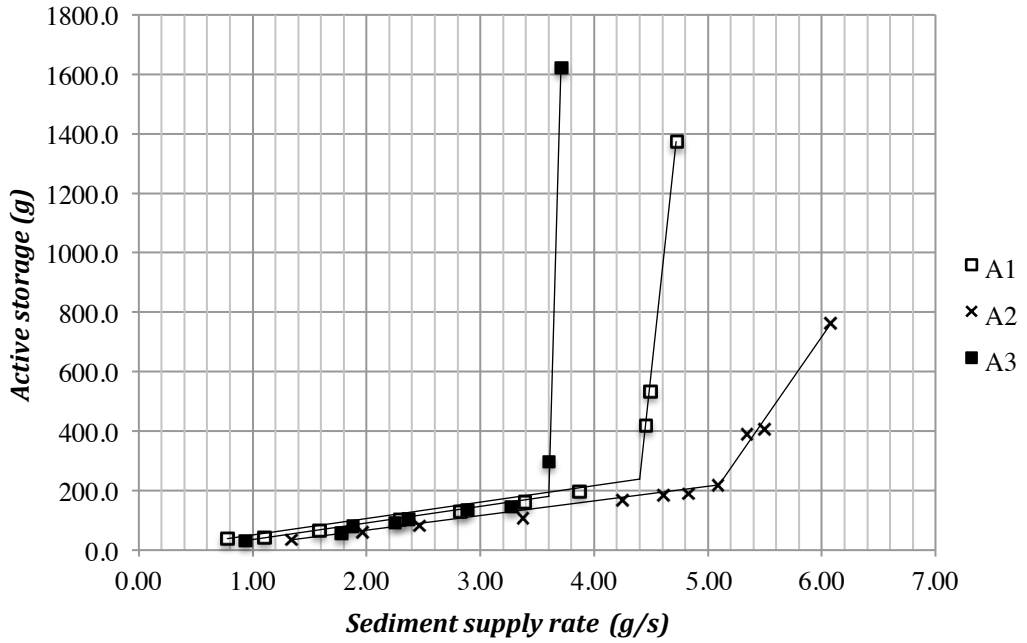


Figure 3-7 Active storage in discrete and collective modes of bed load transport for Series A experiments (quartz sediment)

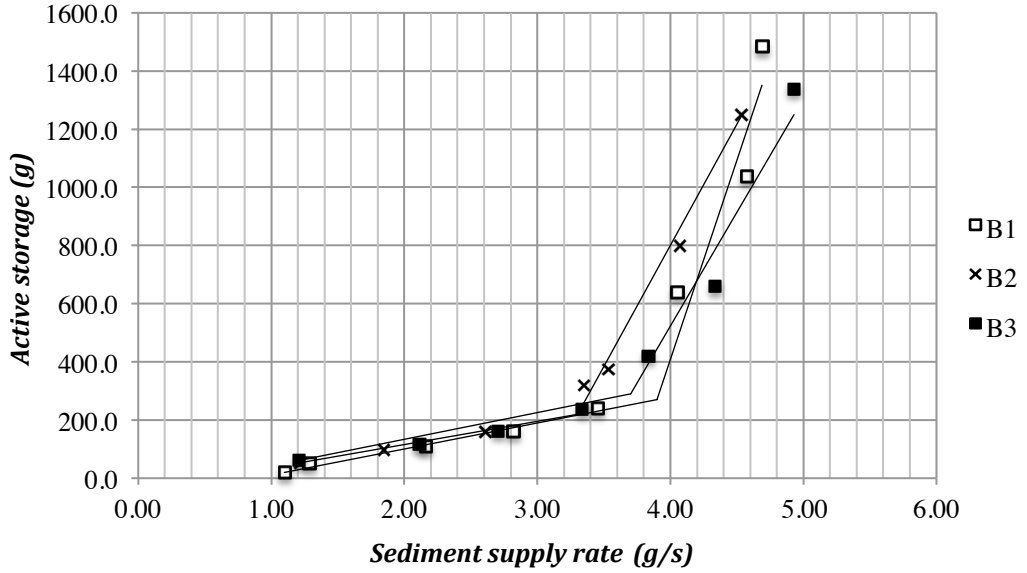


Figure 3-8 Active storage in discrete and collective modes of bed load transport for Series B experiments (quartz sediment)

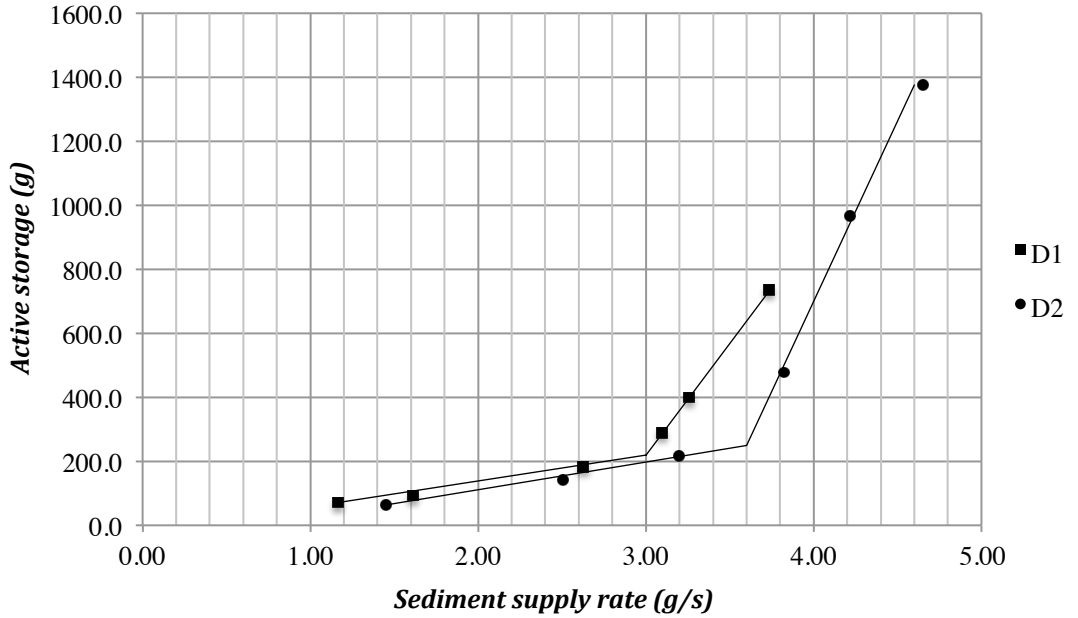


Figure 3-9 Active storage in discrete and collective modes of bed load transport for Series D experiments (quartz sediment)

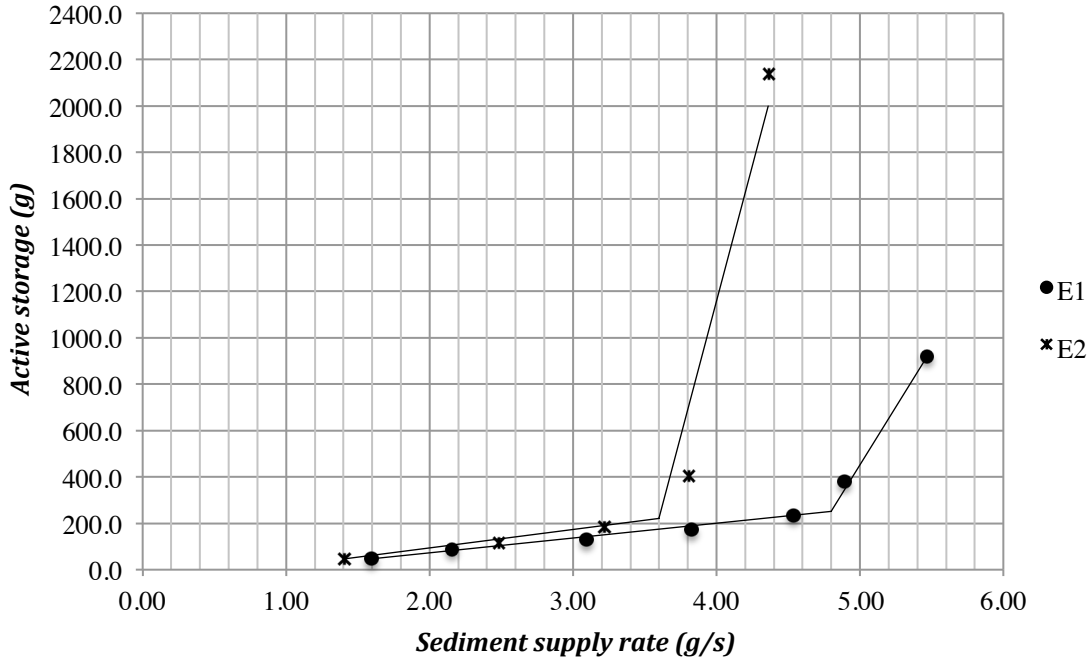


Figure 3-10 Active storage in discrete and collective modes of bed load transport for Series E experiments (quartz sediment)

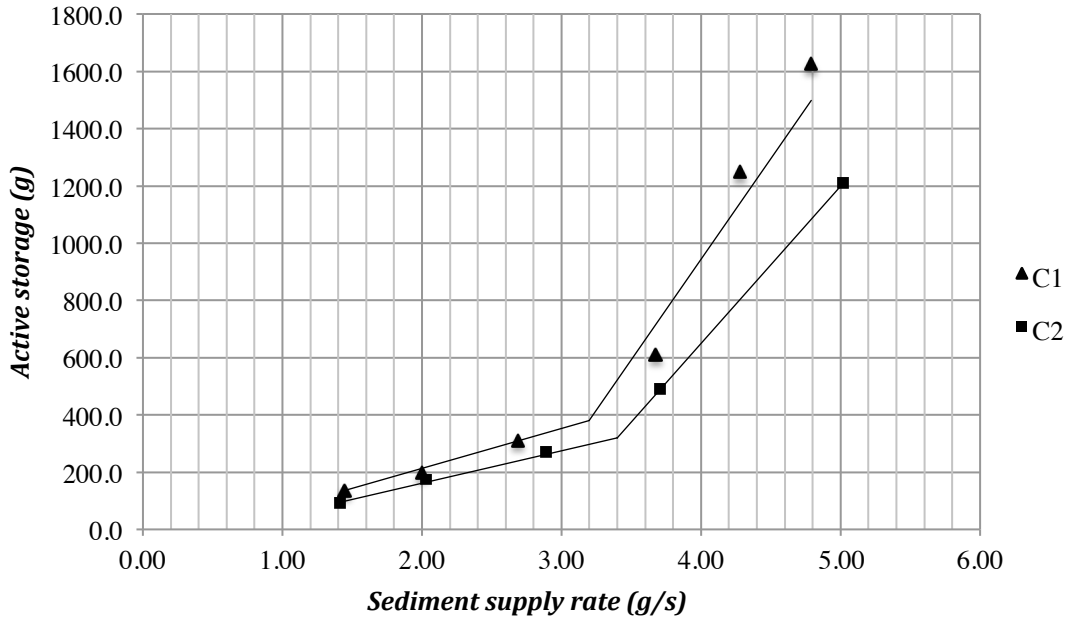


Figure 3-11 Active storage in discrete and collective modes of bed load transport for Series C experiments (coal sediment)

For each series, distinct kinks are noticeable and represent the threshold between discrete and collective movement of bed load transport. Trend lines were fitted through the discrete data and through the collective data and the intersection of these trend lines was recorded as the threshold value of sediment transport rate. The threshold values (q_{sbm}) are listed in Table 3-1. Examples of discrete and collective modes of movement are shown in Figure 3-12 and 3-13 respectively.

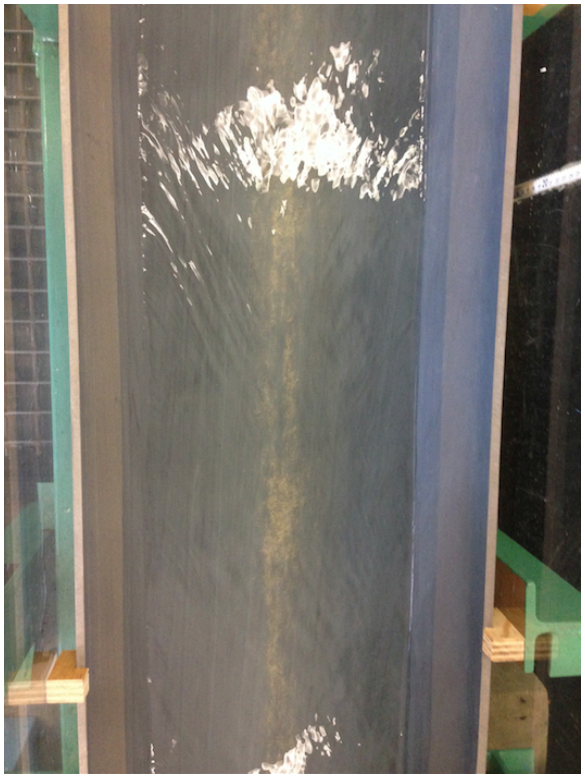


Figure 3-12 Discrete bed load movement

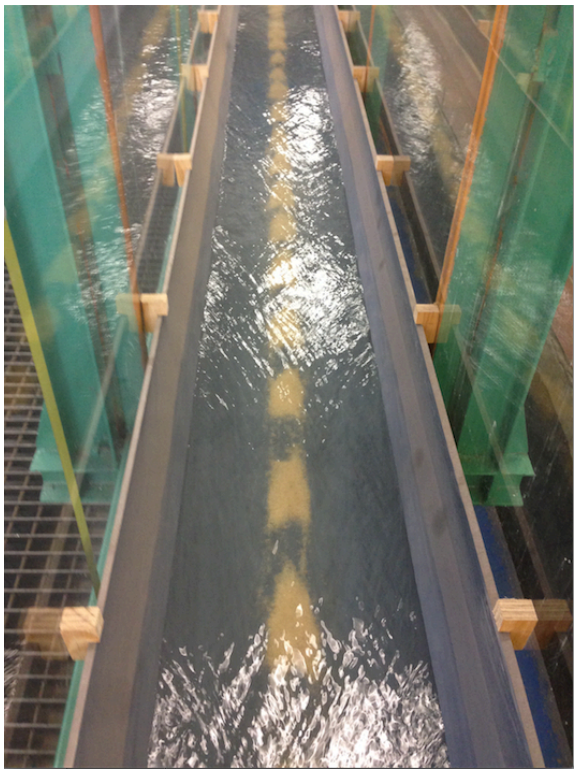


Figure 3-13 Collective bed load movement

3.3 Threshold relationships

The mode of movement is expected to be dependent on the following parameters (James et al., 2011):

- mean grain size, d_{50}
- shape of sediment particles, SF
- density of sediment, ρ_s
- sorting of sediment mixture, σ
- bed roughness, k_s
- density of water, ρ
- absolute viscosity of water, μ
- depth of flow, D
- flow velocity, U
- bed slope, S
- bed load transport rate, Q_s

A dimensional analysis was used in order to express the dependencies between the relevant variables. James et al. (2011) consider the relationship shown in equation 3-1 for smooth rectangular beds, which is adapted to the smooth pipe for this study.

$$\frac{u_*}{w} = f(q^*, R_*) \qquad (3\text{-}1)$$

where:

$$q^* = \frac{q_s}{\rho_s \sqrt{(S_s - 1) g {d_{50}}^3}}$$

$$R_* = \frac{\rho u_* d_{50}}{\mu}$$

where:

u_*/w = movability number

u_* = shear velocity

R_*	=	shear Reynolds number
w	=	sediment settling velocity
ρ_s	=	density of sediment
ρ	=	density of water
μ	=	absolute viscosity of water
g	=	acceleration due to gravity
q^*	=	dimensionless bed load parameter
q_s	=	unit width sediment supply rate
S_s	=	relative density of sediment
d_{50}	=	mean grain size

It should be noted that the width dimension used to calculate the unit-width bed load rate for the case of the smooth pipe was the width across the top of the flow, which varied for different flow depths.

Considering the relationship shown in equation 3-1, the threshold relationship is plotted in Figure 3-14 and can be expressed as equation 3-2.

$$\left(\frac{u_*}{w}\right)_{bm} = 0.0768 \ln\left(\frac{q^*}{R_*}\right) + 0.759 \qquad (3\text{-}2)$$

with $R^2 = 0.977$.

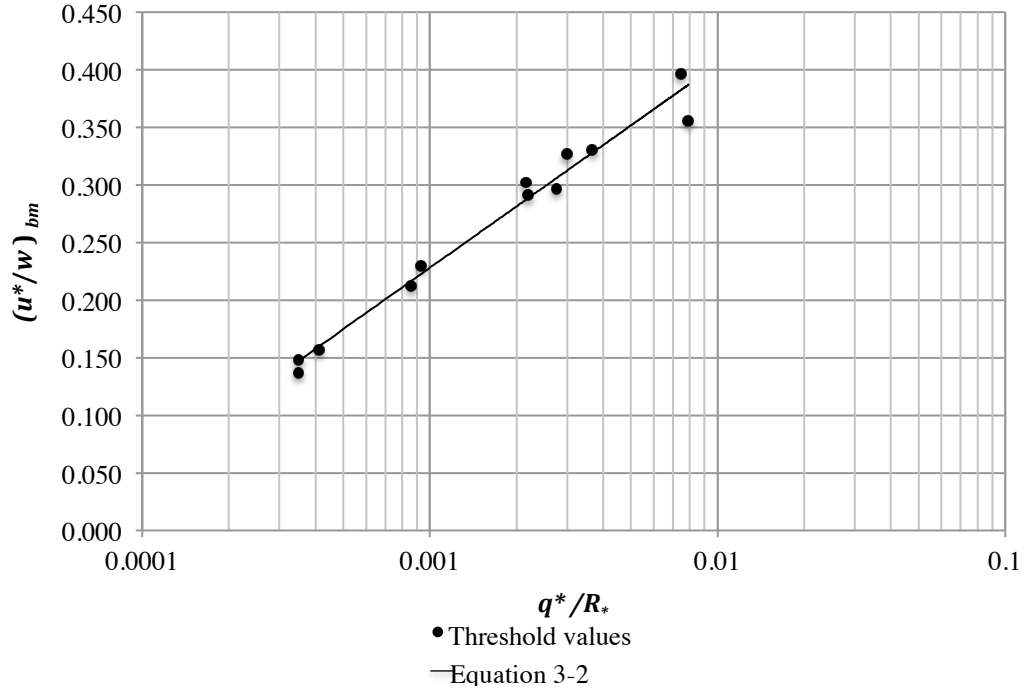


Figure 3-14 Threshold conditions between discrete and collective bed load movement

The data were compared to the data obtained in the experiments by James et al. (2011) for the threshold relationship for the case of a smooth flat bed. The results are plotted in Figure 3-15. The relationship describing the combination of the data can be expressed as equation 3-3.

$$\left(\frac{u_*}{w}\right)_{bm} = 0.0719 \ln\left(\frac{q^*}{R_*}\right) + 0.722 \qquad (3\text{-}3)$$

with $R^2 = 0.974$

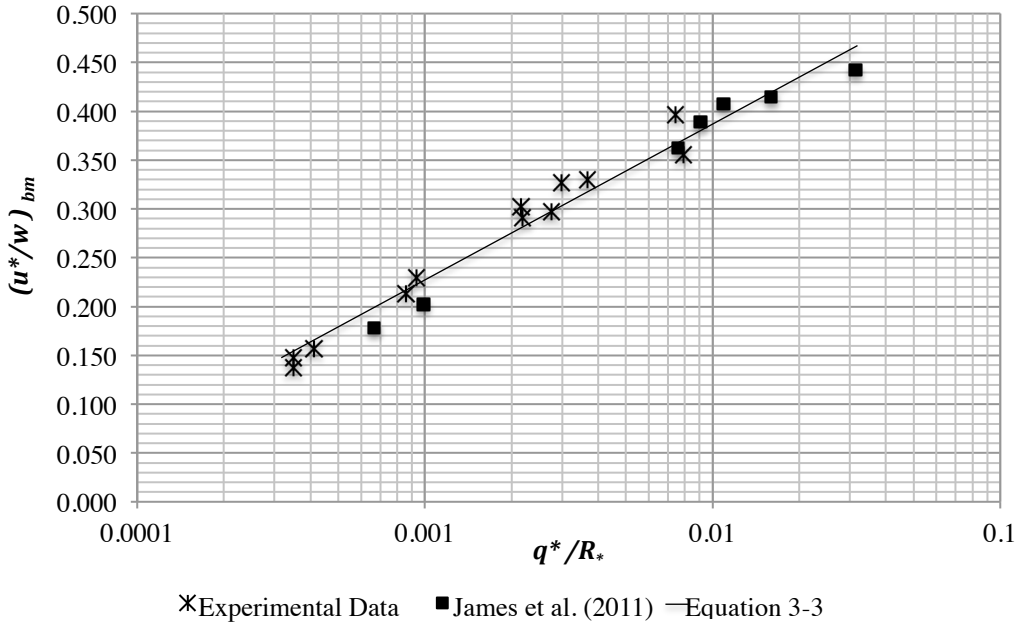


Figure 3-15 Threshold conditions between discrete and collective movement for experimental data and James et al. (2011) data

The data obtained for sediment flow through a smooth pipe in this study fits the relationship proposed by James et al. (2011) well. This indicates that the mode of bed load transport is independent of the shape of the channel bed.

3.4 Conclusion

The data presented for the threshold conditions for a pipe show a distinct trend. The correlation between the pipe flow data and the rectangular channel data indicate the independence of the transport mode on the shape of the channel bed. This allowed for the data to be combined and applied more broadly.

4 ACTIVE STORAGE PREDICTION

4.1 Introduction

It is important for the quantity of active storage in a pipe to be known for practical reasons. For short duration flushing flows the active storage is not completely removed and becomes a residual deposit on the pipe bed. As residual deposits remain stationary, they begin to consolidate and gain strength greater than for unconsolidated particles. The consolidated sediment deposit then requires a higher boundary shear stress and velocity to be transported. This may incur unnecessary costs. The residual sediment also acts to reduce the available flow area of a pipe and thus its capacity. This may result in surcharging stormwater drains or sewers. These effects are highly undesirable due to the harmful environmental, economic and social impacts.

The quantity of active storage is significantly different for the different modes of sediment transport. The effective velocity of the sediment movement may be the same for a case of sediment being transported discretely and a case of sediment being transported in bed forms however the active storage will differ significantly. For this reason, it is important to determine the mode of bed load transport and thus the quantity of active storage required to be transported.

All the data considered in Chapter 3 were manipulated to determine whether a relationship exists allowing the quantity of active storage to be predicted. The data consisted of 174 data points. Dimensionless relationships were analysed considering the full data set, the quartz and coal data sets separately and the discrete and collective data sets separately.

The relationship presented in this chapter is compared to the data presented in James et al. (2011) and their correlation discussed.

4.2 Active storage relationships

A relationship was determined between the active storage concentration (C_{AS}) and the ratio of applied sediment supply rate to the critical sediment supply rate

(q_s/q_{crit}). The critical sediment supply rate is the parameter that can be determined from equation 3-3. This dimensionless relationship can be defined by equation 4-1 and is presented in Figure 4-1.

$$C_{AS} = 87.0x10^{-6}e^{3.09\left(\frac{q_S}{q_{crit}}\right)} \quad (4\text{-}1)$$

with $R^2 = 0.849$.

where C_{AS} is the active storage concentration in the flume, defined as:

$$C_{AS} = \frac{V_s}{V_w} \quad (4\text{-}2)$$

where:

V_s = volume of sediment

V_w = volume of water

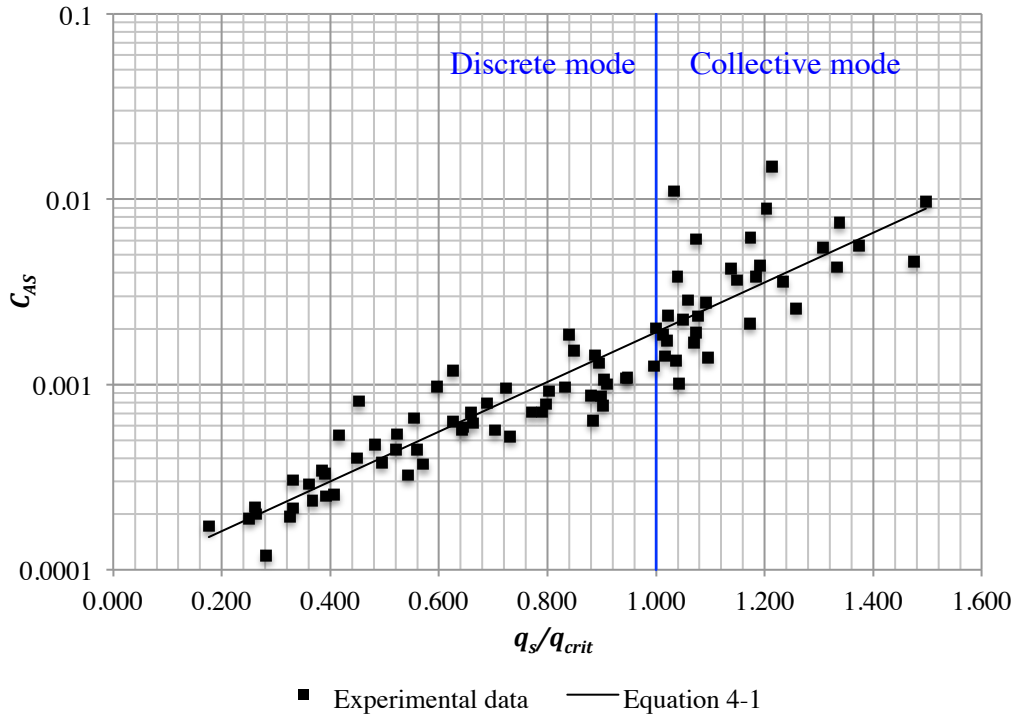


Figure 4-1 Dimensionless relationship for active storage

The data show good correlation between the ratio of applied supply rate and critical supply rate and the active storage concentration. As the transport mode passes the threshold, bed forms start to form and may change the flow depth along the channel length. The presence of bed forms along the channel causes a changing roughness value to be experienced by the flow. These effects have not been considered in this study therefore equation 4-1 is considered less reliable for $q_s/q_{crit} > 1$, i.e. the collective mode shown on Figure 4-1. For this case, the flow depth, and thus the volume of water (V_w), would be underestimated resulting in an overestimation of the volume of active storage (V_s). Although this may be a conservative prediction, the active storage relationship is unknown within the collective mode of transport.

The data were compared to that of James et al. (2011) and the results are presented in Figure 4-2.

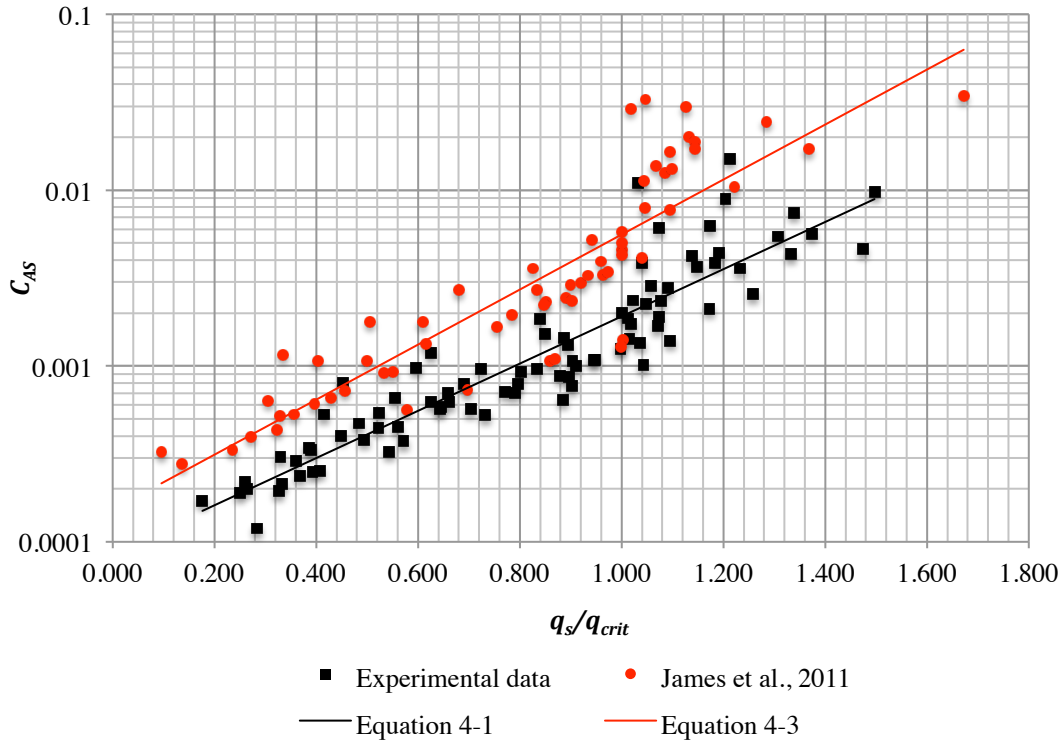


Figure 4-2 Dimensionless relationship for active storage compared to James et al. (2011) data

The dimensionless relationship for the James et al. (2011) data can be defined by equation 4-3.

$$C_{AS} = 153x10^{-6}e^{3.60\left(\frac{q_S}{q_{crit}}\right)} \quad (4\text{-}3)$$

with $R^2 = 0.794$.

The data from this study follow a similar trend to the James et al. (2011) data. The difference in equations 4-1 and 4-3 may be accounted for by the different bed shapes, suggesting that equation 4-1 is applicable to pipes and equation 4-3 is applicable to flat beds. The comparison indicates that a flat bed can accommodate a higher sediment transport rate than a pipe.

The dimensionless relationship describing the combination of the flat bed and pipe data can be defined by equation 4-4 and is presented in Figure 4-3.

$$C_{AS} = 112x10^{-6}e^{3.29\left(\frac{q_S}{q_{crit}}\right)} \quad (4\text{-}4)$$

with $R^2 = 0.700$.

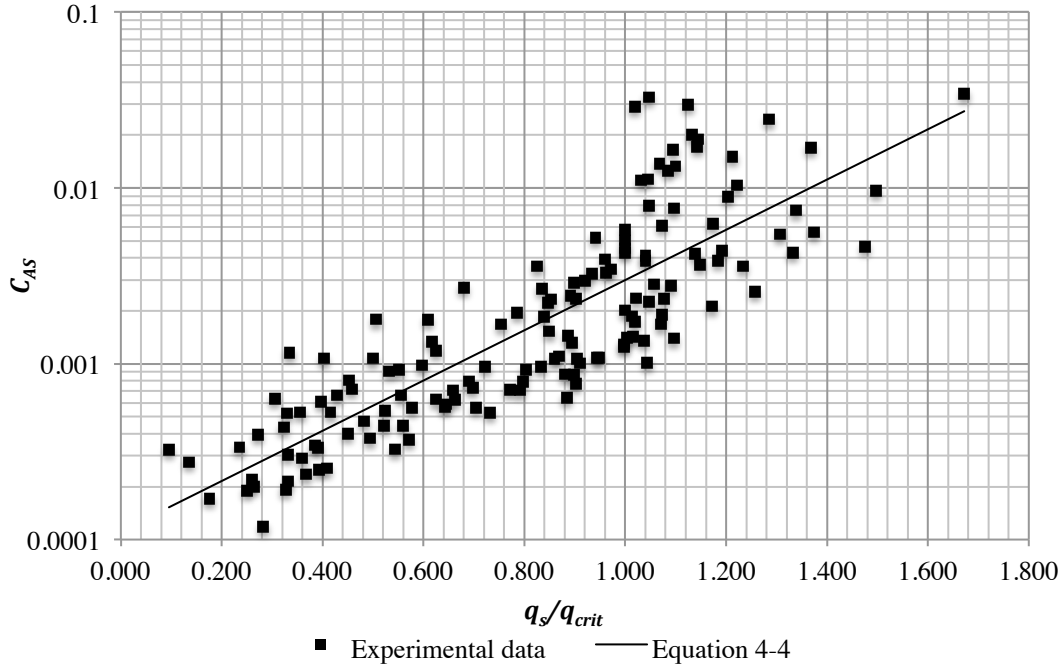


Figure 4-3 Dimensionless relationship for active storage for combined data

More scatter and a lower R^2 value suggest that the individual relationships are more applicable to each case. It is predicted that equations with similar gradients between equations 4-1 and 4-3 (such as equation 4-4) would define the dimensionless relationship for active storage for larger pipe diameters than the one used in this study. This would occur as the bed of the pipe would tend to flatten out as the diameter increases.

4.3 Application

The application of these results depends on the parameters required to be determined. The simultaneous application of equations 3-3 and 4-1 (for a pipe) or 4-3 (for a flat bed) allow for all necessary parameters to be determined either directly or by iteration. Detailed design guidelines are presented in Chapter 6.

4.4 Conclusion

The quantity of active storage required to be flushed from a pipe is important in determining effective self-cleansing flows. Equation 4-1 is presented as a dimensionless relationship for the prediction of the active storage volume in a pipe. Equation 4-3 is a similar relationship, presented for the prediction of the active storage volume on a flat bed. Both equations are dependent on the sediment supply rate and hydraulic conditions of the flow.

5 FLUSHING DURATIONS FOR SELF-CLEANSING FLOWS

5.1 Introduction

Current design practices stipulate a minimum velocity criterion in order to achieve no deposited sediment in the pipe. The duration for which this velocity should be achieved is not considered in design practices. In the experiments described in Chapter 3, the flow velocity remained constant for each series however the mode of bed load transport differed showing that for a given velocity the volume of active sediment storage in the pipe may vary significantly dependent on the sediment supply rate, sediment properties and hydraulic conditions of the flow. The differing average sediment velocities of the two transport modes suggest that a longer duration of the same velocity may be required to transport the sediment present in the pipe as bed forms. This is the focus of this chapter.

Experiments were performed in Flume B of the hydraulics laboratory, using the 300mm diameter, PVC half pipe as described in Chapter 3. Four series of experiments were conducted with varying hydraulic conditions and sediment properties (Table 5-1). The results of these experiments are presented in this chapter.

5.2 Results

The aim of this series of experiments was to investigate the effects of modes of bed load transport on the required duration to achieve a clean pipe. A combination of flow rate and slope achieving uniform flow conditions was selected and remained constant for each series of experiments. The uniform flow was confirmed by measuring the flow depth along the pipe at 1.0m intervals using an electronic pointer gauge. The sediment was supplied to the system through a sediment feeder at the upstream end of the tilting section of the flume in which the 300mm diameter half pipe was positioned. For each data point, it was first ensured that an equilibrium condition was achieved by collecting and weighing the input and output sediment masses. Once this was achieved, the sediment

feeder was switched off abruptly and the time taken for all the active sediment storage to be flushed from the pipe was measured.

Two sediment densities were tested; quartz (ρ_s = 2650kg/m^3) and coal (ρ_s = 1550kg/m^3). Four grades of quartz were tested, the median grain size of which were determined through sieve analyses and the sediment fall velocity (w) was determined using the graph proposed by Graf and Acaroglu (1968). The experiments considering the coal sediment proved to be difficult. No clear end of sediment travelling through the pipe could be defined and for this reason, the coal data are excluded from the study. The series of experiments tested are listed in Table 5.1.

Table 5-1 Experimental conditions for time investigation

Series	**Sediment Type**	d_{50} **(mm)**	w **(m/s)**	S	Q **(L/s)**	D **(mm)**
T1	Quartz	2.00	0.201	0.0030	18.6	48.9
T2	Quartz	0.85	0.105	0.0040	13.9	39.3
T3	Quartz	0.70	0.093	0.0027	24.8	57.2
T4	Quartz	1.40	0.130	0.0035	9.3	34.3

The results of this set of experiments are presented in Figure 5-1 to Figure 5-4. For each series of experiments tested, the threshold sediment supply rate was determined using the equation proposed in Chapter 3 for both the pipe data only (equation 3-2) and the combined pipe and flat bed data (equation 3-3). These values are marked on Figure 5-1 to Figure 5-4 and the values are presented in Table 5-2.

Table 5-2 Threshold sediment supply rates

Series	q_{sbm} **(g/s) (equation 3-2)**	q_{sbm} **(g/s) (equation 3-3)**
T1	4.46	4.36
T2	3.41	3.79
T3	3.51	4.02
T4	3.19	3.30

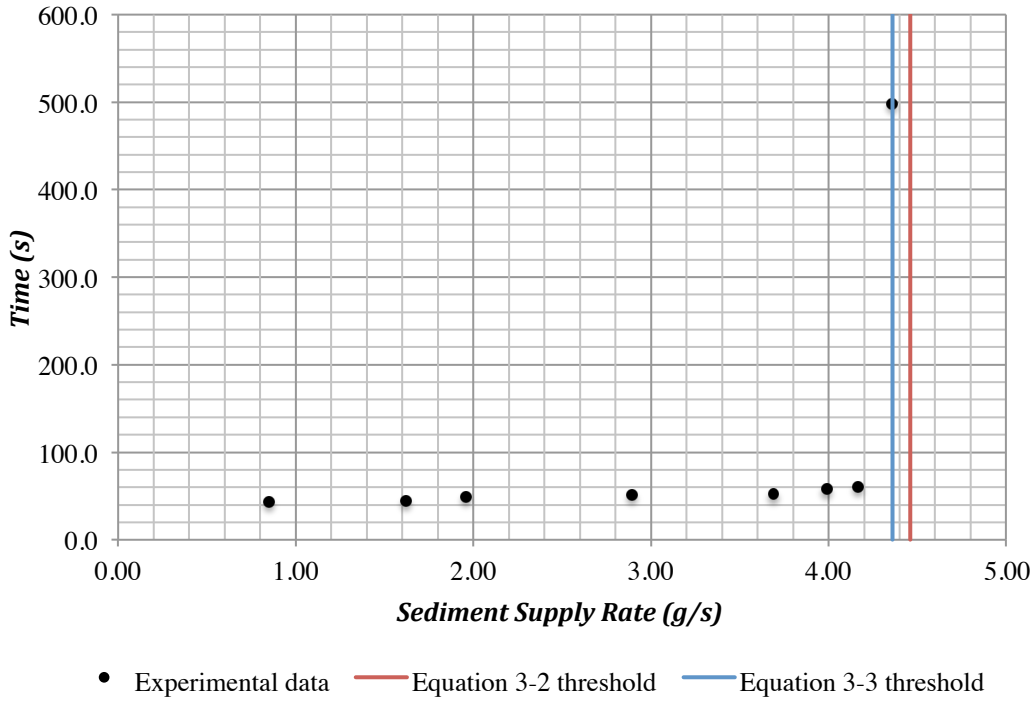


Figure 5-1 Time taken for active storage in Series T1 to be cleared

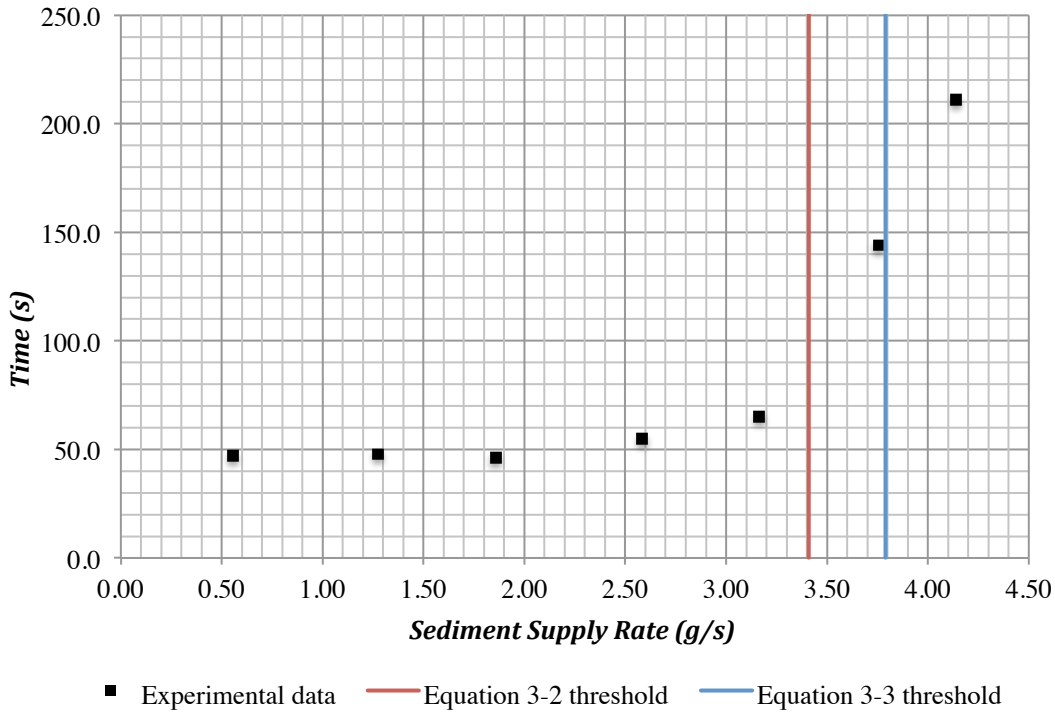


Figure 5-2 Time taken for active storage in Series T2 to be cleared

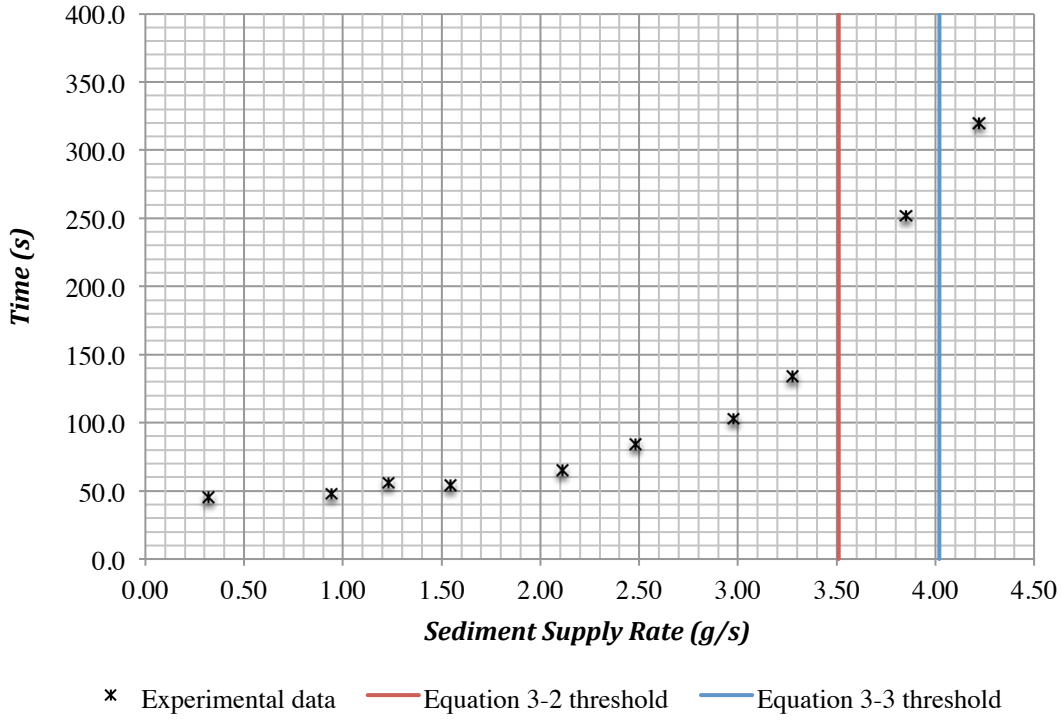


Figure 5-3 Time taken for active storage in Series T3 to be cleared

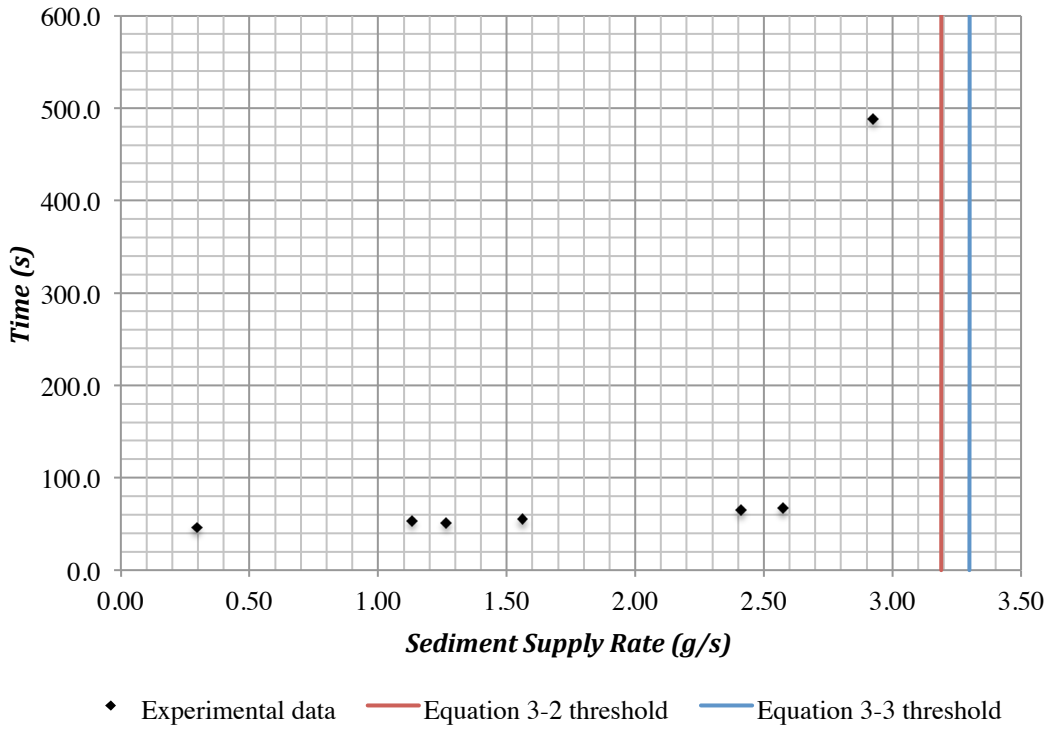


Figure 5-4 Time taken for active storage in Series T4 to be cleared

Two trends are noticeable from Figure 5-1 to Figure 5-4. When the mode of bed load transport is discrete, the time taken to clear all the active storage from the pipe is near to constant. This occurs as the sediment particles are all travelling at the same speed and few collisions are occurring. As the mode approaches collective movement, more particle collisions occur thus reducing the average velocity of the sediment. For all cases, the clearing duration for the collective mode of bed load transport increases dramatically.

It is observed that the dramatic increase in time taken to clear the sediment for Series 4 occurs before the threshold value determined by equation 3-2 and equation 3-3 which may be due to the sensitivity of equations 3-2 and 3-3 to the flow depth in calculating the critical sediment supply rate. A difference of 2mm in measured flow depth may result in differences of critical sediment supply rates of up to 15%. The results of a sensitivity analysis are presented in Table 5-3 and 5-4.

This sensitivity analysis, along with the possible errors in measurement readings indicate the importance of considering appropriate safety factors in design methodologies.

Table 5-3 Sensitivity of critical sediment supply rate to flow depth considering equation 3-2

Flow depth	D	D **- 2mm**		D **+ 2mm**	
Series	q_{sbm} **(g/s) (equation 3-2)**	q_{sbm} **(g/s) (equation 3-2)**	**% change in** q_{sbm}	q_{sbm} **(g/s) (equation 3-2)**	**% change in** q_{sbm}
T1	4.46	4.15	7.0	4.78	7.2
T2	3.41	2.96	13.0	3.90	14.3
T3	3.51	3.19	9.3	3.86	9.9
T4	3.19	2.80	12.3	3.62	13.3

Table 5-4 Sensitivity of critical sediment supply rate to flow depth considering equation 3-3

Flow depth	D	D - 2mm		D + 2mm	
Series	q_{sbm} (g/s) (equation 3-3)	q_{sbm} (g/s) (equation 3-3)	% change in q_{sbm}	q_{sbm} (g/s) (equation 3-3)	% change in q_{sbm}
T1	4.36	4.04	7.2	4.68	7.5
T2	3.79	3.28	13.6	4.36	15.0
T3	4.02	3.63	9.7	4.43	10.4
T4	3.30	2.88	12.7	3.75	13.9

5.3 Analysis of results

The time relationships recorded during this set of experiments indicate that the duration required to flush all the sediment from the flume in the discrete transport mode is significantly less than would be required for the collective mode. Within the discrete mode, the flushing duration is approximately constant at the duration at which a single particle would take to travel the distance of the flume. This occurs as the particles travel individually. As the transport mode approaches the collective mode, particles begin colliding with each other and thus reduce the average velocity of the sediment. This results in a longer duration being required to flush all sediment from the flume.

For sediments with a range of particle sizes, individual particles will travel with different velocities and begin to collide before the critical sediment supply rate is reached. The values at which this phenomenon would begin to occur would depend on the grain size distribution. The sediments tested in this study have a narrow range of particle sizes and this effect is not considered. Safety factors may be used in design to account for this.

The results of this study suggest that the discrete mode of transport is desirable when considering flushing flows. The slopes of the curves describing the relationship between sediment supply rate and time during the collective mode of transport are particularly steep. For a small increase in sediment supply rate in this zone, the flushing duration may increase significantly. This sensitivity introduces

a high level of uncertainty into design and is not recommended. For this reason, flushing flows in the discrete transport mode are recommended.

To determine a general relationship for predicting the required flushing durations for the design velocities determined through the results of Chapters 3 and 4, the experimental results of series T were analysed (see Appendix G for experimental readings). In the discrete zone, the average sediment velocity is considered to be constant, suggesting that the duration is independent of the sediment supply rate. From the experimental data, a duration prediction relationship was unable to be determined.

To predict the duration for which the design velocity is required to be applied, the sediment grain velocity is required as this corresponds to the average velocity of the active storage in the pipe. Existing methods for predicting the grain velocity were tested and compared to the experimental readings. The following methods were considered:

- Julien and Bounvilay (2013) equation for smooth flat beds

$$v_P = u_* \left\{ 6.5 \tan^{-1}\left(\frac{u_* d_{50}}{26\nu}\right) + \frac{1}{2\kappa} \ln\left[1 + \left(\frac{u_* d_{50}}{26\nu}\right)^2\right] \right\} \quad (5\text{-}1)$$

where:

v_P	=	grain velocity
u_*	=	shear velocity
d_{50}	=	mean grain size
κ	=	von Karman constant
ν	=	kinematic viscosity

- Cheng and Emadzadeh (2014) equation for smooth flat beds

$$\chi = C_3 R_* \sqrt{\theta} \quad (5\text{-}2)$$

$$\chi = \frac{v_P}{\sqrt{\Delta g d_{50}}} \quad (5\text{-}3)$$

$$\theta = \frac{{u_*}^2}{\Delta g d_{50}} \tag{5-4}$$

where:

C_3	=	0.27
χ	=	dimensionless grain velocity
R_*	=	shear Reynolds number
θ	=	dimensionless bed shear stress
v_P	=	grain velocity
Δ	=	effective specific density
g	=	acceleration due to gravity
u_*	=	shear velocity
d_{50}	=	mean grain size

The methods presented by Julien and Bounvilay (2013) and Cheng and Emadzadeh (2014) are empirical equations based on data from experiments on a flat bed channel. Although this study deals with pipes, the data from Chapter 3 are found to fit the flat bed data for the threshold relationships by James et al. (2011). Therefore, these grain velocity methods are considered here for the determination of an initial estimate of the flushing duration.

The results of the predicted grain velocities compared to the measured velocities are presented in Table 5-5. The percentage errors in velocity predictions are presented in Table 5-6.

Table 5-5 Comparison of experimental grain velocities and predicted grain velocities

	Experimental	**Eqn 5-1**	**Eqn 5-2**
Series	v_P **(m/s)**		
T1	0.250	0.292	0.478
T2	0.213	0.188	0.221
T3	0.226	0.154	0.173
T4	0.229	0.211	0.281

Table 5-6 Percentage errors in predicting grain velocities

	Eqn 5-1	Eqn 5-2
Series	**Error (%)**	
T1	16.7	90.9
T2	-11.8	4.0
T3	-32.0	-23.4
T4	-7.7	22.6

The results suggest that equation 5-1 proposed by Julien and Bounvilay (2013) most accurately predicts the grain velocity in the discrete mode of transport. For three of the four data points, equation 5-1 underestimates the grain velocity whereas equation 5-2 overestimates the grain velocity for three of the four data points. An underestimation of grain velocity would result in a higher flushing duration being stipulated and thus a conservative design. For these reasons, the method of Julien and Bounvilay (2013) was selected as a preliminary method for estimating the flushing duration.

5.4 Conclusion

The mode of bed load transport has an important effect of flushing durations. Significantly higher durations are recorded for the collective mode, thus making the discrete mode more effective in flushing active storage through pipes. The average velocity of sediment transport in the discrete mode is equivalent to the individual grain velocity and is used to determine the flushing duration.

6 DESIGN GUIDELINE AMENDMENTS

6.1 Introduction

The current design requirements for flushing flows define a single minimum velocity of 0.7m/s that must be achieved at a certain frequency of occurrence. This method does not account for the effects of the sediment characteristics or concentration, nor the hydraulic conditions of the flow. This study has shown that these factors should be considered and that the duration for which the flushing velocity should be maintained is important in achieving a clean pipe.

A set of design graphs, equations and design procedures are presented in this chapter. Two numerical examples are presented to illustrate the application of these procedures and to compare the results to those of the existing design requirements for flushing flows.

The methods described in this chapter are recommended to replace the current design methodology for flushing flows in smooth pipes and are applicable to conditions within the testing ranges of this study.

6.2 Design graphs and equations

The design graphs and equations presented in this section are to be used in conjunction with the design procedures in section 6-3. The relationships presented in this section are derived from experimental data and are therefore only known to be reliable within the ranges of conditions tested in this study. The test limits can be expressed as:

- $0.350x10^{-3} < \frac{q^*}{R_*} < 0.0314$ and
- $0.137 < \frac{u_*}{w} < 0.442$

Relationships presented in the design graphs as solid lines indicate experimentally derived relationships. Dotted lines indicate a predicted extrapolation of the data.

6.2.1 Critical sediment supply rate

To determine the critical sediment supply rate, the threshold relationship (equation 3-3) is manipulated and q_{crit} is solved for directly. The manipulated equation is presented in equation 6-1.

$$q_{crit} = \frac{u_* \rho_s}{\nu} \sqrt{(S_s - 1) g {d_{50}}^5}\, e^{\left(\frac{13.9 u_*}{w} - 10.0\right)} \tag{6-1}$$

where:

u_*	=	$\sqrt{gRS}$
q_{crit}	=	critical sediment supply rate
ρ_s	=	density of sediment
ν	=	kinematic viscosity of water
S_s	=	relative density of sediment
g	=	acceleration due to gravity
d_{50}	=	mean grain size
D	=	flow depth
w	=	particle settling velocity
R	=	hydraulic radius
S	=	pipe slope

The computed critical sediment supply rate is the maximum supply rate that the flow conditions can transport in the discrete transport mode.

6.2.2 Minimum flow depth

The determination of the minimum flow depth is an intermediate calculation leading to the calculation of the minimum flushing flow rate (Q) required to transport a given sediment supply rate. The dimensionless relationship shown to fit the data best can be defined by equation 6-2 and is presented in Figure 6-1.

$$\frac{D}{d_{50}} = 2.09 \left(\frac{q_s}{\rho_s \sqrt{g {d_{50}}^3 S}} \right) + 3.10 \tag{6-2}$$

with $R^2 = 0.962$.

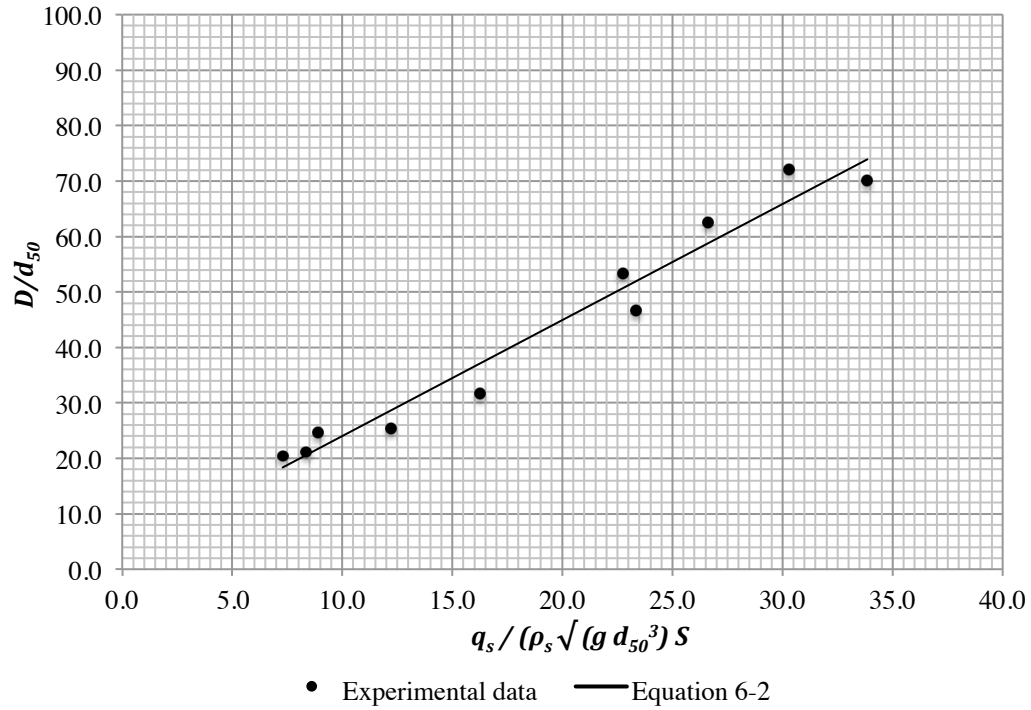
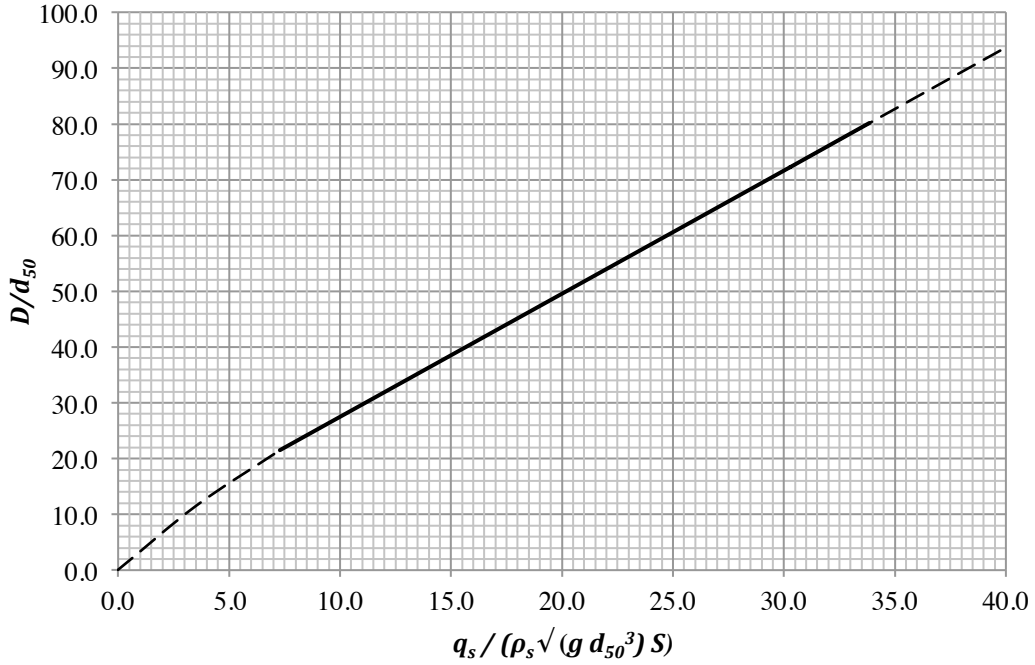


Figure 6-1 Dimensionless relationship for minimum flow depth

The form of this relationship shows that the value of the minimum flow depth is independent of the pipe diameter. This agrees with the conclusion presented in Chapter 3 that the bed load transport is independent of the shape of the bed and thus the diameter of the pipe.

The relationship presented in Figure 6-1 is extrapolated to produce the minimum flow depth design graph presented in Figure 6-2. The variable definitions are included with the graph for ease of use.



D	=	flow depth
d_{50}	=	mean grain size
q_s	=	unit width sediment supply rate
ρ_s	=	density of sediment
g	=	acceleration due to gravity
q_s	=	unit width sediment supply rate
S	=	pipe slope

Figure 6-2 Minimum flow depth design graph

The $\frac{D}{d_{50}}$ value can be read off the graph and multiplied by the mean grain size (d_{50}) to determine the minimum required flow depth. A resistance equation such as the Manning equation or the Darcy-Weisbach equation can be used with this flow depth to determine the minimum flushing flow rate.

6.2.3 Flushing duration

A relationship was determined in order to predict the required minimum flushing duration for which the flushing flow rate should be applied in order to remove all the sediment in the pipe. This relationship is defined by equation 6-3 and is presented in Figure 6-3.

$$t = 5118\left(\frac{\mu}{U\,\rho_s\,d_{50}}\right) + 1.51 \qquad (6\text{-}3)$$

with $R^2 = 0.979$.

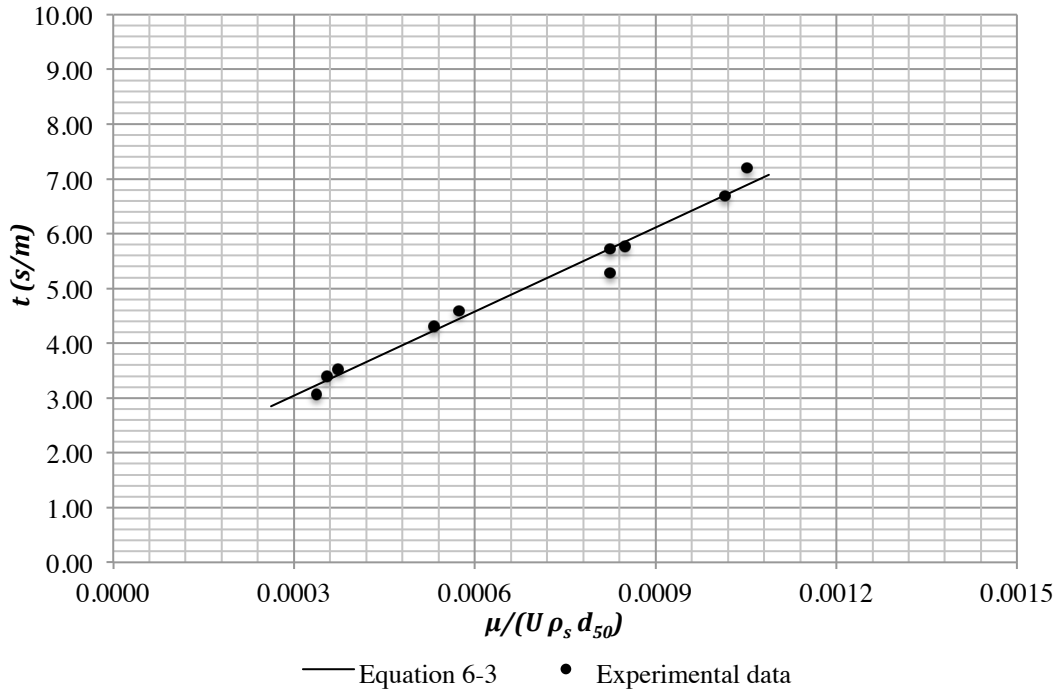
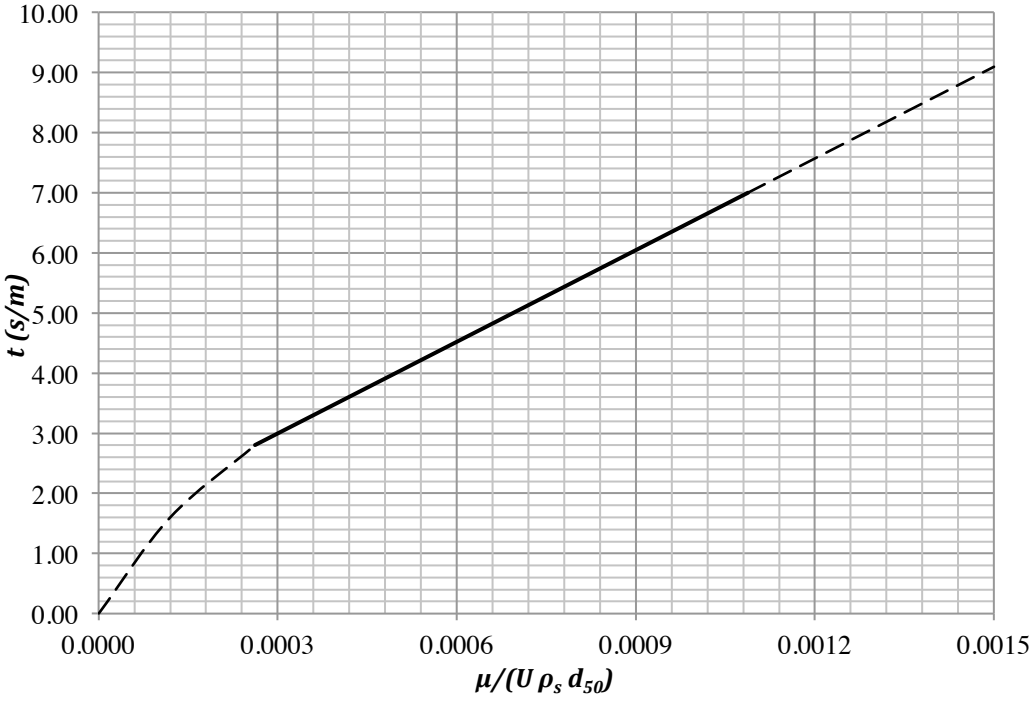


Figure 6-3 Flushing duration relationship

This relationship was determined by relating the results of equation 5-1 by Julien and Bounvilay (2013) to experimental conditions.

The relationship presented in Figure 6-3 is extrapolated to produce the minimum flushing duration design graph presented in Figure 6-4. The variable definitions are included with the graph for ease of use.



t	=	flushing duration per metre length
μ	=	absolute viscosity of water
U	=	mean flow velocity
ρ_s	=	density of sediment
d_{50}	=	mean grain size

Figure 6-4 Minimum flushing duration design graph

The t value that can be read off the graph is the duration required to flush the sediment from 1m of pipe. Multiplying this value by the total pipe length gives the total required flushing duration.

6.2.4 Mass of active storage

To determine the mass of active storage, equation 4-1 is manipulated and the mass of active storage (m_{AS}) is solved for directly. The manipulated equation is presented in equation 6-4.

$$m_{AS} = 87.0x10^{-6}\, A\, \rho_s\, e^{3.09\left(\frac{q_s}{q_{crit}}\right)} \qquad (6\text{-}4)$$

where:

m_{AS} = mass of active storage

A	=	flow area
ρ_s	=	density of sediment
q_s	=	unit width sediment supply rate
q_{crit}	=	critical sediment supply rate

The computed mass of active storage is the total mass of active storage in 1m of pipe under given conditions. Multiplying this by the total length of pipe gives the total mass of active storage in the pipe.

6.3 Design procedures

6.3.1 Case 1: Unknown sediment supply rate and flushing discharge duration

The known properties for this case are:

- Density of sediment, ρ_s
- Mean grain size, d_{50}
- Sediment settling velocity, w
- Pipe diameter, d
- Pipe slope, S
- Flow rate, Q

It is required to determine the following:

- The maximum flushable sediment supply rate of a known sediment for a known discharge in a specified pipe within the discrete transport mode
- The flushing discharge duration
- The mass of active storage

Flushable sediment supply rate

The maximum flushable sediment supply rate can be determined by solving equation 6-1. This value represents the threshold condition and it is therefore

recommended that a value less than the critical supply rate be selected for design to ensure discrete movement of the sediment.

Flushing discharge duration

The required duration for which the known discharge should be applied can be approximated from the graph in Figure 6-4. The parameter $\frac{\mu}{U\ \rho_s\ d_{50}}$ can be determined from the known properties and the corresponding duration can be read off the graph as the time to clear 1m of pipe. Multiplying this by the total length of pipe will give the total required duration.

Mass of active storage

The mass of active storage can be calculated from equation 6-4. This equation computes the mass of active storage in 1m of the pipe. Multiplying this value by the length of the pipe gives the total mass of active storage in the pipe.

6.3.2 Case 2: Unknown flushing discharge and flushing discharge duration

The known properties for this case are:

- Density of sediment, ρ_s
- Mean grain size, d_{50}
- Sediment settling velocity, w
- Pipe diameter, d
- Pipe slope, S
- Unit width sediment supply rate, q_s

It is required to determine the following:

- The flushing discharge capable of transporting a known sediment supply rate in a specified pipe within the discrete transport mode
- The flushing discharge duration
- The mass of active storage

Flushing discharge

The flushing discharge is the minimum discharge at which a known sediment supply rate can be flushed from a pipe within the discrete transport mode. The minimum flow depth can be determined from the minimum flow depth design graph in Figure 6-2. The value $\frac{q_s}{\rho_s \sqrt{g {d_{50}}^3}\, S}$ can be computed from the known sediment properties and bed slope, and the corresponding $\frac{D}{d_{50}}$ value can be read off the graph. This value is then multiplied by the mean grain size (d_{50}) to obtain the minimum flow depth.

The minimum flow depth can be used with a resistance equation to determine the minimum flushing discharge required to transport the sediment supply rate in the discrete mode.

Flushing discharge duration

The required duration for which the calculated flushing discharge should be applied can be approximated from the graph in Figure 6-4. The parameter $\frac{\mu}{U\, \rho_s\, d_{50}}$ can be determined from the known properties and the corresponding duration can be read off the graph as the time to clear 1m of pipe. Multiplying this by the total length of pipe will give the total required duration.

Mass of active storage

The mass of active storage can be calculated from equation 6-4. This equation computes the mass of active storage in 1m of the pipe. Multiplying this value by the length of the pipe gives the total mass of active storage in the pipe.

6.4 Numerical examples

6.4.1 Case 1

The known properties are:

ρ_s = 2650kg/m^3

d_{50} = 1.70mm

w = 0.180m/s

d = 300mm

S = 0.0032

Q = 0.00581m^3/s

n = 0.01

L = 40m

From this data, the following geometric properties are determined:

D = 0.0580m

A = 0.00959m^2

P = 0.273m

R = 0.0351m

Determine the maximum flushable sediment supply rate from equation 6-1:

$$u_* = \sqrt{gRS}$$

$$u_* = \sqrt{9.81(0.0351)(0.0032)}$$

$$\underline{u_* = 0.0332\text{m/s}}$$

$$q_{crit} = \frac{u_* \rho_s}{\nu} \sqrt{(S_s - 1) g {d_{50}}^5}\, e^{\left(\frac{13.9 u_*}{w} - 10.0\right)} \qquad (6\text{-}1)$$

$$q_{crit} = \frac{(0.0332)(2650)}{1e^{-6}} \sqrt{\left(\frac{2650}{1000} - 1\right)(9.81)(0.00170)^5}\, e^{\left(\frac{13.9(0.0332)}{0.180} - 10.0\right)}$$

$$\underline{q_{crit} = 0.0241\text{kg/s/m}}$$

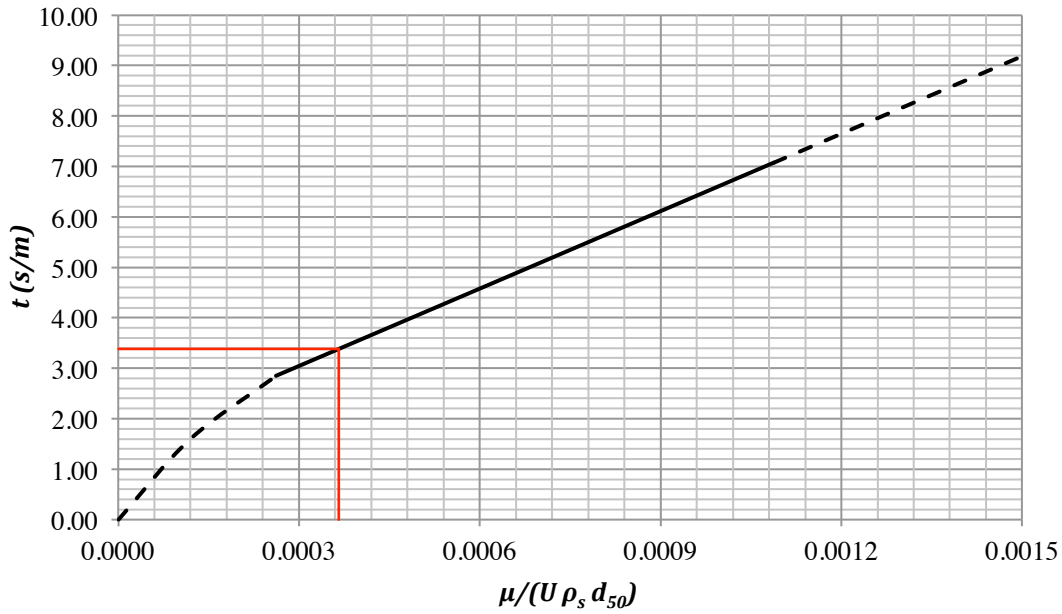
Determine the flushing discharge duration from Figure 6-4:

$U = \frac{1}{n} R^{2/3} S^{1/2}$ (Manning equation)

$$U = \frac{1}{0.01}(0.0351)^{2/3}(0.0032)^{1/2}$$

$$\underline{U = 0.606\text{m/s}}$$

$$\frac{\mu}{U\,\rho_s\,d_{50}} = \frac{1e^{-3}}{0.606\,(2650)\,(0.00170)} = 0.000366$$



t	=	flushing duration per metre length
μ	=	absolute viscosity of water
U	=	mean flow velocity
ρ_s	=	density of sediment
d_{50}	=	mean grain size

Figure 6-4 Minimum flushing duration design graph

From the graph, $t = 3.4\text{s/m}$

$\therefore t_{40m} = 3.4(40)$

$\therefore t_{40m} = 136\text{s}$

Determine total mass of active storage from equation 6-4:

Assume q_s/q_{crit} = 1 to produce the maximum active storage for this example.

$$m_{AS} = 87.0x10^{-6}\,A\,\rho_s\,e^{3.09\left(\frac{q_s}{q_{crit}}\right)} \qquad (6\text{-}4)$$

$$m_{AS} = 87.0x10^{-6}\,(0.00959)\,(2650)\,e^{3.09(1)}$$

$m_{AS} = 0.0486\text{kg/m}$

The total mass of active storage:

$\therefore m_T = 0.0486\ (40)$

$\underline{\therefore m_T = 1.94\text{kg}}$

Therefore, the maximum flushable sediment supply rate in the discrete zone for a discharge of 0.00581m^3/s is 0.0241kg/s/m. The discharge must be applied for 136s to completely flush the sediment from the pipe. The maximum mass of active storage in the pipe is 1.94kg.

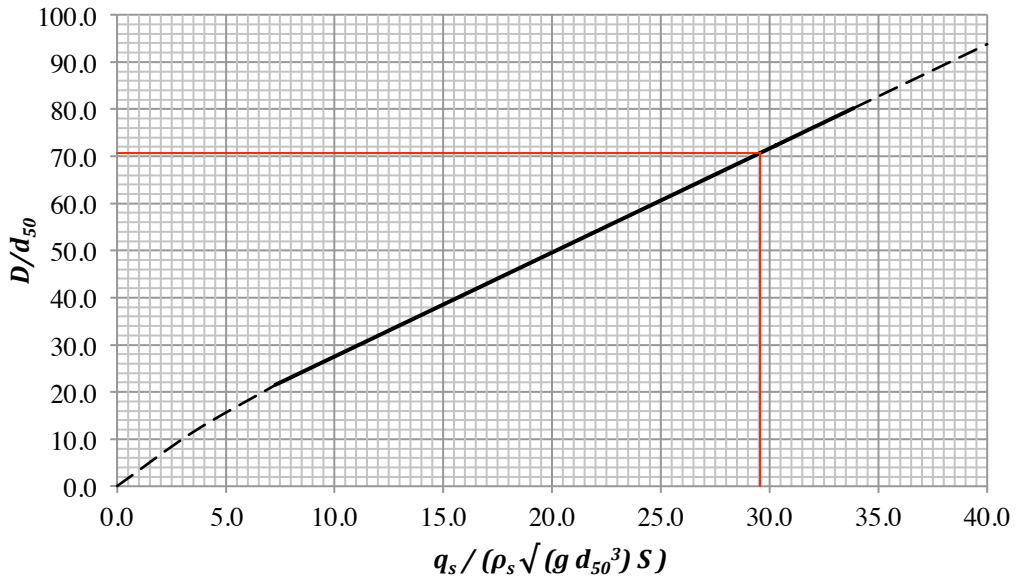
6.4.2 Case 2

The known properties are:

ρ_s = 2650kg/m^3
d_{50} = 0.85mm
w = 0.105m/s
d = 300mm
S = 0.0025
n = 0.01
L = 150m
q_s = 0.0152kg/s/m

Determine the minimum flow depth from Figure 6-2.

$$\frac{q_s}{\rho_s \sqrt{g d_{50}{}^3}\, S} = \frac{0.0152}{2650 \sqrt{(9.81)(0.00085)^3}\, 0.0025} = 29.6$$



D	=	flow depth
d_{50}	=	mean grain size
q_s	=	unit width sediment supply rate
ρ_s	=	density of sediment
g	=	acceleration due to gravity
q_s	=	unit width sediment supply rate
S	=	pipe slope

Figure 6-2 Minimum flow depth design graph

From the graph, $\frac{D}{d_{50}} = 70.5$

, $\therefore D = 70.5\ (0.00085)$

, $\underline{\therefore D = 0.060\text{m}}$

From the calculated flow depth, the following geometric properties are determined:

$A = 0.0101\text{m}^2$

$P = 0.278\text{m}$

$R = 0.0362\text{m}$

Determine the minimum flushing discharge from the Manning equation:

$$Q = \frac{A}{n} R^{2/3} S^{1/2}$$

$$Q = \frac{(0.0101)}{0.01} (0.0362)^{2/3} (0.0025)^{1/2}$$

$$\underline{Q = 0.00553 \text{m}^3/\text{s}}$$

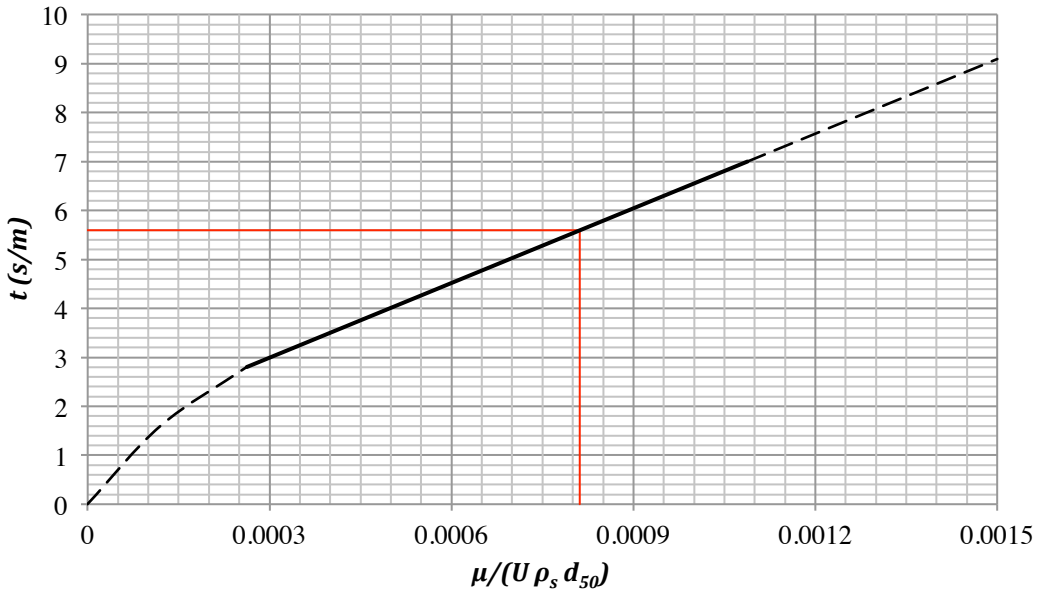
Determine the flushing discharge duration from Figure 6-4:

$$U = \frac{\text{Q}}{\text{A}}$$

$$U = \frac{0.00553}{0.0101}$$

$$\underline{U = 0.547 \text{m/s}}$$

$$\frac{\mu}{U\ \rho_S\ d_{50}} = \frac{1e^{-3}}{0.547\ (2650)\ (0.00085)} = 0.000812$$



t	=	flushing duration per metre length
μ	=	absolute viscosity of water
U	=	mean flow velocity
ρ_s	=	density of sediment
d_{50}	=	mean grain size

Figure 6-4 Minimum flushing duration design graph

From the graph, $t = 5.6\text{s/m}$

$\therefore t_{150m} = 5.6\ (150)$

$\underline{\therefore t_{150m} = 840\text{s}}$

Determine total mass of active storage from equation 6-4:

Assume q_s/q_{crit} = 1 to produce the maximum active storage for this example.

$$m_{AS} = 87.0x10^{-6}\ A\ \rho_s\ e^{3.09\left(\frac{q_s}{q_{crit}}\right)} \qquad (6\text{-}4)$$

$m_{AS} = 87.0x10^{-6}\ (0.0101)\ (2650)\ e^{3.09(1)}$

$\underline{m_{AS} = 0.0512\text{kg/m}}$

The total mass of active storage:

$\therefore m_T = 0.0512\ (150)$

$\underline{\therefore m_T = 7.68\text{kg}}$

Therefore, the minimum flushing discharge for a sediment supply rate of 0.0152kg/s/m transported in the discrete transport mode is 0.00553m^3/s. The discharge must be applied for 840s to completely flush the sediment from the pipe. The maximum mass of active storage in the pipe is 7.68kg.

6.5 Remarks

Comparing the minimum flow velocities in the numerical examples presented in section 6.4 with the existing design criterion of a minimum flow velocity of 0.7m/s suggests that the existing design criterion would be sufficient in flushing the sediment within the discrete mode of transport for each case. These examples fall within the testing ranges of this study. An extrapolation of the design graphs and equations suggests that the design criterion of a minimum velocity of 0.7m/s may not always be sufficient.

6.6 Conclusion

The design procedures presented in this chapter are presented as improvements to the current design methods for self-cleansing flows. The proposed methods provide a more holistic approach to self-cleansing pipes as the derivation of the results consider the effects of the sediment properties and concentration, hydraulic conditions and geometric properties whereas the previous design methods did not.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

Sediment deposition is undesirable in urban drainage systems owing to the harmful environmental, economical and social impacts. Self-cleansing design criteria were introduced to avoid these problems and are currently based on a minimum velocity criterion. This method was found to be inadequate in considering the effects influencing the behaviour of sediments transport. The primary purpose of this research has been to define a criterion to distinguish between discrete and collective bed load transport modes and consider the applications thereof. This research has contributed to the development of criteria and methodologies as improvements to the current self-cleansing design criteria and methodologies.

7.2 Experimental investigation of modes of bed load transport

The mode of bed load transport was found to significantly affect the self-cleansing flow criteria. To be able to account for these effects in design, a study was carried out to determine a criterion for predicting which mode of transport would occur for known sediment supply rates, hydraulic conditions and channel geometries.

Equation 3-1 was developed and presented as a new relationship for predicting the mode of bed load transport. The data were found to closely match those of James et al. (2011) for a flat bed suggesting that the mode of bed load transport is independent of the shape of the bed. The data were combined and a new relationship proposed as equation 3-3.

7.3 Active storage prediction

The importance of determining the quantity of active storage in a pipe was established with regards to defining self-cleansing design criteria.

Equation 4-1 was presented as a new relationship for predicting the volume of active storage present in a pipe. A similar relationship (equation 4-3) was presented for predicting the volume of active storage in a flat bed based on James et al. (2011) data. These relationships showed the volume of active storage to be dependent on the sediment supply rate and the hydraulic conditions of the flow.

The new relationships can be used with the threshold relationship proposed in Chapter 3 (equation 3-3) to determine the critical sediment supply rate and the volume of active storage for given sediment properties and hydraulic conditions.

7.4 Flushing durations for self-cleansing flows

Flushing durations are currently not considered in self-cleansing design methods. This research indicates the importance of defining a flushing duration in order for the flushing flow to be most effective. A method based on previous research is presented to determine the grain velocity and thus an estimate of the required flushing duration for flow in the discrete mode of transport.

7.5 Design guideline amendments

The current design criteria for self-cleansing flows in pipes is considered inadequate. This research has contributed to the improvement of the design criteria and methodologies describing flushing flows and durations.

A set of design guidelines is provided as an amendment to the current design method.

7.6 Recommendations for further research

This research report has produced methods that are usable for situations with sediment properties and hydraulic conditions within the ranges tested during the laboratory investigations. The relationships presented in this research are based on empiricism. Further extension of the database through additional laboratory experiments could strengthen the reliability of these relationships.

All the data used were produced through laboratory experiments. Confirmation of these data through field investigations would be valuable. The general applicability of the presented relationships to data considering different sediment densities, sizes and size grading should be further researched to extend its usage. It may also be useful to determine the applicability of the proposed relationships when considering an existing sediment volume in a pipe as opposed to a sediment being supplied to the pipe as in this study.

The methods provided are based only on experiments performed in a half pipe and are therefore only applicable to part full flowing pipes. Investigating the effects and relationships of modes of bed load transport in pipes flowing full would be valuable.

This research recommends that the transport rate should be within the discrete mode of transport to be most effective. Experimental limitations did not allow for the collective mode of transport to be extensively researched. Research into the average sediment velocity in the collective mode and a comparison between whether a high velocity and short duration (for the discrete mode) or a low velocity and high duration (for the collective mode) is more economical. This may be valuable for application in flat areas in which the required slope to achieve discrete movement cannot be achieved.

Further investigation into the relationships defining the collective mode of transport and the applicability of the proposed relationship to this mode would be useful. This may include the effects of bed forms on the flow depth and the bed roughness in the pipe channel.

Determining the flushing durations for the coal sediments proved challenging during this research. Further investigation into the flushing durations of sediments with lower densities than quartz may be valuable. Testing the applicability of the results presented in this report to sediments with higher density sediments would also be useful.

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APPENDIX A:

SIEVE ANALYSIS DATA

Sediment A

Mass of container	68.2	g
Mass of sample + container	1786.2	g
Mass of sample	1718.0	g

Sieve no.	**Sieve size (mm)**	**Sieve mass (g)**	**Sieve mass + mass retained (g)**	**Mass retained (g)**	**% retained**	**% passing**
4	4.750	455.0	455.0	0.0	0.00%	99.99%
8	2.360	483.2	727.1	243.9	14.20%	85.80%
16	1.180	437.2	1901.8	1464.6	85.25%	0.55%
28	0.600	354.6	362.3	7.7	0.45%	0.10%
40	0.425	333.8	334.6	0.8	0.05%	0.05%
50	0.300	329.4	329.6	0.2	0.01%	0.04%
100	0.150	301.1	301.3	0.2	0.01%	0.03%
200	0.075	291.1	291.3	0.2	0.01%	0.02%
pan	pan	358.0	358.3	0.3	0.02%	0.00%
				1717.9	***99.99%***	***-99.99%***

d_{50}	2.00	mm

Sediment B

Mass of container	68.3	g
Mass of sample + container	1640.3	g
Mass of sample	1572.0	g

Sieve no.	**Sieve size (mm)**	**Sieve mass (g)**	**Sieve mass + mass retained (g)**	**Mass retained (g)**	**% retained**	**% passing**
4	4.750	455.0	454.8	-0.2	-0.01%	99.96%
8	2.360	488.6	487.9	-0.7	-0.04%	100.01%
16	1.180	437.2	470.5	33.3	2.12%	97.89%
28	0.600	354.5	1850.1	1495.6	95.14%	2.75%
40	0.425	333.6	372.3	38.7	2.46%	0.29%
50	0.300	329.4	332.5	3.1	0.20%	0.09%
100	0.150	301.1	302.3	1.2	0.08%	0.01%
200	0.075	291.1	291.2	0.1	0.01%	0.01%
pan	pan	358.0	358.1	0.1	0.01%	0.00%
				1571.2	***99.95%***	***-99.95%***

d_{50}	0.85	mm

Sediment C

Mass of container	296.5	g
Mass of sample + container	1079.7	g
Mass of sample	783.2	g

Sieve no.	**Sieve size (mm)**	**Sieve mass (g)**	**Sieve mass + mass retained (g)**	**Mass retained (g)**	**% retained**	**% passing**
4	4.750	455.0	455.0	0.0	0.00%	99.62%
8	2.360	483.2	483.3	0.1	0.01%	99.60%
16	1.180	437.1	511.2	74.1	9.46%	90.14%
28	0.600	355.0	987.7	632.7	80.78%	9.36%
40	0.425	334.3	373.6	39.3	5.02%	4.34%
50	0.300	329.6	347.1	17.5	2.23%	2.11%
100	0.150	301.2	312.9	11.7	1.49%	0.61%
200	0.075	291.1	295.0	3.9	0.50%	0.11%
pan	pan	358.0	358.9	0.9	0.11%	0.00%
				780.2	***99.62%***	***-99.62%***

d_{50}	0.92	mm

Sediment D

Mass of container	68.4	g
Mass of sample + container	1217.6	g
Mass of sample	1149.2	g

Sieve no.	**Sieve size (mm)**	**Sieve mass (g)**	**Sieve mass + mass retained (g)**	**Mass retained (g)**	**% retained**	**% passing**
4	4.750	455.1	455.1	0.0	0.00%	99.78%
8	2.360	422.6	422.5	-0.1	-0.01%	99.79%
16	1.180	451.7	455.5	3.8	0.33%	99.46%
28	0.600	415.8	1243.8	828.0	72.05%	27.41%
40	0.425	349.2	506.6	157.4	13.70%	13.71%
50	0.300	329.3	443.2	113.9	9.91%	3.80%
100	0.150	301.0	341.8	40.8	3.55%	0.25%
200	0.075	291.0	293.9	2.9	0.25%	0.00%
pan	pan	358.2	358.2	0.0	0.00%	0.00%
				1146.7	***99.78%***	***-99.78%***

d_{50}	0.70	mm

Sediment E

Mass of container	199.9	g
Mass of sample + container	807.9	g
Mass of sample	608.0	g

Sieve no.	**Sieve size (mm)**	**Sieve mass (g)**	**Sieve mass + mass retained (g)**	**Mass retained (g)**	**% retained**	**% passing**
4	4.750	463.9	463.9	0.0	0.00%	98.95%
8	2.360	483.1	483.3	0.2	0.03%	98.91%
16	1.180	393.2	765.6	372.4	61.25%	37.66%
28	0.600	348.0	557.1	209.1	34.39%	3.27%
40	0.425	483.1	494.8	11.7	1.92%	1.35%
50	0.300	293.7	299.1	5.4	0.89%	0.46%
100	0.150	471.8	472.5	0.7	0.12%	0.35%
200	0.075	291.7	292.1	0.4	0.07%	0.28%
pan	pan	320.6	322.3	1.7	0.28%	0.00%
				601.6	***98.95%***	***-98.95%***

d_{50}	1.40	mm

APPENDIX B:

LISTING OF SERIES A EXPERIMENTAL DATA

Date 29/04/2014

Series A1

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	2.00	mm
Settling velocity, w	0.201	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.66	rad
Width across top	0.294	m
Flow area, A	0.00748	m^2
Wetted perimeter, P	0.249	m
Hydraulic radius, R	0.0300	m

Flume length	11.4	m
Slope, S	0.0030	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.0	
Flow regime	Supercritical	

Average flow depth, D	0.0488	m
Flow rate, Q	18.6	L/s
Flow rate, Q	0.0186	m^3/s
Velocity, U	2.486	m/s

Initial mass (Bucket and bag)		
Dry	356.8	g
Wet	503.8	g

q_{sbm}	4.40	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	93.0	104.2	116.2	120	38.7
2	132.5	156.8	148.6	120	42.9
3	190.2	220.0	228.7	120	65.7
4	275.7	316.1	322.0	120	100.6
5	339.3	380.3	394.4	120	128.4
6	407.0	439.6	443.1	120	160.7
7	534.8	600.9	597.9	120	420.7
8	464.7	521.6	538.5	120	197.6
9	566.8	626.5	574.5	120	1374.2
10	539.4	605.1	610.2	120	533.9

Date 30/04/2014

Series A2

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	2.00	mm
Settling velocity, w	0.201	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.51	rad
Width across top	0.257	m
Flow area, A	0.00575	m^2
Wetted perimeter, P	0.226	m
Hydraulic radius, R	0.0254	m

Flume length	11.4	m
Slope, S	0.0040	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.7	
Flow regime	Supercritical	

Average flow depth, D	0.0407	m
Flow rate, Q	15.5	L/s
Flow rate, Q	0.0155	m^3/s
Velocity, U	2.695	m/s

Initial mass (Bucket and bag)		
Dry	356.9	g
Wet	512.5	g

q_{sbm}	5.10	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	161.1	156.8	175.6	120	34.7
2	235.5	249.4	257.4	120	59.6
3	295.7	312.0	311.2	120	82.2
4	405.3	453.1	459.7	120	107.9
5	509.5	570.8	558.4	120	167.5
6	553.0	619.1	623.7	120	185.2
7	641.5	696.6	692.3	120	389.3
8	579.5	640.9	637.0	120	188.5
9	659.4	710.8	705.0	120	407.3
10	610.3	666.9	661.1	120	217.5
11	729.3	774.1	778.0	120	762.0

Date 30/04/2014

Series A3

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	2.00	mm
Settling velocity, w	0.201	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.42	rad
Width across top	0.237	m
Flow area, A	0.00487	m^2
Wetted perimeter, P	0.213	m
Hydraulic radius, R	0.0228	m

Flume length	11.4	m
Slope, S	0.0034	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	4.9	
Flow regime	Supercritical	

Average flow depth, D	0.0363	m
Flow rate, Q	10.8	L/s
Flow rate, Q	0.0108	m^3/s
Velocity, U	2.220	m/s

Initial mass (Bucket and bag)		
Dry	356.9	g
Wet	512.5	g

q_{sbm}	3.60	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	112.5	115.5	122.2	120	32.1
2	225.9	248.1	240.5	120	79.2
3	346.6	380.3	360.9	120	135.7
4	213.4	218.0	221.9	120	55.6
5	270.0	299.6	307.2	120	92.2
6	284.2	315.4	308.8	120	103.7
7	393.2	418.3	417.6	120	147.3
8	432.0	461.4	440.4	120	296.3
9	445.7	480.6	363.1	120	1620.1

APPENDIX C

LISTING OF SERIES B EXPERIMENTAL DATA

Date 06/05/2014

Series B1

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.85	mm
Settling velocity, w	0.105	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.49	rad
Width across top	0.251	m
Flow area, A	0.00551	m^2
Wetted perimeter, P	0.223	m
Hydraulic radius, R	0.0247	m

Flume length	11.4	m
Slope, S	0.0040	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.4	
Flow regime	Supercritical	

Average flow depth, D	0.0395	m
Flow rate, Q	13.9	L/s
Flow rate, Q	0.0139	m^3/s
Velocity, U	2.522	m/s

Initial mass (Bucket and bag)		
Dry	357.6	g
Wet	509.7	g

q_{sbm}	3.90	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	132.3	157.2	156.4	120	19.8
2	154.5	167.4	176.1	120	50.6
3	259.7	308.2	315.2	120	109.6
4	338.5	430.3	418.0	120	159.8
5	415.0	529.4	526.7	120	239.3
6	486.5	609.5	589.7	120	637.5
7	563.2	714.9	612.9	120	1483.7
8	549.2	691.9	625.6	120	1038.7

Date 06/05/2014

Series B2

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.85	mm
Settling velocity, w	0.105	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.65	rad
Width across top	0.291	m
Flow area, A	0.00737	m^2
Wetted perimeter, P	0.248	m
Hydraulic radius, R	0.0297	m

Flume length	11.4	m
Slope, S	0.0032	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.4	
Flow regime	Supercritical	

Average flow depth, D	0.0483	m
Flow rate, Q	19.9	L/s
Flow rate, Q	0.0199	m^3/s
Velocity, U	2.702	m/s

Initial mass (Bucket and bag)		
Dry	357.6	g
Wet	509.7	g

q_{sbm}	3.30	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	145.6	183.9	168.1	120	52.6
2	221.6	279.4	273.1	120	99.4
3	313.1	405.7	383.2	120	157.5
4	402.2	498.7	487.9	120	318.3
5	488.4	599.8	567.5	120	798.7
6	423.8	512.1	531.6	120	374.3
7	544.2	676.6	635.8	120	1250.4

Date 06/05/2014

Series B3

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.85	mm
Settling velocity, w	0.105	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.87	rad
Width across top	0.363	m
Flow area, A	0.0103	m^2
Wetted perimeter, P	0.281	m
Hydraulic radius, R	0.0367	m

Flume length	11.4	m
Slope, S	0.0028	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	4.6	
Flow regime	Supercritical	

Average flow depth, D	0.0609	m
Flow rate, Q	24.8	L/s
Flow rate, Q	0.0248	m^3/s

Initial mass (Bucket and bag)		
Dry	357.8	g
Wet	503.7	g

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	144.8	171.2	176.8	120	60.1
2	253.7	332.1	332.9	120	115.5
3	324.9	410.1	428.9	120	163.2
4	400.3	493.1	487.0	120	238.3
5	460.4	576.1	541.7	120	419.0
6	520.5	669.4	681.6	120	658.8
7	591.6	713.7	700.9	120	1338.2

APPENDIX D

LISTING OF SERIES C EXPERIMENTAL DATA

Date 19/05/2014

Series C1

Experimental Properties

Property		Units
Sediment type	Coal	
Mean grain size, d_{50}	0.92	mm
Settling velocity, w	0.052	m/s
Sediment density, ρ_s	1550	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.81	rad
Width across top	0.341	m
Flow area, A	0.00949	m^2
Wetted perimeter, P	0.272	m
Hydraulic radius, R	0.0349	m

Flume length	11.4	m
Slope, S	0.0010	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	3.1	
Flow regime	Supercritical	

Average flow depth, D	0.0576	m
Flow rate, Q	15.5	L/s
Flow rate, Q	0.0155	m^3/s
Velocity, U	1.633	m/s

Initial mass (Bucket and bag)		
Dry	358.5	g
Wet	515.1	g

q_{sbm}	3.20	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	173.5	220.6	254.0	120	135.8
2	240.1	316.1	337.5	120	198.7
3	322.5	434.8	458.1	120	311.4
4	441.0	603.9	559.9	120	611.7
5	287.4	387.2	359.2	60	1626.6
6	256.9	346.3	331.5	60	1248.5

Date 19/05/2014

Series C2

Experimental Properties

Property		Units
Sediment type	Coal	
Mean grain size, d_{50}	0.92	mm
Settling velocity, w	0.052	m/s
Sediment density, ρ_s	1550	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.85	rad
Width across top	0.356	m
Flow area, A	0.0100	m^2
Wetted perimeter, P	0.278	m
Hydraulic radius, R	0.0361	m

Flume length	11.4	m
Slope, S	0.0120	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	3.2	
Flow regime	Supercritical	

Average flow depth, D	0.0599	m
Flow rate, Q	17.1	L/s
Flow rate, Q	0.0171	m^3/s
Velocity, U	1.702	m/s

Initial mass (Bucket and bag)		
Dry	358.5	g
Wet	515.1	g

q_{sbm}	3.40	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	169.3	242.8	248.6	120	94.3
2	243.4	343.5	364.9	120	173.4
3	346.6	470.6	494.7	120	270.6
4	222.5	313.2	307.5	60	490.3
5	300.9	428.0	-515.1	60	817.9

APPENDIX E

LISTING OF SERIES D EXPERIMENTAL DATA

Date 22/05/2014

Series D1

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.70	mm
Settling velocity, w	0.093	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.81	rad
Width across top	0.340	m
Flow area, A	0.00947	m^2
Wetted perimeter, P	0.272	m
Hydraulic radius, R	0.0348	m

Flume length	11.4	m
Slope, S	0.0027	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.0	
Flow regime	Supercritical	

Average flow depth, D	0.0575	m
Flow rate, Q	24.8	L/s
Flow rate, Q	0.0248	m^3/s
Velocity, U	2.618	m/s

Initial mass (Bucket and bag)		
Dry	358.8	g
Wet	510.4	g

q_{sbm}	2.97	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	193.6	240.1	242.6	120	93.2
2	315.1	383.2	378.7	120	183.3
3	371.8	453.2	474.8	120	289.7
4	390.6	469.1	474.7	120	399.5
5	448.2	548.5	499.6	120	735.6
6	139.8	177.8	181.3	120	71.3

Date 22/05/2014

Series D2

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.70	mm
Settling velocity, w	0.093	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.73	rad
Width across top	0.313	m
Flow area, A	0.00831	m^2
Wetted perimeter, P	0.259	m
Hydraulic radius, R	0.0321	m

Flume length	11.4	m
Slope, S	0.0030	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.1	
Flow regime	Supercritical	

Average flow depth, D	0.0525	m
Flow rate, Q	21.7	L/s
Flow rate, Q	0.0217	m^3/s
Velocity, U	2.611	m/s

Initial mass (Bucket and bag)		
Dry	358.8	g
Wet	510.4	g

q_{sbm}	3.56	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	174.1	225.7	222.6	120	63.7
2	300.5	378.8	390.2	120	142.0
3	383.5	455.4	484.8	120	216.9
4	458.4	561.5	538.3	120	476.9
5	558.4	672.8	652.0	120	1375.5
6	505.8	595.1	582.4	120	964.6

APPENDIX F

LISTING OF SERIES E EXPERIMENTAL DATA

Date 28/05/2014

Series E1

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	1.40	mm
Settling velocity, w	0.130	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.64	rad
Width across top	0.288	m
Flow area, A	0.00723	m^2
Wetted perimeter, P	0.246	m
Hydraulic radius, R	0.0294	m

Flume length	11.4	m
Slope, S	0.0031	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.2	
Flow regime	Supercritical	

Average flow depth, D	0.0477	m
Flow rate, Q	18.6	L/s
Flow rate, Q	0.0186	m^3/s
Velocity, U	2.573	m/s

Initial mass (Bucket and bag)		
Dry	358.8	g
Wet	510.4	g

q_{sbm}	4.80	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	191.4	218.9	224.9	120	46.8
2	258.8	303.0	309.1	120	87.3
3	371.4	422.1	430.8	120	128.6
4	459.1	506.6	556.7	120	172.1
5	544.1	632.8	622.5	120	236.0
6	655.9	741.4	658.3	120	920.2
7	587.1	659.9	645.3	120	378.4

Date 28/05/2014

Series E2

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	1.40	mm
Settling velocity, w	0.130	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.40	rad
Width across top	0.233	m
Flow area, A	0.00470	m^2
Wetted perimeter, P	0.211	m
Hydraulic radius, R	0.0223	m

Flume length	11.4	m
Slope, S	0.0035	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.2	
Flow regime	Supercritical	

Average flow depth, D	0.0354	m
Flow rate, Q	10.8	L/s
Flow rate, Q	0.0108	m^3/s
Velocity, U	2.300	m/s

Initial mass (Bucket and bag)		
Dry	358.8	g
Wet	510.4	g

q_{sbm}	3.60	g/s

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Active storage (g)
1	168.7	191.2	198.0	120	47.0
2	297.9	342.1	347.7	120	112.7
3	386.5	443.7	433.5	120	185.8
4	457.0	525.1	464.2	120	403.9
5	523.7	577.0	463.8	120	2138.0

APPENDIX G

LISTING OF SERIES T EXPERIMENTAL DATA

Date 22/09/2014

Series T1

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	2.00	mm
Settling velocity, w	0.201	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.66	rad
Width across top	0.294	m
Flow area, A	0.00750	m^2
Wetted perimeter, P	0.249	m
Hydraulic radius, R	0.0301	m

Flume length	11.4	m
Slope, S	0.0030	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.0	
Flow regime	Supercritical	

Average flow depth, D	0.0489	m
Flow rate, Q	18.6	L/s
Flow rate, Q	0.0186	m^3/s
Velocity, U	2.479	m/s

Initial mass (Bucket and bag)		
Dry	359.6	g
Wet	518.5	g

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Clearance time (s)
1	102.1	93.3	98.6	120	38.0
2	194.7	202.1	178.4	120	45.0
3	235.2	223.8	253.8	120	49.0
4	347.3	363.2	379.2	120	51.0
5	442.9	466.0	458.5	120	52.0
6	522.8	540.8	493.1	120	498.0
7	478.8	496.4	521.1	120	58.0
8	500.1	523.8	532.8	120	61.0

Date 23/09/2014

Series T2

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.85	mm
Settling velocity, w	0.105	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.48	rad
Width across top	0.250	m
Flow area, A	0.00546	m^2
Wetted perimeter, P	0.222	m
Hydraulic radius, R	0.0246	m

Flume length	11.4	m
Slope, S	0.0040	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.5	
Flow regime	Supercritical	

Average flow depth, D	0.0393	m
Flow rate, Q	13.9	L/s
Flow rate, Q	0.0139	m^3/s
Velocity, U	2.545	m/s

Initial mass (Bucket and bag)		
Dry	359.6	g
Wet	518.5	g

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Clearance time (s)
1	153.0	173.2	178.3	120	48.0
2	222.9	256.2	261.5	120	46.0
3	310.0	350.1	362.7	120	55.0
4	379.6	438.5	445.6	120	65.0
5	450.7	528.9	520.9	120	144.0
6	496.5	571.0	540.3	120	211.0
7	66.8	75.6	62.0	120	47.0

Date 26/09/2014

Series T3

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	0.70	mm
Settling velocity, w	0.093	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.81	rad
Width across top	0.338	m
Flow area, A	0.00939	m^2
Wetted perimeter, P	0.271	m
Hydraulic radius, R	0.0346	m

Flume length	11.4	m
Slope, S	0.0027	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	5.1	
Flow regime	Supercritical	

Average flow depth, D	0.0572	m
Flow rate, Q	24.8	L/s
Flow rate, Q	0.0248	m^3/s
Velocity, U	2.642	m/s

Initial mass (Bucket and bag)		
Dry	359.6	g
Wet	518.5	g

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Clearance time (s)
1	113.3	128.0	132.8	120	48.0
2	147.3	158.2	153.4	120	56.0
3	185.2	215.3	225.7	120	54.0
4	253.2	294.3	308.3	120	65.0
5	298.0	345.8	367.4	120	84.0
6	357.3	411.7	411.8	120	103.0
7	393.3	462.9	483.1	120	134.0
8	38.7	39.9	49.8	120	45.0
9	462.1	542.0	546.7	120	252.0
10	506.5	563.2	573.8	120	320.0

Date 29/09/2014

Series T4

Experimental Properties

Property		Units
Sediment type	Quartz	
Mean grain size, d_{50}	1.40	mm
Settling velocity, w	0.130	m/s
Sediment density, ρ_s	2650	kg/m^3
Gravitation acceleration, g	9.81	m/s^2

Property		Units
θ	1.38	rad
Width across top	0.228	m
Flow area, A	0.00447	m^2
Wetted perimeter, P	0.207	m
Hydraulic radius, R	0.0216	m

Flume length	11.4	m
Slope, S	0.0035	
Pipe diameter, d	0.30	m
Pipe radius	0.15	m

Froude number	4.7	
Flow regime	Supercritical	

Average flow depth, D	0.0343	m
Flow rate, Q	9.30	L/s
Flow rate, Q	0.00930	m^3/s
Velocity, U	2.079	m/s

Initial mass (Bucket and bag)		
Dry	359.6	g
Wet	518.5	g

Experimental Readings

Reading no.	Input (dry) (g)	Input (wet) (g)	Output (wet) (g)	Collection time (s)	Clearance time (s)
1	35.3	35.0	84.5	120	46.0
2	135.7	144.8	133.6	120	53.0
3	152.0	163.4	172.6	120	51.0
4	187.3	217.2	228.5	120	55.0
5	289.2	331.7	352.1	120	65.0
6	351.1	388.2	382.5	120	488.0
7	309.1	332.8	364.2	120	67.0