

# WATER INJECTION DREDGING

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## 1 Description of method

Almost all harbour basins suffer from the problem of siltation of sediments. Usually, the deposited materials inside a harbour basin consist of relatively coarse, sandy materials (0.1 to 0.2 mm) near the entrance and relatively fine, muddy materials (<0.032 mm) at the end of the harbour basin. The in-situ dry density of the top layer of the consolidated mud generally is in the range of 200 to 400 kg/m<sup>3</sup>.

If a deeper channel (sink) is situated in front of the harbour entrance, the removal of deposited mud layers in small-scale harbour basins can be most efficiently done by using the method of water injection dredging (WID). This method is based on the fluidization of the top layer of the mud bed by pressurized injection of large volumes of water from a dredge vessel. The dredge vessel consists of a barge with pumps and a wide manifold (5 to 10 m) being a horizontal jet pipe (diameter of 0.6 to 0.8 m) with nozzles (diameter of 0.05 m) which can be lowered to a small distance above the bed, see **Figure 1**. The injected flow rate is in the range of 1 to 2 m<sup>3</sup>/s. The injected water velocities are in the range of 5 to 10 m/s.



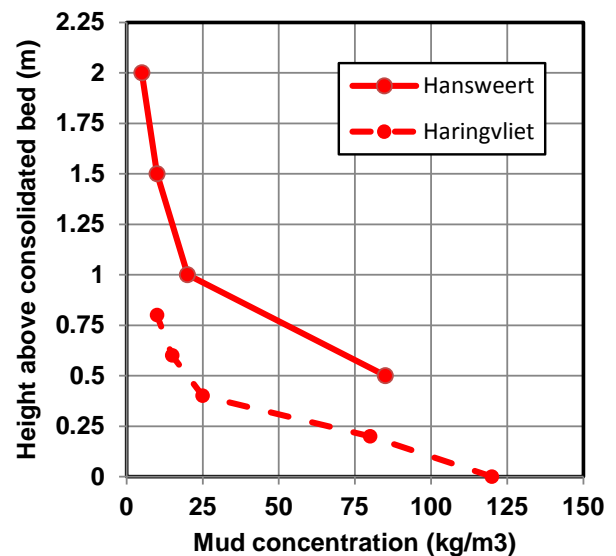
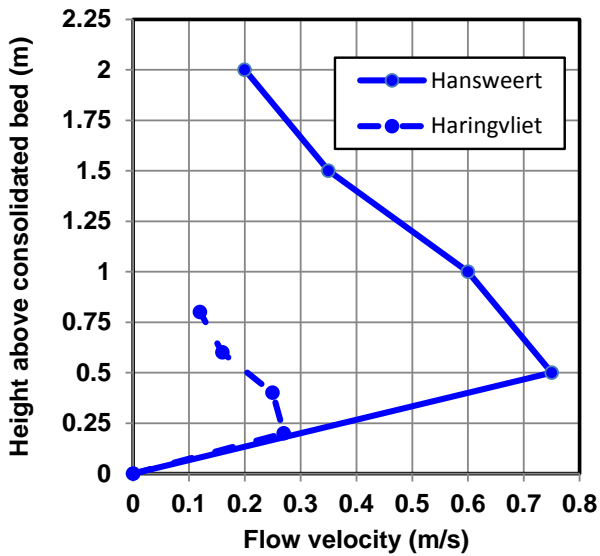
**Figure 1** Water injection dredging equipment (jet pipe is above water surface)

## 2 Field experiments

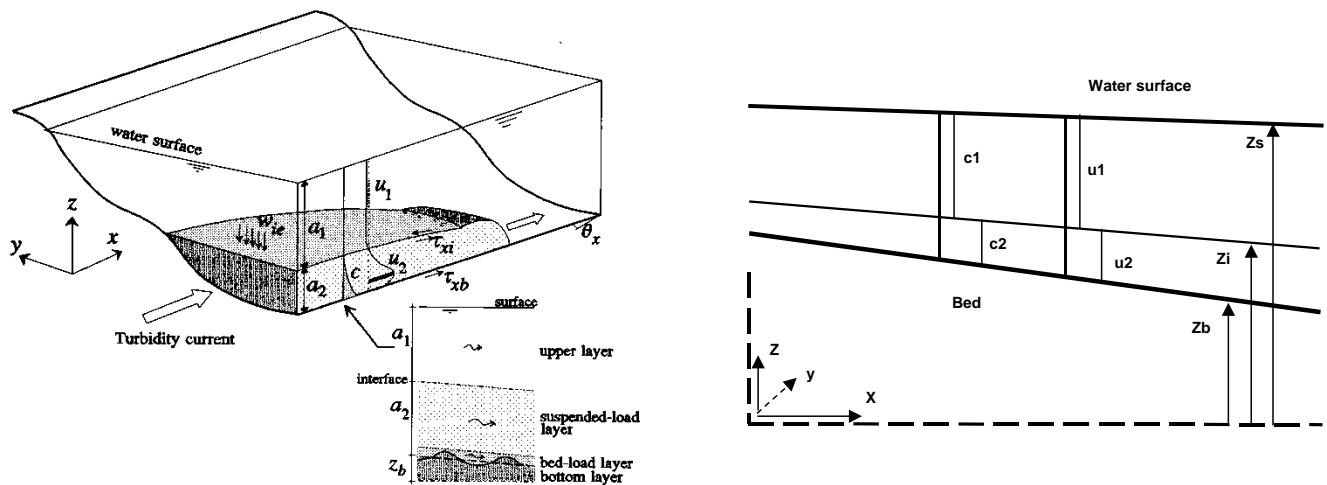
Rijkswaterstaat (1986) has tested the WID method and carried out transport measurements using an instrumented frame placed on the consolidated bed in conditions with a fluid mud flow generated by WID in the harbour basin of Hansweert along the Westerschelde Estuary, The Netherlands. The measurement location was close to the harbour entrance to detect the characteristics of the mud flow leaving the basin. Some data are given in **Figure 2**. The effective thickness of the fluid mud layer with concentrations in the range of 20 to 100 kg/m<sup>3</sup> was of the order of 1 m. The maximum flow velocity was found to be about 0.75 m/s at 0.5 m above the consolidated bed. The fluid mud flow leaving the basin persisted during the flood period of the tidal cycle with tidal filling velocities of about 0.1 to 0.2 m/s in the entrance.

The water injection rates were in the range of 5000 to 10000 m<sup>3</sup>/hour using a jet pipe width of 6 to 9 m. The average production rates in terms of the mud volume forced out of the harbour basin were found to be in the range of 800 to 1000 m<sup>3</sup>/hour for a dredge vessel with a pipe width of 9 m. Thus, the effective production rate is about 100 m<sup>3</sup>/m/hour. The total deposition volume of 170.000 m<sup>3</sup> in the harbour of Hansweert was removed in about 160 hours. The most effective working method was to first create a main channel inside the basin working from the entrance to the end of the basin. The mud along the sides of the basin can flow towards the deeper channel. The dredging of the main channel had to be repeated various times.

A similar experiment has been done in the Haringvliet basin (see **Figure 2**; Deltares, 1994a).



**Figure 2** Flow velocity and mud concentration generated by WID in Harbour of Hansweert and in Haringvliet basin, The Netherlands



**Figure 3** Schematization of fluid mud flow as two-layer system

### 3 Fluid mud flow dynamics

Using the WID-method, a high-concentration fluid mud mixture is generated above the bed which behaves as a density current with a clear front flowing to deeper areas driven by the excess density relative to the surrounding clear water. To initiate and maintain a high-velocity and high-concentration flow of mud, the densimetric Froude number of the mud flow should be larger than 1 (supercritical mud flow).

The dynamics of the high-concentration mud flow in a relatively clear water can be schematized as a two layer system with a high-concentration mud flow layer 2 near the bed and a relatively clear upper layer 1 ( $c_1 \cong 0$ ), see **Figure 3**.

The Froude number of the mud flow is defined as  $Fr_2 = u_2 / [(\Delta\rho_2/\rho_2)gh_2]^{0.5}$  with  $u_2$ = mud flow velocity,  $h_2$ = mud flow layer thickness,  $c_2$ = mud concentration,  $\Delta\rho_2 = \rho_2 - \rho_w = (\rho_s - \rho_w)c_2$

$Fr_2 > 1$  implies that the mud flow is supercritical and the mud layer depth  $h_2$  is smaller than the critical depth of the mud layer flow  $h_{2,cr} = [q^2 / \{((\rho_s - \rho_w)/\rho_2)c_2 g \cos\beta\}]^{1/3}$  with  $q = u_2 h_2$  = flow rate inside mud layer and  $\beta$  = bed slope angle (Van Rijn, 2005, 2012). The flow rate and the mud flow characteristics will depend on the upstream boundary conditions.

The stability of the high concentration mud layer depends on the local Richardson number defined as:

$$Ri = \frac{|g \partial\rho_m/\partial z|}{\rho_m (\partial u/\partial z)^2} \quad (1)$$

The Ri-number represents the balance between the stabilizing effects of gravity and the density gradient and the destabilizing effect of the near-bed velocity gradient which produces turbulent eddies and hence diffusion/mixing (Deltares, 1994; Mastbergen and Van den Berg, 2003).

Experimental research shows that the high concentration mud layer is stable for  $Ri > 0.25$ .

**Figure 4** shows two situations with a high-concentration mud layer:

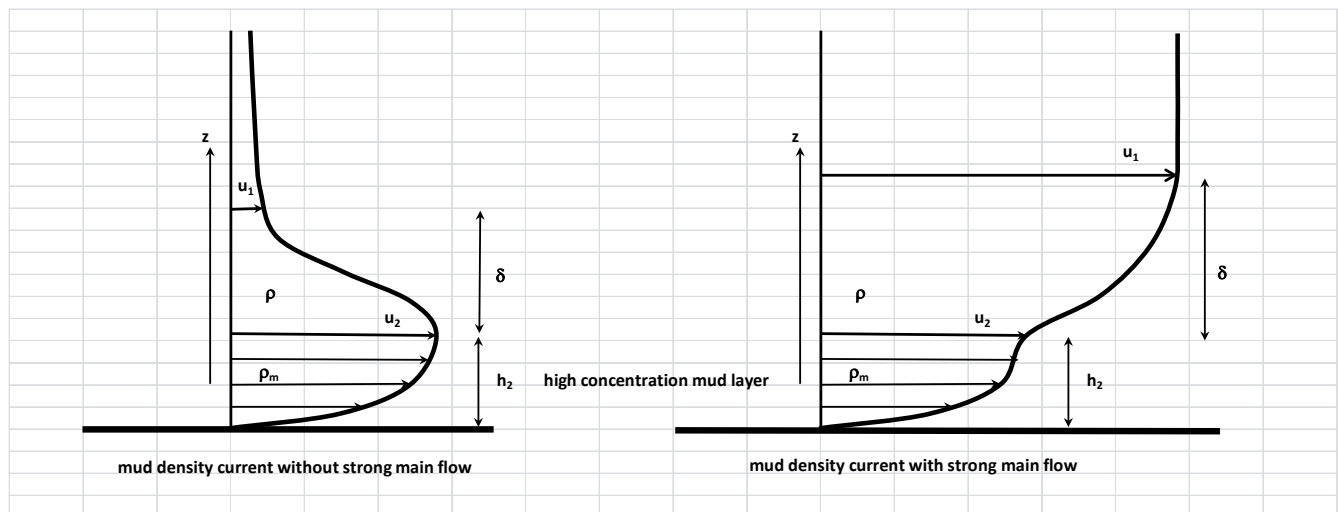
- *left*: mud density current in still water (no main flow in upper layer); the density current will exert a drag force on the upper layer generating a weak upper current in the same direction  $u_1 \ll u_2$ ; a very weak return current may be generated close to the water surface depending on the basin geometry
- *right*: mud density current under a strong current  $u_1$  in the upper layer; a stable density current cannot develop if the upper flow is too strong (large mixing capacity).

The density gradient can be estimated as  $\partial\rho_m/\partial z \cong \Delta\rho_2/\delta = (\Delta\rho_2/\rho_2) (\rho_2/\delta)$

The velocity gradient can be estimated as  $\partial u/\partial z \cong u_2/\delta$

Based on this, the Ri-number can be expressed as:  $Ri = [g (\Delta\rho_2/\rho_2) h_2 (\delta/h_2)] / (u_2)^2 = (Fr)^{-2} (\delta/h_2)$

with:  $Fr$  = Froude number =  $u_2 / [(\Delta\rho_2/\rho_2) g h_2]^{0.5}$  and  $(\delta/h_2) \cong 1$ .



**Figure 4** Velocity structure of high-concentration mud flow

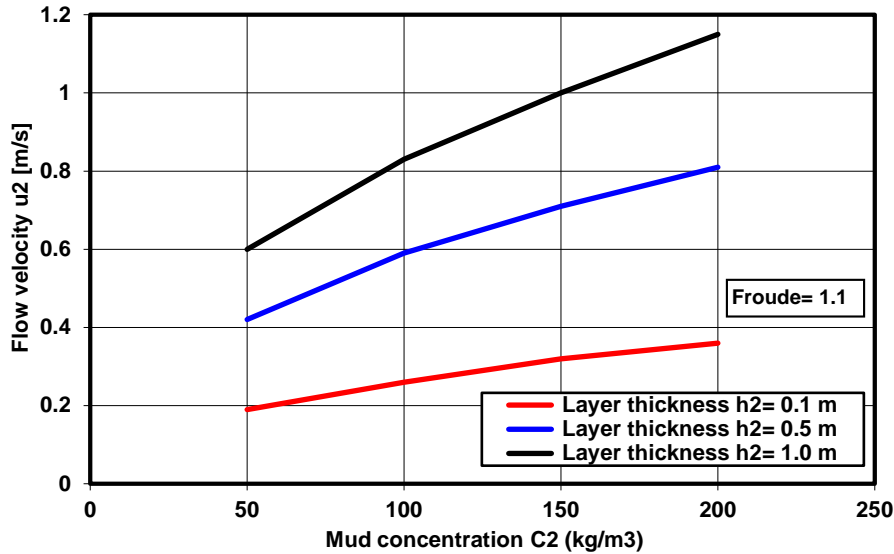
Stable conditions for  $Ri > 0.25$  or  $Fr_2 < 2$  (mud flow without much mixing and pronounced interface).

In the case of a strong upper flow the Froude number of the mud flow is  $Fr_2 = (u_1 - u_2) / [(\Delta\rho_2 / \rho_2) g h_2]^{0.5}$

Practical values of  $h_2$ ,  $u_2$  and  $c_2$  are in the range of  $h_2 = 0.1$  to  $1$  m,  $u_2 = 0.1$  to  $1$  m/s and  $c_2 = 50$  to  $200$   $kg/m^3$  (volumetric concentration). The flow velocity of a supercritical mud flow can be expressed as:  $u_2 = Fr_2 [(\Delta\rho / \rho_2) g h_2]^{0.5}$ .

Assuming  $Fr_2 = 1.1$ ,  $\Delta\rho_2 = c_2(\rho_s - \rho_w)$  and  $\rho_2 = \rho_s c_2 + (1 - c_2)\rho_w$ , the relationship between these parameters is given in **Figure 5** ( $c_2$  as volumetric concentration = ratio of mass concentration and sediment density). Mud flow velocities in the range of  $0.2$  to  $1$  m/s can be generated by WID. The measured mean mud flow velocity of about  $0.6$  m/s in the harbour of Hansweert (see Section 2) for a layer with a thickness of  $1$  m and a mean concentration of  $50$   $kg/m^3$  is in good agreement with the results of **Figure 2**.

According to **Ning Chien (1998)**, the front velocity of a supercritical mud flow above a consolidated bed can be expressed as:  $u_{front} = [\{2 / (1 + \lambda)\} \{(\Delta\rho / \rho_w) g h_2\}]^{0.5}$  with  $\lambda =$  friction coefficient ( $= 0.6$ ). The front velocity is about equal to the average velocity  $u_2$  behind the front for  $Fr_2 \approx 1$  and is about  $70\%$  of the mean velocity  $u_2$  for  $Fr_2 \approx 2$ . Hence, the front velocity is somewhat smaller than the mud flow velocity behind the front.



**Figure 5** Mud flow velocity as function of mud layer thickness and mud concentration for  $Fr_2 = 1.1$

In the case of a 1D channel with a constant slope angle ( $\beta$ ) it is most easy to formulate the equations with respect to a tilting coordinate system. The hydrostatic pressure is now defined along the  $n$ -axis, yielding a  $g \cos\beta$ -term (for all terms with  $\partial h_2 / \partial s$ ) and a  $g \sin\beta$ -term for the slope force along the  $s$ -axis (decomposition of gravity force vector). The water depth is defined to be perpendicular to the bed. Assuming that the flow is steady, that the velocities ( $u_1$ ) and sediment concentrations ( $c_1$ ) in the upper layer are negligibly small ( $c_1 \approx 0$ , density is equal to the fluid density), that the flow in the lower layer is fully turbulent and that the pressure is hydrostatic, the equations for 1D conditions can be expressed by Eqs. (2) to (4), (Van Rijn, 2005, 2012; Van Rijn, 2004).

**momentum balance of mixture in lower layer 2 in s-direction**

$$\rho_2 \partial(u_2^2 h_2) / \partial s + (\rho_s - \rho_w) h_2 c_2 [(g \cos\beta) \partial h_2 / \partial s - g \sin\beta] + (\rho_s - \rho_w) (0.5 g h_2^2 \cos\beta + u_2^2 h_2) \partial c_2 / \partial s + (\tau_i + \tau_b) = 0 \quad (2)$$

**mass balance for fluid in lower layer 2**

$$\partial(u_2 h_2 (1 - c_2)) / \partial s - W_i - W_b = 0 \quad (3)$$

**mass balance for sediment in lower layer 2**

$$\partial(u_2 c_2 h_2) / \partial s - S_i - S_b = 0 \quad (4)$$

with :

$h_1, h_2$  = thickness of upper and lower layer ( $h_1 + h_2 = h$  = flow depth),  
 $c_1, c_2$  = depth-averaged volumetric suspended sediment concentration in upper layer 1 and lower layer 2,  
 $u_1 = q_1 / h_1, u_2 = q_2 / h_2$  = velocity in upper layer 1 and lower layer 2,  
 $W_i$  = exchange of fluid at the interface,  
 $W_b$  = exchange of fluid at the bed,  
 $S_i$  = exchange of sediment at the interface,  
 $S_b$  = exchange of sediment at the bed,  
 $\rho_2$  = mixture density of lower layer,  
 $\rho_w$  = fluid density (clear water in upper layer 1),  
 $\rho_s$  = sediment density,  
 $\tau_i$  = shear stress at interface,  
 $\tau_b$  = bed shear stress ( $= \rho C_d u_2^2$ ),  
 $C_d$  = bottom friction coefficient ( $= g / C^2$ ),  $C$  = Chézy coefficient,  
 $C_{di}$  = interface friction coefficient,  
 $\beta$  = angle of bed slope in s-direction,  
 $s$  = coordinate along bed slope.

The equations (2), (3) and (4) define a set of three equations with three unknown parameters  $u_2, h_2$  and  $c_2$ , which can be solved for given boundary conditions and closure expressions (Van Rijn, 2005, 2012). Equations (2) and (3) can be reformulated (eliminating the term  $\partial u_2 / \partial s$  using Eq. 3), yielding:

$$\partial h_2 / \partial s = [1 / (\gamma_2 (1 - c_2))] [\gamma_1 (1 - c_2) - (1 - c_2)(\tau_i + \tau_b) - 2\rho_2 u_2 (W_i + W_b) - \gamma_3 \partial c_2 / \partial s] \quad (5)$$

with:

$$\begin{aligned} \gamma_1 &= (\rho_s - \rho_w) h_2 c_2 g \sin \beta \\ \gamma_2 &= (\rho_s - \rho_w) h_2 c_2 g \cos \beta - \rho_2 (u_2)^2 = (\rho_s - \rho_w) h_2 c_2 g \cos \beta [1 - (h_{2,cr} / h_2)^3] \\ \gamma_3 &= 2\rho_2 h_2 (u_2)^2 + (\rho_s - \rho_w) (1 - c_2) h_2 (u_2)^2 + 0.5(\rho_s - \rho_w) (1 - c_2) (h_2)^2 g \cos \beta \end{aligned}$$

Defining a critical depth ( $Fr_2 = 1$ ) and an equilibrium depth (uniform flow of water and sediment;  $\tau_i + \tau_b = \rho_2 (C_d + C_{di}) q^2 / h_2^2$ ), Equation (4) can also be expressed as:

$$\partial h_2 / \partial s = \frac{(\sin \beta / \cos \beta) [1 - (h_{2,eq} / h_2)^3 - \alpha_1 (W_i + W_b) / u_2 - \alpha_2 h_2 \partial c_2 / \partial s]}{[1 - (h_{2,cr} / h_2)^3]} \quad (6)$$

with:

$$\begin{aligned} \alpha_1 &= \frac{2\rho_2 (u_2)^2}{(\rho_s - \rho_w) (1 - c_2) h_2 c_2 g \sin \beta} \\ \alpha_2 &= \frac{2\rho_2 (u_2)^2 + (\rho_s - \rho_w) (1 - c_2) (u_2)^2 + 0.5(\rho_s - \rho_w) (1 - c_2) h_2 g \cos \beta}{(\rho_s - \rho_w) (1 - c_2) c_2 h_2 g \sin \beta} \end{aligned}$$

$$h_{2,cr} = [q^2 / \{((\rho_s - \rho_w) / \rho_2) c_2 g \cos \beta\}]^{1/3} = \text{critical depth (defined by Froude number } Fr_2 = 1)$$

$$h_{2,eq} = [(C_d + C_{di}) q^2 / \{((\rho_s - \rho_w) / \rho_2) c_2 g \sin \beta\}]^{1/3} = \text{equilibrium depth (turbulent flow of water and sediment)}$$

$h_{2,eq}=[(\tau_i+\tau_b) (h_2)^2/((\rho_s-\rho_w)c_2 g \sin\beta)]^{1/3}$  = equilibrium depth (laminar flow of water and sediment)

$\tau_b+\tau_i = \rho_2(C_d+C_{di})q^2/h_2^2$  =shear stress turbulent flow ( $C_{di}= 0.333C_d$ , Mastbergen and Van den Berg, 2003)

Equation (6) cannot be used for a slope of  $\beta= 0$ , because  $h_{2,eq}$  is undefined for slope  $\beta= 0$ .

Assuming supercritical flow ( $Fr_2>1$ ), the gradient  $\partial h_2/\partial s$  increases ( $h_2$  grows more rapidly) with:

- increasing entrainment ( $W_i+W_b$ ) of fluid into the lower layer;
- increasing shear stresses ( $\tau_i+\tau_b$ ) at the bed and at the interface;
- increasing concentration in downstream direction (increasing  $\partial c_2/\partial s$ ).

A special case is obtained by neglecting the exchange terms at the upper interface and at the bed ( $W_i=W_b=S_i=S_b= 0$ ). Consequently, it follows that  $\partial q/\partial s= 0$  and  $\partial q_s/\partial s= 0$  and thus  $\partial c_2/\partial s= 0$ , yielding (see also **Ning Chien, 1998**):

$$\partial h_2/\partial s= \frac{(\sin\beta/\cos\beta)[1 - (h_{2,eq}/h_2)^3]}{[1 - (h_{2,cr}/h_2)^3]} \quad (7)$$

Equation (7) shows a close resemblance with the Bélanger equation for clear water flow (Van Rijn, 1990, 2011).

For supercritical flow ( $Fr_2 > 1$ ) with  $h_2 < h_{2,eq}$  and  $h_2 < h_{2,cr}$  it follows that the upper and lower terms of Equations (6) and (7) are both negative and thus  $\partial h_2/\partial s > 0$  or a growing layer thickness in downstream direction.

Equations (6) and (7) are only meaningful for a supercritical density current. In subcritical flow conditions with relatively small velocities the sediment particles will rapidly settle out resulting in a 'normal' Rouse-type of concentration profile. In the latter case the density flow will gradually die out.

The mass balances of fluid and sediment (Eqs. 3 and 4) can be reformulated as:

$$\partial q/\partial s= \partial q_s/\partial s + (W_i+W_b) \quad (8)$$

$$\partial q_s/\partial s = S_b + S_i \quad (9)$$

with:

$$u_2=q/h_2 \quad (10)$$

$$c_2=q_s/q \quad (11)$$

The density difference between the sediment suspension and the clear water is given by:

$$\Delta\rho_2 = \rho_2-\rho_w = [\rho_s c_2 + (1-c_2)\rho_w] - \rho_w = c_2(\rho_s - \rho_w) \quad (12)$$

The relative density difference is given by:

$$\Delta\rho_2/\rho_2 = [c_2(\rho_s - \rho_w)]/[c_2(\rho_s - \rho_w) + \rho_w] = c_2(s-1)/(c_2(s-1)+1) \quad (14)$$

with:

$\rho_2 = \rho_s c_2 + (1-c_2)\rho_w$  = density of fluid-sediment mixture of lower mud layer,

$c_2 = (\rho_2-\rho_w)/(\rho_s-\rho_w)$  = volume concentration of lower layer,

$\rho_w$  = fluid density (fresh water= 1000 kg/m<sup>3</sup>, saline water= 1020 kg/m<sup>3</sup>),

$\rho_s$  = sediment density (2650 kg/m<sup>3</sup>),  
 $s$  = relative density =  $\rho_s/\rho_w$ .

Deposition will take place for  $\tau_b < \tau_{b,d}$ . Erosion will take place for  $\tau_b > \tau_{b,e}$

The above-given set of equations can be solved in a spreadsheet model **TC-MUD**. A similar model **TC-SAND** is available for Sand (Van Rijn, 2005, 2012). Both models are only valid for supercritical flow conditions.

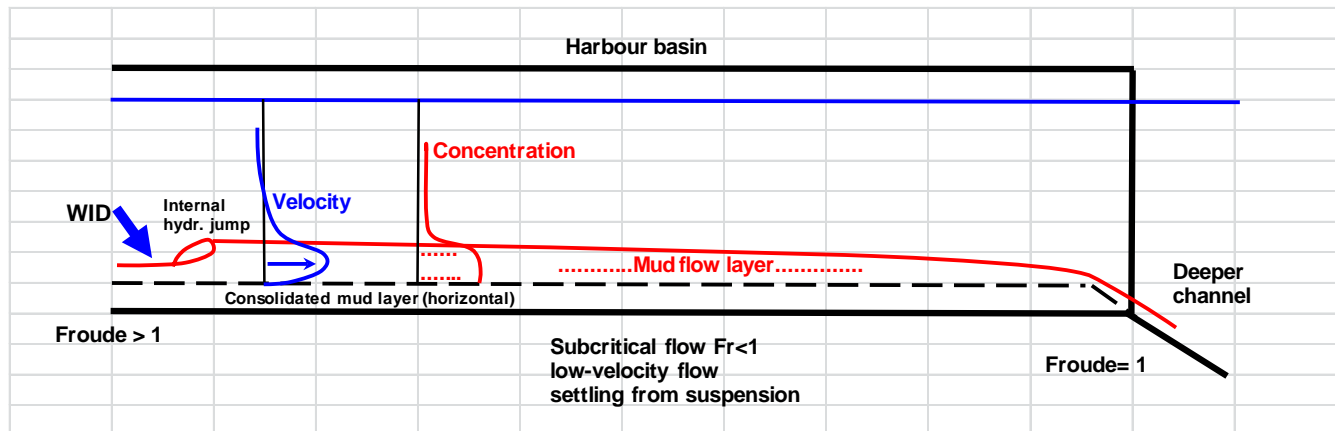
#### 4 Effect of bed slope and sediment composition

##### **Subcritical mud flows**

The behaviour of the fluid mud flow strongly depends on the local bed slope and the sediment grain size (mud or sand).

Mud density currents generated by WID will initially have a Froude number larger than 1 ( $Fr_2 > 1$ ). Hence, the mud flow will start as a supercritical flow.

In the case of an almost horizontal bed (no gravity pull) the mud flow thickness will immediately increase due to relatively large bed friction forces and the Froude number will become smaller than 1 ( $Fr_2 < 1$ ) resulting in an internal hydraulic jump, similar to a free-surface hydraulic jump. The mud flow will continue as a subcritical flow, see **Figure 6**. This subcritical mud flow will only be stable if the main flow of the upper layer is very weak (still water).

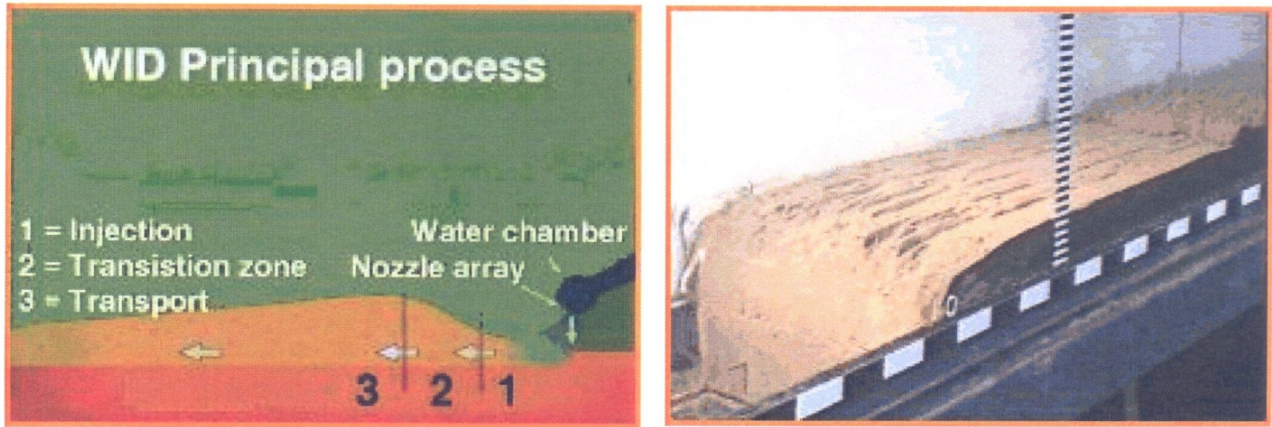


**Figure 6** *Mud flow in subcritical regime*

The mud flow characteristics will depend on the downstream boundary conditions.

If a deeper channel is present (not too far away) at the harbour entrance, the mud flow will slowly accelerate and the Froude number will approach 1 at the transition to the deeper channel section (similar to a free surface water fall). The thickness of the mud flow will slowly decrease (similar to a free surface slope in subcritical flow). The depth at the transition will be the critical depth. This behaviour was clearly observed in a scale test of WID in a laboratory flume ( $h_2 \cong 0.05$  m,  $u_2 \cong 0.1$  m/s; **Figure 7**; Deltares, 1995).

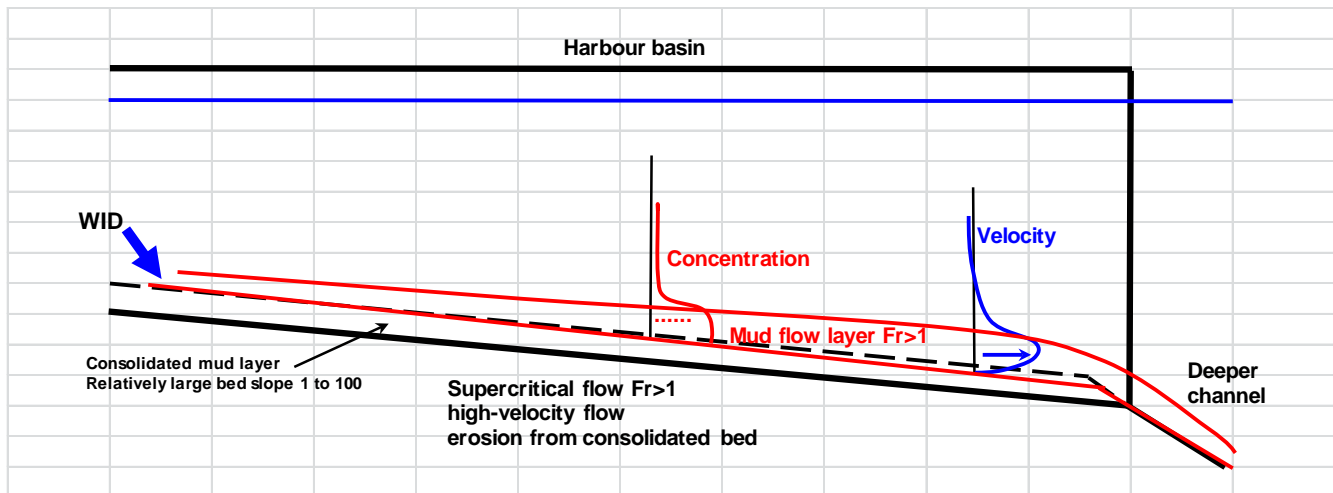
If the deeper section is far away from the source (WID), the mud flow thickness behind the hydraulic jump will slowly decrease due to deposition of mud. The mud flow rate (and velocity  $u_2$ ) will be reduced to very small values. Deposition of mud to the bed will eventually dominate and the mud flow will die out.



**Figure 7** *Mud density flow generated by WID in laboratory flume; hydraulic jump in transition zone 2, (Deltares, 1995)*

### **Supercritical mud flows**

If the bed slope is sufficiently large, the supercritical mud flow will accelerate (no hydraulic jump) and even erode the local bed resulting in an avalanching-type of supercritical mud flow with relatively high flow velocities and Froude numbers much larger than 1. The mud layer thickness will slowly increase, see **Figure 8**. The mud concentration may increase to its maximum value of about 300 to 400 kg/m<sup>3</sup>. The mud flow over a strongly sloping bed may extend over a relatively long distance (kilometers).

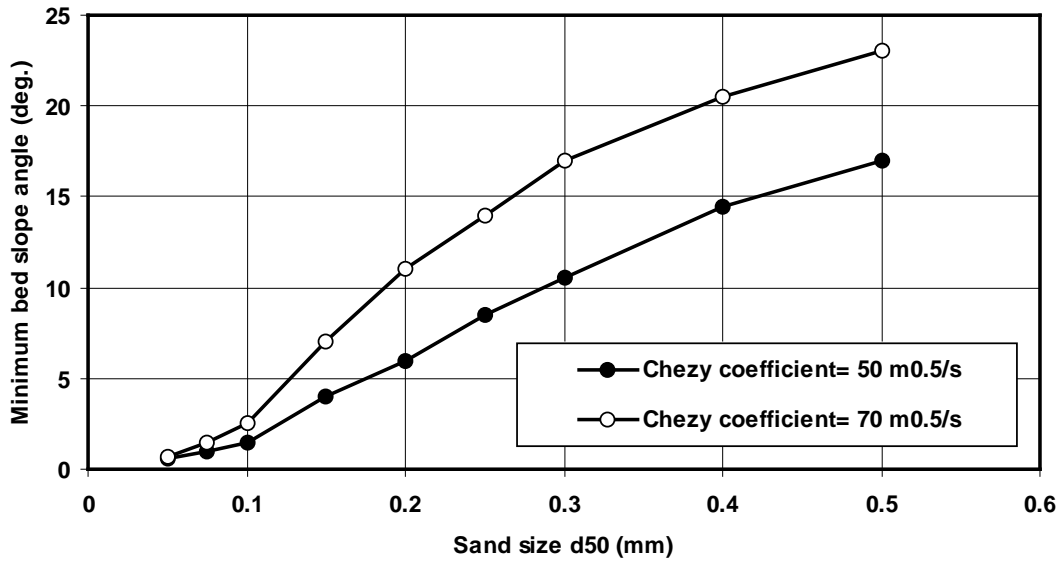


**Figure 8** *Mud flow in supercritical flow regime*

The TC-SAND and TC-MUD models have been used to explore the critical (minimum) bed slope slope at which the flow of sediment will accelerate, see **Figure 9** (Van Rijn, 2005, 2012).

The critical bed slope increases from about 0.6° for very fine sand of 0.05 mm to about 20° for coarse sand of 0.5 mm, as shown in **Figure 9**. The critical slope angle for a mud bed is about 0.2° (1 to 300) to 0.4° (1 to 150) based on an initial concentration of 50 kg/m<sup>3</sup> and an initial layer thickness of 1 m and a settling velocity of 0.1 mm/s (low value due to hindered settling).





**Figure 9** Critical slope angle for self-accelerating conditions as a function of sediment size

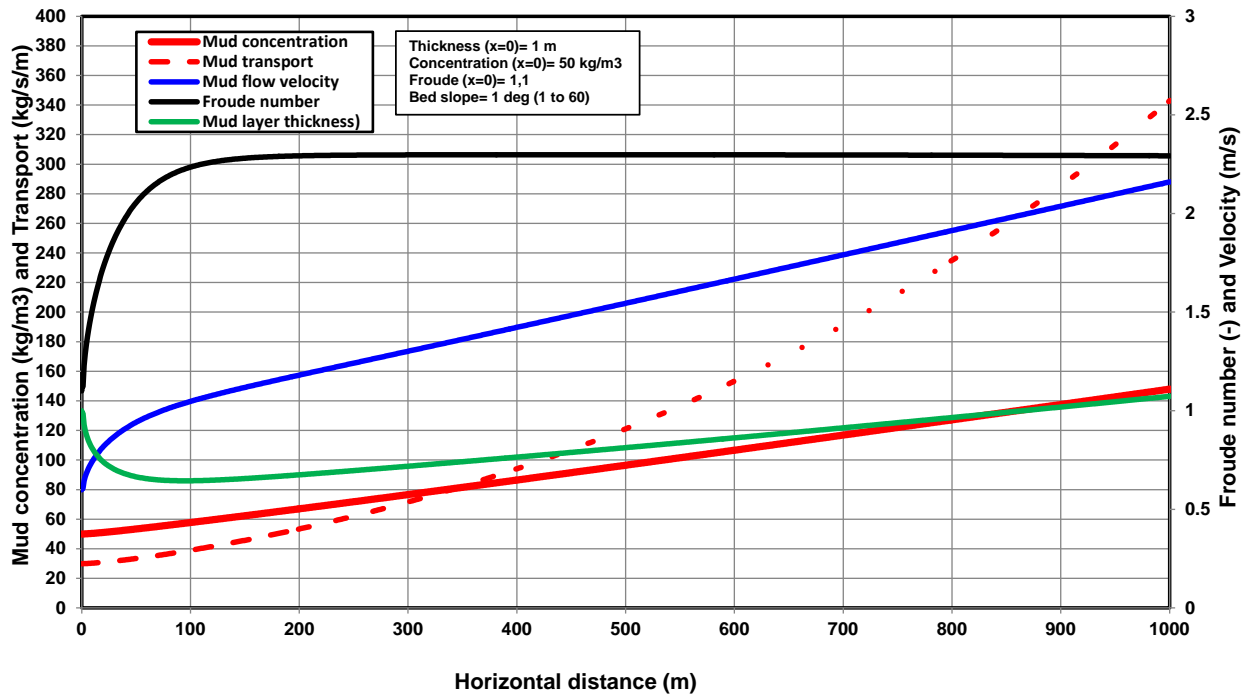
#### **Computation examples of supercritical mud flows**

**Figures 10, 11 and 12** show the mud flow characteristics for three bed slopes of  $1^\circ$  (1 to 60),  $0.5^\circ$  (1 to 120) and  $0.1^\circ$  (1 to 500). The boundary conditions (behind the water injection pipe) are assumed to be  $h_2 = 1$  m,  $c_2 = 50$  kg/m<sup>3</sup> and Froude = 1.1 resulting in an initial mud flow velocity  $u_2 = 0.6$  m/s, see **Figure 5**. In all three cases the Froude number remains larger than 1 over large distances up to almost 1 km (supercritical flow).

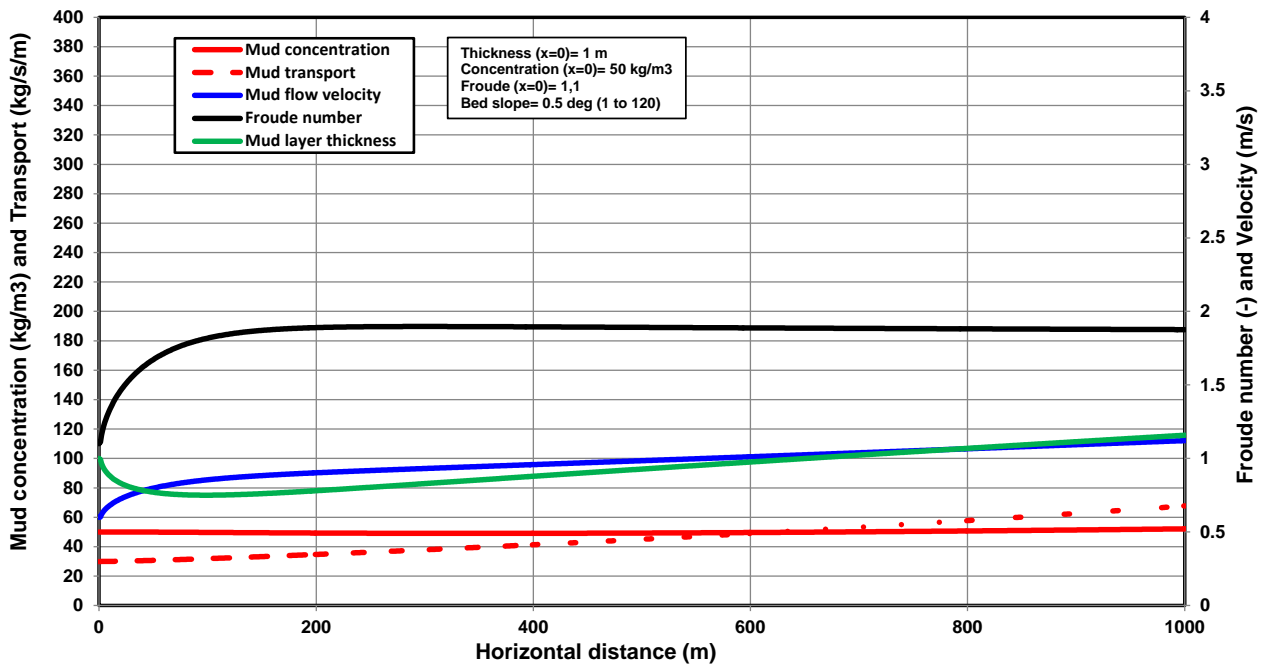
In the case of a bed slope of 1 to 60, the mud flow velocity increases continuously to a value of about 2 m/s after 1 km (self acceleration). The mud transport increases significantly due to erosion of mud from the bed.

In the case of a bed slope of 1 to 120, the mud flow velocity increases slightly due to self acceleration (slight erosion of mud from bed) from 0.6 m/s to 1.1 m/s. The mud concentration increases slightly to about 55 kg/m<sup>3</sup> and the mud transport increases from 30 to about 50 kg/ms after 1 km.

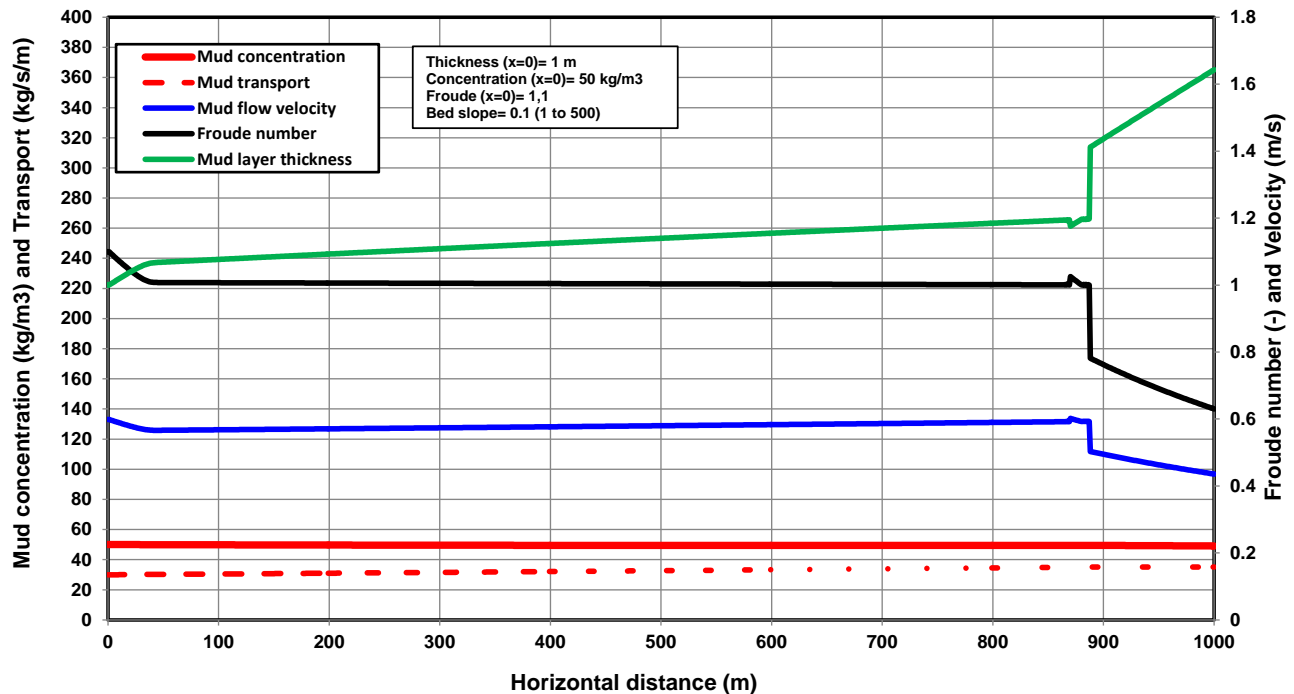
In the case of a bed slope 1 to 500 the mud concentration, velocity and transport decrease very slightly (**Figure 12**). The Froude number decreases to about 1 at about 900 m from the source (WID), where the transition from supercritical to subcritical flow will take place and an internal hydraulic jump will be generated with an increase in layer thickness and a decrease in flow velocity (factor 2). The mud concentration may remain constant or increase somewhat behind the hydraulic jump due to the production of local turbulence but further away the mud concentration will gradually decrease due to settling of mud in subcritical flow with reduced velocities and the mud density flow will slowly disintegrate.



**Figure 10** *Mud concentration, mud flow velocity, mud layer thickness, mud transport and Froude number as function of distance; Bed slope= 1° (1 to 60)*



**Figure 11** *Mud concentration, mud flow velocity, mud layer thickness, mud transport and Froude number as function of distance; Bed slope= 0.5° (1 to 120)*



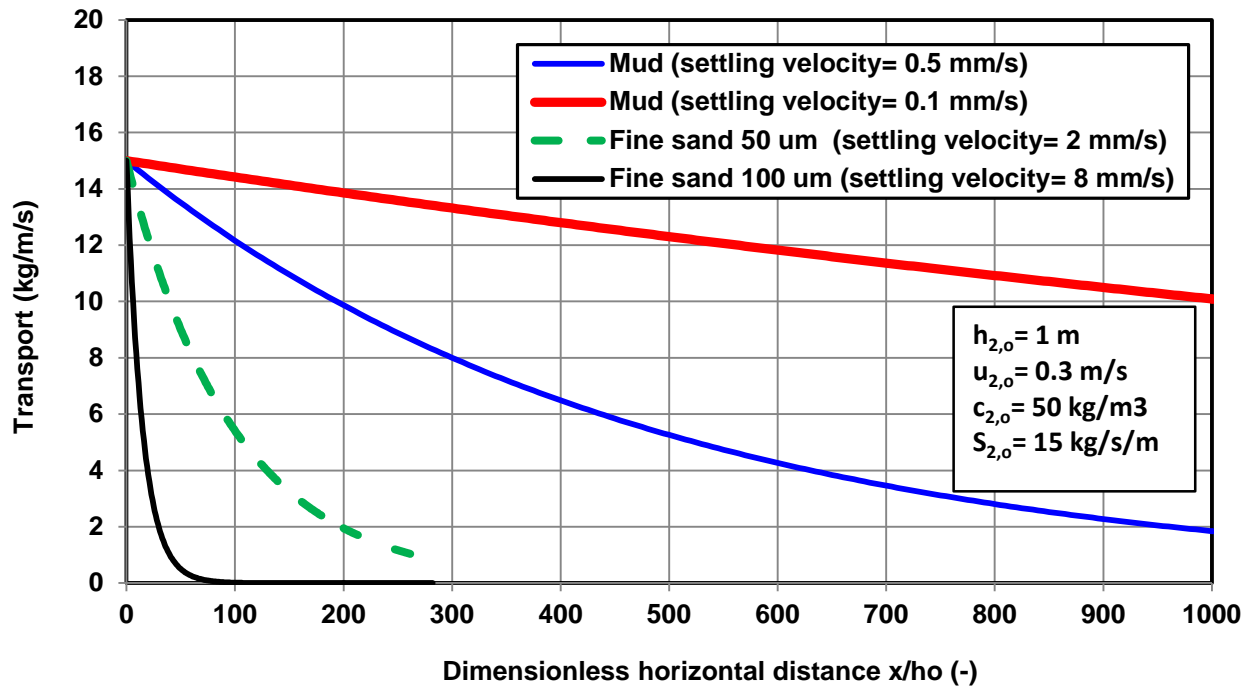
**Figure 12** *Mud concentration, mud flow velocity, mud layer thickness, mud transport and Froude number as function of distance; Bed slope=  $0.1^\circ$  (1 to 500)*

### **Settling in subcritical mud flows**

The settling of sediment in a subcritical sediment flow can be simulated by the SED-TUBE model (Van Rijn, 2005, 2012). The upstream boundary conditions are assumed to be:  $h_{2,0} = 1$  m,  $u_{2,0} = 0.3$  m/s,  $c_{2,0} = 50$  kg/m<sup>3</sup> resulting in  $S_{2,0} = 15$  kg/m/s. The settling velocity of mud will be in the range of 0.1 to 0.5 mm/s. Reflocculation of very fine particles may take place in subcritical flow (saline conditions). Chézy is set to 95 m<sup>0.5</sup>/s (very smooth flow). Similar computations have been made for fine sand of 50  $\mu$ m (0.05 mm) with a settling velocity of 2 mm/s and 100  $\mu$ m-sand (0.1 mm) with a settling velocity of 8 mm/s.

Assuming flocculated mud with an effective settling velocity of 0.5 mm/s, the settling length for mud is of the order of  $x/h_0 = 1000$  or 1000 m with  $h_0 = 1$  m (= layer thickness). The settling length for fine sand of 100  $\mu$ m is about  $x/h_0 = 50$  or 50 m with  $h_0 = 1$  m.

A simple calculation of the settling length can be obtained from the ratio of the settling velocity and the flow velocity, neglecting the upward forces due to turbulence-related mixing. A sand particle of 100  $\mu$ m with a settling velocity of 8 mm/s at the interface of the high-concentration layer with velocity of 0.3 m/s (= 300 mm/s) falls at a slope of 8 to 300 or 1 to 37.5 resulting in a settling length of 37.5 m for a layer thickness of 1 m. Thus, after about 40 m all sand particles have settled to the bed.



**Figure 13** *Suspended transport as function of distance and settling velocity for subcritical flow*

## 5 WID Production rates

In practice the bed slope of a harbour basin will be in the range of 1 to 500 and 1 to 1000 m (almost horizontal). In that case the supercritical flow produced by WID will almost immediately reduce to subcritical flow ( $Fr_2 \cong 0.5-1$ ) and an internal hydraulic jump will be generated close to WID (within 10 m). The flow velocity will be reduced by a factor 2 from about 0.6 in supercritical flow to about 0.3 m/s in subcritical flow. The high-concentration flow will remain stable (see Equation 1) as no sediment can be mixed up to the upper layer where the flow is very weak ( $\cong 0.05$  m/s). The initial transport rate is given by:  $S_{2,o} = u_{2,o} h_{2,o} c_{2,o}$ . Using  $u_{2,o} = 0.25-0.5$  m/s,  $h_{2,o} = 0.5$  to 1 m and  $c_{2,o} = 25$  to 50  $kg/m^3$ , the initial transport rate is in the range of 3 to 25  $kg/m/s$  or 10 to 100 tons/m/hour.

Assuming a harbour basin length of 0.5 km, subcritical flow conditions and the presence of a relatively deep channel outside the harbour entrance, the transport rate of mud leaving the basin is as a conservative estimate of the order of about 50% (due to deposition and spreading) of the initial rate resulting in a range of 5 to 50 tons/m/hour. This is equivalent to 15 to 150  $m^3/m$  per hour, assuming an in-situ dry density of the consolidated bed of 0.3  $t/m^3$ .

Thus, the effective production rate of WID (mud leaving through the entrance into the deeper channel outside the harbour basin) is about 15 to 150  $m^3/m/hour$ . The observed production rate of about 100  $m^3/m/hour$  for the harbour of Hansweert (see Section 2) shows that relatively high production rates are feasible using WID.

The removal of sandy material (particles in the range of 50 to 200  $\mu m$ ) from the basin entrance using WID is more complicated and may involve various repeated step-wise efforts to bring the sediments from the inside of the basin to the harbour entrance.

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