

Effects of driftwood on backwater rise at submerged culverts.

Master Thesis

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Personal picture on cover.

Preface

Before you lies the master thesis “Effects of driftwood on backwater rise at submerged culverts.” It has been written to fulfill the graduation requirements of the Civil Engineering degree at the Delft University of Technology, for the specialisation of Hydraulic Engineering. I was engaged in researching and writing this thesis from February to December 2023.

First and foremost I would like to express my gratitude to my committee. This thesis would not have been possible without them. I want to thank Davide Wüthrich for being chair of my committee and for always encouraging me to keep improving this thesis. I would like to thank Daan Poppema for sharing his experience related to driftwood on numerous occasions. I want to express my gratitude to Wim Uijttewaal for joining the committee on short notice and for sharing his expertise with me during the final months of this project.

I would also like to thank Martine Rutten for her support in bringing this thesis to a good start. Lastly, I would like to thank the staff of the Waterlab at the Delft University of Technology for enabling me to conduct the experiments that were essential to this thesis.

Dennis Ronckers
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Abstract

Accumulations of floating debris at culverts and inverted siphons cause partial blockage of the hydraulic structure's inlet, leading to an increase in head loss that is also referred to as backwater rise. This effect is of high importance in flood management, because backwater rise associated with floating debris accumulations can cause or intensify flooding in the area upstream of the hydraulic structure, yet it is often not adequately addressed in practice or in design guidelines due to a lack of fundamental understanding. A prime example of this was observed at the Geul inverted siphon located near the village of Bunde in the south of Limburg, the Netherlands. During the 2021 floods the inverted siphon experienced a reduction in conveyance capacity due to accumulation of large wood at its inlet, intensifying flooding of nearby villages.

This thesis studied the development of floating debris accumulations at fully submerged and outlet-controlled culverts and inverted siphons, and analysed the effect that floating debris accumulations have on backwater rise. Experiments were performed in a flume under varying conditions of Froude number and submergence rate, achieved by adjusting the upstream flow depth compared to the culvert's height. The scale model was representative of inverted siphons typical of the Limburg province. Upstream and downstream water levels were measured after addition of fixed batches of debris to the flume. The debris mixture consisted of six size classes of natural wood and was based on logs observed at the Geul inverted siphon. A grate was placed at the culvert's inlet to prevent logs from passing through. In addition, flow velocities were measured at various locations using Laser Doppler Anemometry.

Based on visual observations and a feasibility study three mechanisms were proposed that describe the development of floating debris accumulations, and the feasibility of these mechanisms was confirmed in preliminary analyses. Furthermore, the experimental results were analysed to determine the influence on backwater rise of canopy drag due to skin friction within and below the accumulation, as well as form drag due to partial blockage of the inlet. It was found that in all cases inlet blockage was the dominant cause of head loss. Using this knowledge a theoretical framework based on a momentum balance equation was derived that relates the inlet blockage ratio to backwater rise, separating the contributions of the grate and debris. A regression analysis was then performed to obtain a design equation for the inlet blockage ratio caused by debris. In conclusion, this thesis provides improved understanding of the development of floating debris accumulations and their influence on backwater rise at submerged culverts and inverted siphons. The results enable more accurate predictions of water levels that can occur during floods and thus offer a solid foundation for flood management strategies.

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List of Symbols

δ	Opening height of the culvert or inverted siphon	[m]
ΔH	Head loss, also referred to as backwater rise	[m]
ρ_d	Density of debris	[kg/m ³]
ρ_w	Density of water	[kg/m ³]
a_b	Bulk factor	[–]
a_f	Spatially averaged frontal area of debris per jam volume	[1/m]
B	Width of the flume, channel and culvert or inverted siphon	[m]
c_D	Drag coefficient of individual log	[–]
$c_{D,grate}$	Drag coefficient of rigid grate	[–]
d_l	Log diameter	[m]
F_0	Initial upstream Froude number, defined as $U_0^2/(gh_0)$	[–]
g	Gravitational acceleration	[m/s ²]
h_0	Initial upstream water depth	[m]
h_0/δ	Submergence rate	[–]
h_{up}	Upstream water depth	[m]
K	Loss factor	[–]
K_{fric}	Friction loss factor	[–]
K_{gd}	Loss factor for grate and debris combined	[–]
K_g	Grate loss factor	[–]
K_{in}	Inlet loss factor	[–]
K_{out}	Outlet loss factor	[–]
L_A	Length of debris accumulation	[m]
L_l	Log length	[m]
p	Inlet blockage	[–]
p_d	Inlet blockage due to debris	[–]
p_g	Inlet blockage due to the grate	[–]
Re	Reynolds number, defined as $U_{up}h_{up}/\nu$	[–]
U_0	Initial upstream depth-averaged flow velocity	[m/s]
U_{cul}	Average flow velocity inside the culvert	[m/s]
U_{up}	Upstream average flow velocity	[m/s]
V_d	Design solid volume of debris	[m ³]
V_l	Loose volume of debris accumulation	[m ³]
V_s	Solid volume of debris	[m ³]
v_x	Horizontal (streamwise) flow velocity	[m/s]

v_y	Vertical flow velocity	[m/s]
$V_{s,rel}$	Dimensionless solid volume of debris	[m ³]
$v_{y,req}$	Required downward flow velocity for downward transport of a log to the culvert inlet	[m/s]

Introduction

1.1. Motivation

In July 2021 heavy rainfall caused severe floods in Western Europe. One of the most affected rivers in The Netherlands was the Geul. Within its catchment a lot of damage was caused near the village of Bunde. Here the Juliana Canal crosses the landscape, and the Geul river can only flow further downstream through an inverted siphon passing underneath the canal. However, during the July 2021 flood event the capacity of this inverted siphon proved to be significantly lower than the incoming Geul discharge, causing the water to flow into urban areas instead.

The primary reason why the inverted siphon's capacity was insufficient during this flood event was the simultaneous high water level downstream. Less than two kilometers further downstream there is the confluence of the Geul and the Meuse, a much larger river. High water levels on the Meuse reduced the available water head over the inverted siphon, decreasing its capacity.

The capacity of the inverted siphon was further reduced by debris accumulations at the inlet. Large branches and logs being carried by the Geul were unable to pass through the structure and caused partial blockage of the inlet. Figures 1.1 and 1.2 show pictures of this debris taken right after the flood event. Figure 1.3 shows a picture taken in February 2022 of another debris accumulation during a high discharge event, reflecting winter conditions. It shows that the inverted siphon is prone to accumulation of debris at its inlet.



Figure 1.1: Debris at the Geul inverted siphon (1)
(Deltares, 2022)



Figure 1.2: Debris at the Geul inverted siphon (2)
(Deltares, 2022)



Figure 1.3: Debris at the Geul inverted siphon (3)

1.2. Problem statement

Partial blockage of an inverted siphon's inlet increases the energy head loss over the structure. Given a fixed discharge and downstream water level, the upstream water level will rise due to such a blockage. This effect is called backwater rise. If the embankments upstream are not high enough the river will overflow, which may cause significant flood damage to nearby buildings and infrastructure. When this happens water levels are unlikely to rise much further. As a result, the discharge through the inverted siphon becomes less, which is referred to as a conveyance capacity reduction.

For the purpose of flood management it is essential to predict the backwater rise associated with blockage due to debris. However, the processes that determine this effect are not well understood, and as a result reliable design equations are either not available or apply only to specific scenarios.

1.3. Hydraulics of culverts and inverted siphons

Inverted siphons and culverts share many similarities. The key difference is that culverts usually have a straight barrel, whereas inverted siphons are characterised by a barrel that goes down, passes underneath an obstacle and then goes back up to the surface. Three inverted siphons are described in Appendix A, including the Geul inverted siphon.

Culverts can experience various flow patterns. Which flow pattern or behaviour will occur is mainly governed by the discharge, downstream water level and resistance of the culvert. The classification of flow patterns is based on several characteristics. The first characteristic is whether there is free-surface flow at the inlet or if the inlet is submerged. The second characteristic is whether there is free-surface flow or pressurised flow inside the barrel. The third characteristic is whether the flow is inlet-controlled or outlet-controlled. When the flow is outlet-controlled an increase in downstream water level will cause a direct increase in upstream water level, which will not happen when the flow is inlet-controlled.

Figure 1.4 illustrates a classification of eight flow patterns based on the aforementioned characteristics based on Chanson (2004). The solid red box indicates the flow pattern that is the main focus of this thesis, as inverted siphons typically experience this flow pattern because the drop in elevation causes a pressurised flow. The flow pattern marked by the dashed red box was only considered briefly because it occurs only under very specific circumstances.

1.4. Knowledge gaps

Various research has been performed on floating debris accumulations at hydraulic structures. A literature study was performed and documented in Chapter 2 to identify gaps in existing knowledge. This study showed that both the development of floating debris accumulations, as well as the effect of floating debris accumulations on flow hydraulics, are not always well understood because the underlying mechanisms can vary greatly across different types of hydraulic structures.

Perhaps the most well understood type of structure regarding floating debris are (rows of) vertical piles. Various papers (e.g. Schmoker & Hager, 2013; Schalko et al., 2019) have explored both the development of accumulations as well as their effect on flow hydraulics, and design equations have been proposed. Check dams have also been analysed in recent literature (e.g. Piton et al., 2020). Schalko et al. (2019) recommend further research towards debris accumulation and backwater rise for a wider range of hydraulic structures.

Follett et al. (2020) showed that backwater rise due to debris can be described analytically by means of a momentum balance, rather than empirically, as is most common in literature. However, their paper focused only on rectangular accumulations between two rows of pile structures, and similar analyses for other types of hydraulic structures are lacking.

Understanding of debris accumulation and backwater rise for submerged and outlet-controlled culverts and inverted siphons is lacking, and design equations are not available. Some research is available for ice at inverted siphons by Fu et al. (2020) and they stress the importance of including this in design guidelines, but it is unclear to what extent analogies can be made with natural wood.

Scale experiments are a proven method and literature offers various recommendations regarding the use of wood in scale testing. Literature concludes that underlying mechanisms of debris accumulation are not understood well enough yet for accurate numerical modelling of log-structure interactions (De Cicco, 2017).

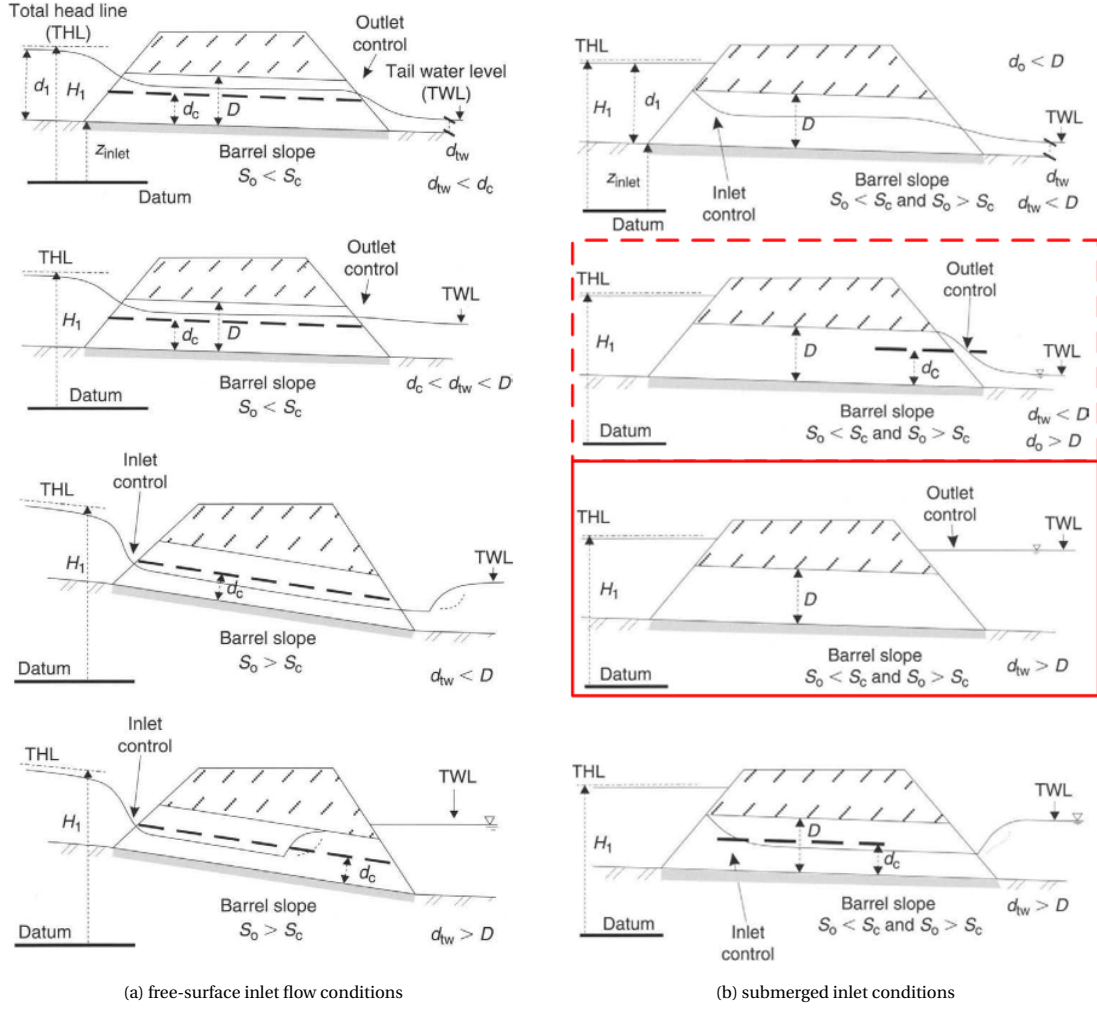


Figure 1.4: Classification of culvert flow patterns (Chanson, 2004). Figure (a) shows free-surface inlet flow conditions. Figure (b) shows submerged inlet conditions. The solid outlined flow pattern is the focus of this thesis. The dashed outlined flow pattern was found to be difficult to replicate in laboratory conditions and was not considered further in this thesis.

1.5. Research objectives

Based on the knowledge gaps summarised in Section 1.4, three research objectives were determined for fully submerged and outlet-controlled culverts and inverted siphons.

Fundamental understanding of the development of floating debris accumulations was found to be lacking. Therefore the first objective is to understand the mechanisms that govern the development of debris accumulations.

For these structures and flow conditions the energy loss associated with a developed debris accumulation originates from two sources. First partial blockage of the culvert inlet, caused by a grate or debris, generates a form drag. Second, skin friction within and below the accumulation generates a canopy drag. Both of these terms contribute towards debris head loss, but little knowledge on the influence of each of these terms is available. Therefore the second objective is to examine the influence of canopy drag and form drag due to inlet blockage on debris head loss.

The last objective is to find a method to predict backwater rise due to floating debris accumulations. This objective requires the results of the first and second objective.

The research objectives can be summarised as follows:

1. Understand the mechanisms that govern the development of floating wooden debris accumulations at fully submerged culverts and inverted siphons.
2. Determine the influence of canopy drag and form drag due to inlet blockage on debris head loss at fully submerged culverts and inverted siphons.
3. Develop a method to predict backwater rise due to floating wooden debris accumulations at fully submerged culverts and inverted siphons.

1.6. Research approach

A literature study concluded that scale experiments are a proven and effective means of researching debris accumulations consisting of natural wood at various hydraulic structures. Numerical modelling was not deemed to be a suitable option because the underlying mechanisms of debris accumulation are not yet fully understood. Therefore it was decided to conduct scale experiments to meet the research objectives. The experimental setup consisted of a culvert scale model placed in a flume, and was used for two measurement campaigns. The first campaign consisted of gradual addition of debris to the flume under different flow conditions, to study the development of debris accumulations as well as measure backwater rise. During the second campaign flow velocities were measured below developed debris accumulations by means of Laser Doppler Anemometry. The results were then analysed to calculate flow velocities through the debris accumulation, and subsequently a momentum balance equation was applied to determine the influence of canopy drag and form drag due to inlet blockage. Using these insights, a theoretical framework was developed which was then combined with a regression analysis to find a design equation to predict backwater rise.

1.7. Thesis layout

The literature review is documented in Chapter 2 of this report. Chapter 3 describes the experimental setup, type of scaling and the experiment plan, as well as the debris mixture that is representative of real conditions. Next, in Chapter 4 the backwater rise measurements without debris are compared to equations from literature. Subsequently the development of debris accumulations is described in Chapter 5. Velocity measurements are analysed in Chapter 6, to derive flow velocities through the debris accumulation and determine the influence of canopy drag and form drag due to inlet blockage. In Chapter 7 the theoretical framework to describe backwater rise is derived, which is then combined with a regression analysis to find a design equation that predicts backwater rise. Chapter 8 provides a discussion on the methods used throughout this report. Finally, conclusions and recommendations for further research are given in Chapter 9.

Literature review

2.1. Development of debris accumulations at pile structures

The development of debris accumulations has mostly been researched at pile structures. According to Schmoker & Hager (2013), under natural circumstances, the accumulation at pile structures can be divided into two phases: "(1) Initial accumulation at debris rack with major backwater rise, and (2) Formation of debris carpet with minor backwater rise". However, it is not clear if the minor backwater rise observed in phase 2 was caused by further compaction of the accumulation at the rack or by the presence of the carpet itself.

Schalko et al. (2019) describe this process by means of a parameter $V_{s,rel}$, defined as V_s/Bh_0^2 (with V_s the solids volume of debris, B the flow width and h_0 the approach flow depth). Initially, for $V_{s,rel} \leq 20\%$, floating debris hits the piles at water level h_0 . This increases the flow velocity below the accumulation and additional debris is dragged to the bottom part of the accumulation until $V_{s,rel} \approx 40\%$. Up to this point the shape of the accumulation can be approximated box-like. Further addition of debris will cause the formation of a floating carpet but will also serve to compact the accumulation. This process is shown in Fig. 2.1.

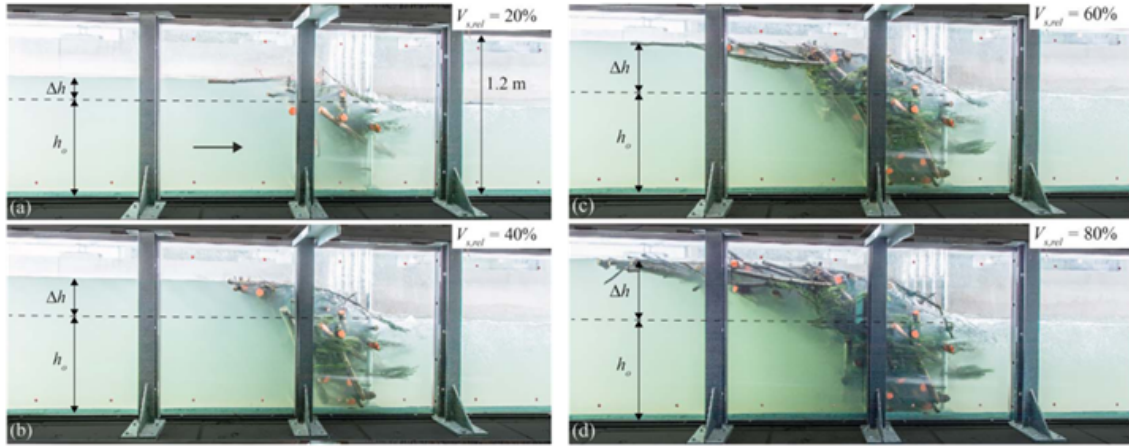


Figure 2.1: Development of debris accumulation at pile structure (Schalko et al., 2019)

A different process occurs at individual bridge piers with a large spacing such that logs cannot span the distance between two piers. In this scenario three stages can be defined: an unstable phase with rapid growth of the pier accumulation, a stable phase that only experiences moderate changes in the accumulation structure, and finally a critical phase where the accumulation breaks free from the pier through an oscillatory rotational movement (Panici & de Almeida, 2018).

2.2. Backwater rise at pile structures

There are design equations available to compute the backwater rise due to debris accumulation at pile structures. Schmoker & Hager (2013) found Eq. 2.1 for $0.5 < F_o < 1.5$. While this equation is only of limited use as it does not indicate the effect of different parameters and because it is derived for an arbitrary debris volume, it does show clearly how the backwater rise increases with the approach flow Froude number F_o .

$$\frac{h}{h_0} = 1.4 + 1.9F_o \quad (2.1)$$

A better method in the form of Eq. 2.2 is derived by Schalko et al. (2019). Instead of only being a function of the approach flow Froude number, it also incorporates the ratio between accumulation length and mean log diameter, the organic fine material fraction, and the bulk factor of the accumulation.

$$\frac{\Delta h}{h_0} = 5.4 \cdot f_A \cdot LW_A = 5.4 \cdot f_A \cdot \frac{F_0 u^{1/3} (9FM + 1)}{a_b^{4/3}} = 5.4 \cdot f_A \cdot \frac{F_0 (V_s / (h_0 B d_{lm}))^{1/3} (9FM + 1)}{a_b} \quad (2.2)$$

Δh = backwater rise; h_0 = approach flow depth; f_A = accumulation type factor; LW_A = large wood accumulation factor; F_0 = approach flow Froude number; u = flow diversion factor = ratio between accumulation length and mean log diameter; FM = organic fine material (as percentage of total debris volume); a_b = bulk factor = $\frac{1}{1 - \text{porosity}}$.

$f_A = 1.00$ for predefined box-like accumulation

$f_A = 0.55$ for natural accumulation with fixed bed

In addition they defined a characteristic volume V_c . It is defined such that the derivative of $\Delta h / h_0$ with respect to V_s is equal to 0.10. They found the empirical Eq. 2.3 to predict this volume.

$$V_c = 3.1 F_0 B h_0^2 \quad (2.3)$$

2.3. Backwater rise at check dams

Piton et al. (2020) researched the head losses and release conditions of large wood at several kinds of check dams that are most common in France and Japan. For overflowing structures they found $\Delta h / h_0$ values ranging 0%-50%. For slit and slot dams $\Delta h / h_0$ ranged between 5% and 60%. For widely open structures such as sabo dams and racks $\Delta h / h_0$ ranged between 20% and 100% for subcritical approaching flows and between 170% and 230% for supercritical flows, but the high values for supercritical flow are caused by a low h_0 .

In addition, Piton et al. (2020) also found that large wood is often released over the aforementioned structures when the overflowing depth is about 3-5 times the mean log diameter. This effectively creates an upper limit of the maximum backwater rise caused by debris that can occur at the crest of an overflow structure, as the debris will be released when the backwater rise becomes high enough. However, at the upstream end of the debris accumulation a larger backwater rise is still possible.

2.4. Scale testing with debris

Schmoker & Hager (2013) studied which aspects are important in scale modelling of wooden debris accumulation at pile debris racks. Before each test the debris was watered for a certain time. For watering times up to 8 hours they found no effect, but the debris that had been watered for 1 week lost its floatability and was transported at the channel bottom instead. They recommend to soak debris for a maximum of some hours when modelling flood durations of up to 2 days. Test duration only showed to be important in relation to compaction processes. Furthermore, they found that the effect of the debris mixture on backwater rise is relatively small. Although the backwater rise should increase for finer mixtures (because fine logs may fill the gaps between large logs and increase the accumulation density), they found no consistent trend for $F_0 = 0.5$ and only a minor effect for $F_0 = 1.5$.

Schalko (2018) performed a scale series of experiments to measure backwater rise due to debris jams between two rows of pile structures. Identical tests according to Froude similitude were performed for three scale factors: $\lambda = 50$, $\lambda = 30$ and $\lambda = 6$. In addition, different packing densities were tested. Generally the results from all three scales did not deviate much, and no scale effect could be identified based on the observed backwater rise. The only notable deviations were observed when combining $\lambda = 50$ with a loose accumulation, but this was likely due to difficulties in the manual model setup at a small scale, so scale factors ≤ 50 are recommended.

2.5. Blockage of culverts

Azam et al. (2022) performed flume experiments to study the blockage of box-shaped and circular culverts under flood event conditions. They placed wooden dowels in the flume upstream of the culvert one by one, and the discharge was changed repeatedly to create a stepwise hydrograph. Under these conditions the accumulation of wooden dowels at the culvert inlet was measured, as well as the dowels that passed through the culvert. They concluded that the dowels' diameter has a larger effect on blockage than the feeding rate. For woody debris with a small diameter the blockage increases with feeding rate, but for woody debris with a large diameter they found no significant variation. Furthermore, the maximum percentage of blockage of the culvert cross-section for smaller diameter dowels was 42% higher than the blockage corresponding to larger diameter dowels.

2.6. Numerical modelling of debris

Niroshinie & Nihei (2016) developed a numerical model to calculate backwater rise and floodplain inundation for the Yoshino River in Yamagata, Japan, at various bridges that got blocked by woody debris during a flood in July 2013. They used a 1D river model based on the Saint Venant equation and coupled this to a 2D basin model based on the Navier Stokes equations (assuming depth-averaged flow). This model was then calibrated using field observations of water levels from the July 2013 flood event. The debris flock initially acted like a sluice gate and after gradual inclination acted like a radial gate, but they found that the debris' effect on water level could be reproduced by only changing the cross section and Manning's roughness at the bridges. Their paper, however, does not answer how the correct cross section and Manning's coefficient can be determined based on debris properties rather than field observations of water levels.

Based on a literature study De Cicco (2017) concluded that existing 2D numerical models describe well the transport of cylindrical shaped wood, but that further improvements are necessary to reduce uncertainties such as modelling real logs (instead of cylinders), and improvements towards the modelling of log-structure interactions to accurately include the effect of backwater rise.

2.7. Analytical modelling

Follett et al. (2020) developed an analytical equation to compute head loss due to debris jams based on momentum loss. It is an adapted form of a canopy drag model to represent channel-spanning debris jams. Their initial equation consisted of a balance between hydrostatic forcing, change in momentum and drag within the jam. However, they found the net change in momentum to be only 13% of the hydrostatic forcing and determined this term to be negligible. The final equation is given in the form of Eq. 2.4, with the dimensionless parameter C_A to represent jam geometry defined as Eq. 2.5. As many pairs of H_2 and H_3 satisfy Eq. 2.4, the constraint of minimum energy loss through the jam should be added. Validation by means of 584 experiments showed that measured values correspond well to Eq. 2.4.

$$H_2^2 - H_3^2 - \frac{C_A q^2}{g H_3} = 0 \quad (2.4)$$

$$C_A = LC_D a_f / (1 - \phi)^3 \quad (2.5)$$

H_2 = water level before debris jam; H_3 = water level after debris jam; C_A = dimensionless parameter to represent debris jam geometry; q = discharge per unit width; g = gravitational acceleration; L = debris jam length; C_D = drag coefficient of logs (typically 0.9-1.0); a_f = spatially averaged frontal area per debris jam volume; ϕ = solid volume fraction.

The method described here is one of the few analytical, non-empirical, equations available (only the drag coefficient C_D is empirical) to compute energy or momentum loss due to debris accumulations. It stands to reason that this or a similar approach could be implemented in numerical models to compute backwater rise due to a given debris accumulation geometry and flow conditions.

2.8. Floating ice at inverted siphons

Fu et al. (2020) conducted an experimental study to gain insights into ice accumulation characteristics upstream of inverted siphon inlets and into the resultant impacts on the flow hydraulics in the upstream channel. They partitioned the flow into two non-interactive zones delineated by the position of zero shear stress according to a method by Calkins et al. (1982). The velocity in the upper zone is then influenced by the floating ice, and in the lower zone it is influenced by the bed. Subsequently the hydraulic resistance for the composite cross-section was calculated based on a theoretical method by Zare et al. (2016). They found backwater rise to be substantial and highlighted the importance of taking this effect into consideration in design requirements.

By comparing the composite Manning's coefficient n_c to the open-water Manning's coefficient n_o Fu et al. derived Eq. 2.6 to predict the backwater rise in the channel upstream of an inverted siphon. They also found an empirical relation between the Froude number and dimensionless ice thickness $\bar{t}/H$.

$$\frac{\Delta H}{H_o} = \frac{\left(4 \left(\frac{n_c}{n_o}\right)^3\right)^{\frac{1}{5}}}{1 - \frac{\rho_i}{\rho_w} \cdot \frac{\bar{t}}{H}} - 1 \quad (2.6)$$

The composite Manning's coefficient n_c can be obtained from the composite Darcy-Weisbach resistance coefficient which is defined as Eq. 2.7.

$$f_c = \frac{\left(\frac{R_c}{R_i}\right)(U_m - \bar{U}_i)^2 + \left(\frac{R_c}{R_b}\right)(U_m - \bar{U}_b)^2}{\left(\frac{U_m - \bar{U}_i}{\sqrt{f_i}} \cdot \frac{h_i}{h_c} + \frac{U_m - \bar{U}_b}{\sqrt{f_b}} \cdot \frac{h_b}{h_c}\right)^2} \quad (2.7)$$

Figure 2.2 illustrates the parameter definitions. The subscript o is for open-water conditions. The subscript c is for the composite conditions. The subscript b denotes the bed, and the subscript i denotes the ice.

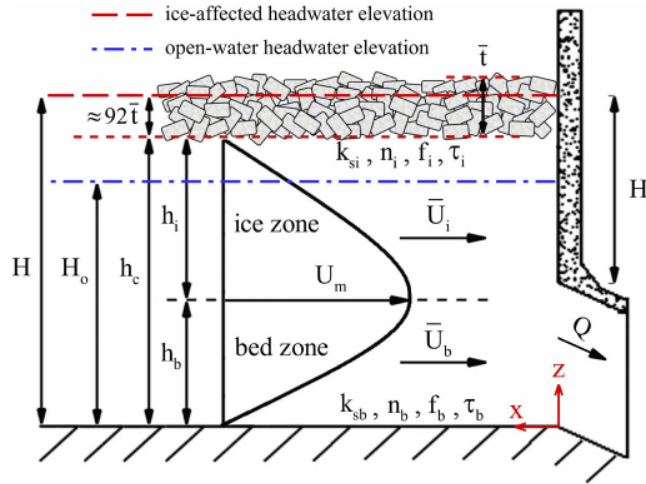


Figure 2.2: Sketch with symbols (Fu et al., 2020)

One remark is that the plane of zero shear stress was assumed to be equal to the plane of maximum velocity, while the ice zone and bed zone are actually delineated by the plane of zero shear stress. A study by Tatinciaux & Gogus (1983) showed that in reality the plane of maximum velocity does not always coincide with the plane of zero shear stress. However, Fu et al. (2020) state that this assumption leads to variations in ice jam characteristics of about 10% and thus conclude that this discrepancy is negligible for ice covered flows.

Experimental setup and approach

3.1. Prototype and scale model

The experiments were performed using a simple culvert scale model placed in a flume as illustrated in Figure 3.1. Because in all experiments fully submerged and outlet-controlled conditions (selected flow pattern from Section 1.3) were maintained, the results from experiments with this culvert scale model are applicable for inverted siphons as well. The only difference is that when inverted siphons are considered, an additional loss factor for the vertical bends should be considered.

Figure 3.1 gives the dimensions of the prototype in blue and of the scale model in red. The width of the flume was 0.40 m. The opening height had to be chosen such that on one hand it was large enough that no major scale effects occurred, but on the other hand small enough that sufficient backwater rise could occur even for submergence rates up to 2.0. A balance was found at a height of 0.12 m.

Taking an opening width of 0.40 m and a height of 0.12 m, the geometry is comparable to a 1:20 scale version of the rectangular inverted siphons included in Appendix A, namely the Geul and the Geleenbeek inverted siphons. Furthermore, the debris mixture (see Section 3.7) was mainly based on natural wood found at the Geul inverted siphon. By setting the scale at 1:20, consistency between the opening geometry and debris dimensions could be guaranteed.

The length of the prototype was set at four times the flume width, since to model a realistic culvert or inverted siphon the length should be several times larger than both the width and the height.

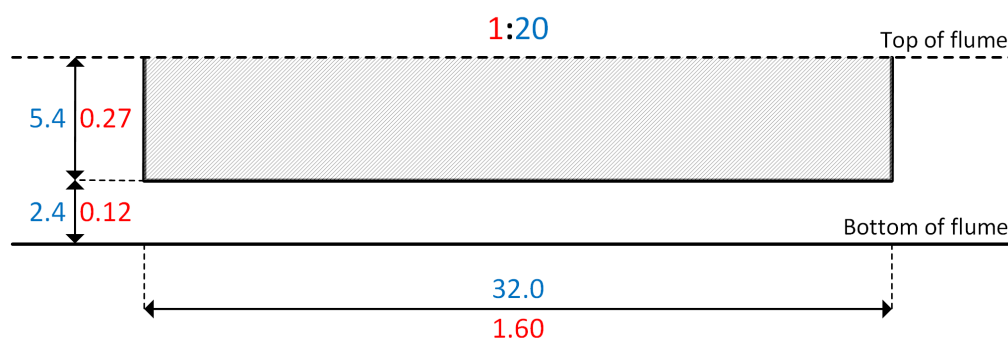


Figure 3.1: Dimensions of the prototype (blue) and model (red), expressed in meter.

At the inlet a metal grate was placed consisting of 1 mm thick bars with a center-to center distance of 5 mm in both the horizontal and vertical direction. These dimensions were chosen keeping in mind the smallest class of the debris mixture as given in Section 3.7. The grate openings of 4 by 4 mm are the largest possible openings that the smallest class of the debris mixture is unable to pass through easily.

The grate was supported by four vertical cylindrical bars with a diameter of 8 mm. One bar was placed near each wall of the flume, and the remaining two bars were spread evenly across the width. The bars and grate are shown in Figure 3.5.

3.2. Similitude and scaling

Variables

In order to analyse and limit potential scale effects, a dimensional analysis is performed using the Buckingham Pi theorem. First all relevant parameters are determined, such that no parameter can be described as a combination of other parameters. These are summarised in Figure 3.2.

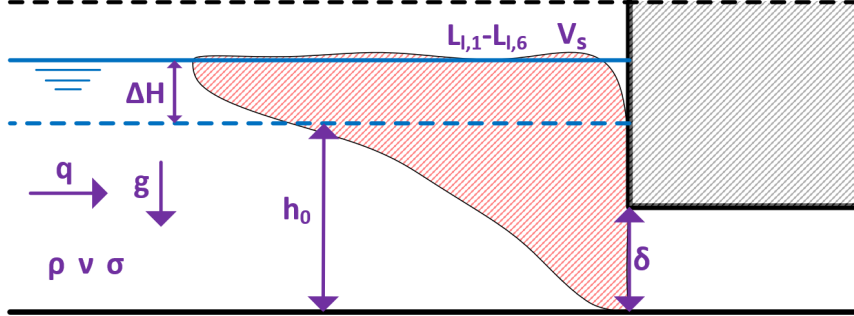


Figure 3.2: Variables. Red area indicates the debris accumulation. Grey area indicates the culvert.

Below all variables are expressed as a function of their base dimensions length [L], time [T] and mass [M].

- | | | | |
|---|------------------------------|---|--------------------------------|
| • ΔH [L] | (head loss) | • V_s [L ³] | (solids volume of debris) |
| • h_0 [L] | (initial depth upstream) | • $L_{l,1}-L_{l,6}$ [L] | (log length per class) |
| • q [L ² T ⁻¹] | (specific discharge) | • ρ [M L ⁻³] | (density of water) |
| • g [L T ⁻²] | (gravitational acceleration) | • ν [L ² T ⁻¹] | (kinematic viscosity of water) |
| • δ [L] | (opening height) | • σ [M T ⁻²] | (surface tension of water) |

It should be noted that the head loss ΔH is the increase in head difference as a result of the floating debris, and does not include any head difference that was already present before the introduction of debris. The definition of ΔH in Figure 3.2 assumes a constant water level downstream before and after adding debris.

The log diameter is not included here because for every log size class the length to diameter ratio is 20, as explained in Section 3.7. As such, its scaling follows directly from the log length scale factor.

Dimensionless parameters

The head loss ΔH is chosen to be the dependent variable.

$$\Delta H = f(h_0, q, g, \delta, V_s, L_{l,1}-L_{l,6}, \rho, \nu, \sigma)$$

As there are $n = 14$ independent variables and $r = 3$ reference dimensions ([L], [T], [M]), $n - r = 11$ dimensionless parameters are required. These dimensionless parameters are a combination of r repeating variables and each of the remaining independent variables. The repeating variables are h_0 , q and ρ , representing the dimensions of length, time and mass respectively.

- $\Pi_1 = f(g, h_0, q, \rho) = q / \left(g^{\frac{1}{2}} h_0^{\frac{3}{2}} \right) = F_0$
- $\Pi_2 = f(\delta, h_0, q, \rho) = \frac{h_0}{\delta}$
- $\Pi_3 = f(V_s, h_0, q, \rho) = V_s / (b h_0^2)$, with b being the width of the flume¹
- $\Pi_4 - \Pi_9 = f(L_i, h_0, q, \rho) = L_i / h_0$
- $\Pi_{10} = f(\nu, h_0, q, \rho) = \frac{q}{\nu} = Re$
- $\Pi_{11} = f(\sigma, h_0, q, \rho) = \frac{q^2 \rho}{\sigma h_0} = We$

¹Here b is used to give a volume per unit width. It is not listed as an independent variable because no 3D effects are considered.

Scaling law

A physical scale model has complete similarity to its real-world prototype when it has geometric, kinematic and dynamic similarity (Heller, 2011). Geometric similarity requires similarity in shape and thus only includes the length dimension. Kinematic similarity also includes the time dimension and requires similarity of motion such as constant ratios of velocity and discharge in the model and prototype. Dynamic similarity also requires that all force ratios are identical, and thus also includes the mass dimension.

Froude similarity will be applied as it is especially suitable for short and highly turbulent phenomena, but as a result it is not possible to also have Reynolds similarity and Weber similarity (Heller, 2011). However, scale effects due to a difference in Reynolds number are usually negligible if both the prototype and the model experience fully turbulent flow. Assuming $\nu = 1 \cdot 10^{-6} \text{ m}^2/\text{s}$ for water, a specific discharge q of at least $0.01 \text{ m}^2/\text{s}$ is required to obtain a Reynolds number of $1 \cdot 10^4$ or higher, which is above the threshold for turbulent flow. Because all experiments use a specific discharge of at least $0.023 \text{ m}^2/\text{s}$, Reynolds similarity may be disregarded. See Table 3.1 for an overview of all Reynolds numbers that were present during experiments.

Weber similarity is usually not relevant as surface tension is negligible for most prototypes in hydraulic engineering except for problems dealing with air entrainment or small water depths (Heller, 2011), and thus Weber similarity may be disregarded as well.

Scale factors

Scale factors are defined as $\lambda = x_P / x_M$, with x_P being the value for a property in the real world prototype and x_M the value for that same property in the scale model. The variables that only consist of a length dimension all have an equal scale factor and are chosen based on the dimensions of the available flume.

- $\lambda_\delta = \lambda_{h_0} = \lambda_{V_s} = \lambda_{L_i} = \lambda_{\Delta H} = 20.0$

From Π_1 and $\lambda_g = 1$ the following scale factor for the specific discharge follows.

- $\lambda_q = \lambda_\delta^{1.5} = 20.0^{1.5} = 89.4$

3.3. Experimental setup

Experiments were conducted in a flume at the WaterLab of the Delft University of Technology. An overview of the setup is shown in Figure 3.3. The inner width of the flume was 40 cm, the available height about 39 cm, and the length was 14 meter. Inside the flume an elevated obstruction was placed to represent a culvert, as shown in Figure 3.4.

By gradually dropping floating debris into the flow just downstream of the flow stabiliser, natural accumulations of debris developed at the culvert. Such debris accumulations cause an increase in water level at the upstream side of the culvert, also referred to as backwater rise. A grate was placed at the culvert inlet to prevent debris from passing through, as shown in Figure 3.5. The debris consisted of natural branches. The mixture of branches is elaborated upon in Section 3.7.

The discharge through the flume was manually adjusted by opening or closing the supply valve. The flume was supplied by a large reservoir placed several meters higher than the flume. The water level of the reservoir was maintained at a constant level by a system of pumps recirculating the water leaving the flume. Discharge was measured by an ultrasonic flow meter placed at the supply pipeline.

At the downstream end of the flume a moving gate was used to control the water level. Surface elevations were measured with point gauges. As the culvert caused a backwater curve, the upstream point gauge was located at a distance of about 30 times the opening height of the culvert to measure the equilibrium depth.

Flow velocities were measured through Laser Doppler Anemometry (LDA). A cart carrying the LDA device was placed upon the rails on the sides of the flume, as shown in Figure 3.6. On one side of the flume the laser emitter was placed, and on the other side the receivers, as shown in Figure 3.7. Horizontal translation could be performed by moving the cart along the rails, and vertical translation by raising or lowering the frame. The system was configured such that both translations could be performed without any relative motion between the emitter and receivers. The device used a He-Ne laser with a 632.8 nm wavelength. It measured velocity in an ellipsoidal volume of approximately 0.1 mm^3 . The measurement frequency was set to 1000 Hz. The applicable velocity range is 0.001 - 2 m/s, with a repeatability of 0.5 mm/s. The laser beams were inclined upward allowing for measurements close to the debris carpet, but as a result the lowest possible measurements were

approximately 25 mm above the flume bed. The device was able to measure both the horizontal and vertical velocity component at each location.

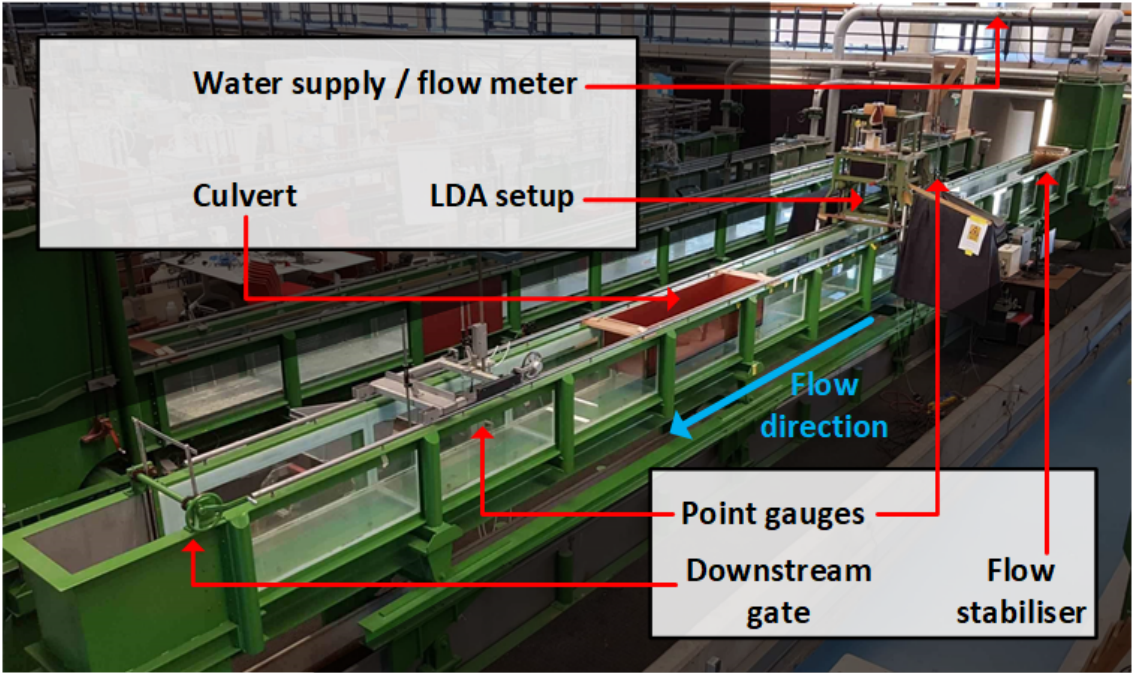


Figure 3.3: Overview of the test setup. The flow direction is from right to left.



Figure 3.4: Side view of the culvert



Figure 3.5: Front view of the inlet grate



Figure 3.6: Cart with LDA setup



Figure 3.7: LDA emitter and receivers

3.4. Test procedure

Each experiment was defined by a combination of upstream Froude number and the submergence rate of the culvert inlet. The submergence rate is defined as the initial equilibrium water level upstream h_0 divided by the opening height of the culvert δ . From the desired water level and Froude number, the required discharge could be obtained. To create the desired flow conditions, first the required discharge was set. Once achieved, the gate was raised or lowered until the desired water level and Froude number were in place.

The flume was slowly filled with water until both the inlet and outlet were fully submerged. If the flume was filled too fast, air pockets were trapped inside the culvert near the inlet. Such air pockets introduce another loss factor that is difficult to account for and thus should be avoided.

After the correct flow conditions were set up, debris was added to the flow near the upstream end of the flume so that it accumulated naturally at the grate. The total debris volume was divided into ten equal batches, which were added one batch at a time. This allowed the measurement of backwater rise for different volumes of debris. After adding a batch of debris, it took about 3 minutes until the accumulation and water level reached a near-equilibrium, after which water levels were measured and another batch of debris was added.

Flow velocity data was obtained using through Laser Doppler Anemometry (LDA). In order to obtain a velocity profile sufficient data points had to be taken in the vertical direction. At each point the horizontal and vertical velocity component were measured for 3 minutes at 1000 Hz to get representative values for both mean velocities as well as velocity fluctuations.

3.5. Flow conditions and experiment plan

Two measurement campaigns were conducted. The first campaign consisted of gradual addition of debris to the flume, to measure backwater rise and in the process study the development of debris accumulations. The second campaign consisted of taking velocity measurements below several debris accumulations, to support a more fundamental understanding of the source of energy losses.

Backwater rise

Table 3.1 shows the experiment plan for backwater rise experiments. The governing parameters that define each experiment are the submergence rate h_0/δ and the initial upstream Froude number F_0 . The columns for Reynolds number and V_c are included for convenience, but they can be derived directly from the aforementioned governing parameters and other constants.

The lowest possible submergence rate was 1.1, because any lower water level at the inlet would trap air inside the culvert, which in turn might lead to inlet-controlled conditions. The largest submergence rate was 2.0, to prevent overflowing of the flume after adding debris.

Table 3.1 summarises which combinations of submergence rate and initial upstream Froude number were tested. The experiment plan always includes the maximum possible Froude number (by increments of 0.05) for each submergence rate, such that the desired flow pattern with fully submerged and outlet-controlled conditions (Section 1.3) was maintained in the experimental setup. A minimum of $F_0 = 0.15$ was applied.

Note that the submergence rate of 1.4 and Froude number of 0.25 were selected to collect the most elaborate data. The effect of submergence rate can be well identified by running experiments with six different submergence rates for the fixed Froude number of 0.25. Similarly, the effect of Froude number becomes more apparent through testing of five different Froude numbers for the fixed submergence rate of 1.4.

All experiments were conducted three times. Since the debris accumulates naturally there can be stochastic differences between repetitions of the same configuration, both in terms of backwater rise and the shape of the debris accumulation. By using at least three repetitions the reliability greatly increases because the averages are more reliable due to a larger sample size, but also because it allows for better recognition of outliers in the data than would have been possible with only one or two repetitions.

Table 3.1: Experiment plan. Each unique experiment is defined by a submergence rate (h_0/δ) and initial upstream Froude number (F_0). The Reynolds number and characteristic volume follow directly from the submergence, Froude number and constants.

Label	Submergence	Froude number	Re [-]	V_c [L]	Remarks
1A	1.1	0.15	$2.3 \cdot 10^4$	3.2	
1C	1.1	0.25	$3.8 \cdot 10^4$	5.4	
1D & 1E	1.1	0.30 & 0.35	N/A	N/A	Inlet-controlled upon adding debris.
2C	1.25	0.25	$4.5 \cdot 10^4$	7.0	
2E	1.25	0.35	$6.4 \cdot 10^4$	9.8	
3A	1.4	0.15	$3.2 \cdot 10^4$	5.2	Measured velocity profiles.
3B	1.4	0.20	$4.3 \cdot 10^4$	7.0	
3C	1.4	0.25	$5.4 \cdot 10^4$	8.7	Measured velocity profiles.
3D	1.4	0.30	$6.5 \cdot 10^4$	10.5	
3E	1.4	0.35	$7.5 \cdot 10^4$	12.2	Measured velocity profiles.
4A	1.6	0.15	$4.0 \cdot 10^4$	6.9	
4C	1.6	0.25	$6.6 \cdot 10^4$	11.4	
4D	1.6	0.30	$7.9 \cdot 10^4$	13.7	
5C	1.8	0.25	$7.9 \cdot 10^4$	14.5	
5D	1.8	0.30	$9.4 \cdot 10^4$	17.4	
6A	2.0	0.15	$5.5 \cdot 10^4$	10.7	
6C	2.0	0.25	$9.2 \cdot 10^4$	17.9	
6D	2.0	0.30	$11.0 \cdot 10^4$	21.4	Only 10% of debris, to prevent overflowing.

Velocity profiles

Experiments 3A, 3C and 3E were selected for the measurement of velocity profiles. They correspond to a submergence rate of 1.4, combined with Froude numbers of 0.15, 0.25 and 0.35 respectively. Velocity profiles were first measured without debris, and then measured at (approximately) the same locations after adding all debris to allow for comparison.

Table 3.2 shows the locations of where velocity profiles were measured. The top row indicates the distance upstream of the inlet, expressed in multiples of the opening height δ , the other rows give the exact locations expressed in cm. Some of the measurements were shifted horizontally (± 1 -3 cm) to a location with better lines of sight, to enable measurements closer to main body of the debris carpet. All measurements were taken in the middle of the flume. Note that the location of -10δ is inside the culvert at 3/4 of its length.

Measurement locations in the vertical direction were determined to cover the full measurable range. Measurements were taken no more than 3 mm apart near the debris carpet, but this was increased gradually with increasing vertical distance to the debris. Without debris a vertical spacing of 1.5 cm was maintained, but also always covering the full measurable range.

Table 3.2: Locations of velocity profile measurements, expressed in cm upstream of the culvert inlet.

Approximate location	-10δ	1δ	2δ	3δ	4δ	5δ	10δ	20δ
3A, without debris	-120	12		36		60	120	240
3A, with debris	-120	12		36		60	120	240
3C, without debris	-120	12		36		60		240
3C, with debris	-120	15	26	38		65		240
3E, without debris	-120	12		36		60		240
3E, with debris	-120	15	24	36	48	60		240

3.6. Debris volume

As explained in Chapter 2 the characteristic debris volume V_c was defined by Schalko (2018) such that the derivative of $\Delta h/h_0$ with respect to V_s is equal to 0.10. This volume is given by Eq. 2.3 for debris at pile structures. In Table 3.1 the value for V_c according to Eq. 2.3 is given for each experiment.

It was decided to use a fixed volume of debris across all experiments. This has a few advantages: the mixture can always be divided into ten equal batches, and the routine that comes with fixed volumes allows for more experiments to be performed within the given amount of time.

A debris volume V_d of 7.4 L was selected. It is the largest volume that could be used during experiments 5D and 6C, as the corresponding backwater rise was such that water levels nearly reached the edge of the flume. In addition, using a volume less than V_c for the higher submergence rates saved time that could be used to perform more experiments. This was prioritised over even larger volumes, as with $V_d = 7.4$ L the ratios of V_d/Bh_0^2 are reasonably large already (ranging from 1.06 to 0.32, for $h_0/\delta = 1.1$ and 2.0 respectively).

$$V_d = 7.4 \text{ [L]}$$

3.7. Debris mixture

The debris mixture used for experiments is based on debris observed at the Geul inverted siphon as documented by Ronckers (2023, Chapter 6). Dimensions of logs were determined from pictures using visual data processing. A description of the structure is given in Section A.2. Modelling of realistic scenarios is ensured by using the Geul inverted siphon as a reference for both the structure prototype and debris mixture.

Figure 3.8 shows the discretised distribution of log sizes at the full scale of the prototype. To create this histogram all identified logs at the Geul inverted siphon were implemented in an initial histogram. However, most shorter logs in the accumulations were not distinguishable from end to end, and therefore could not be measured. For this reason the histogram was supplemented by expert judgement to account for these shorter logs. To create a workable mixture for the experiments the number of size classes was reduced to 6, and the amount of logs in the smallest class was reduced by approximately 50%. The length to diameter ratio for natural logs was assumed to be 20. The geometric scale factor of the experiments was 20 (Section 3.2).

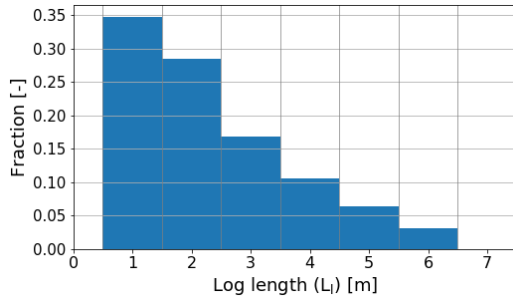


Figure 3.8: Debris mixture (prototype scale)

$L_l \text{ [m]} \times d_l \text{ [m]}$	N_{batch}	N_{total}
1.0×0.05	33	330
2.0×0.10	27	270
3.0×0.15	16	160
4.0×0.20	10	100
5.0×0.25	6	60
6.0×0.30	3	30
Total	95	950

Table 3.3: Number of logs (prototype scale)

In reality many logs shorter than the smallest class in Figure 3.8 were observed at the Geul inverted siphon. However, logs smaller than 5 cm long by 2.5 mm in diameter (smallest class, scale 1:20) were too fragile to work with, and were able to pass through the grate too easily. A few logs longer than the largest mixture class were observed as well, but it was decided to set an upper limit of log length to 3/4 of the flume width.

The required number of logs was determined using the distribution in Figure 3.8 combined with the volume V_d determined in Section 3.6. The logs were modelled as cylinders to calculate their volume, upon which a 10% volume increase was applied to account for irregularities and better represent the true volume. The results are summarised in Table 3.3.

Ruiz-Villanueva et al. (2014) did extensive research on the relation between wood density and buoyancy (defined as the submerged height divided by the log diameter). The logs used in this work correspond well to their category of "dense wood", which has a buoyancy of 21% and average density of $\rho_d = 800 \text{ kg/m}^3$.

Head losses without debris

4.1. Methodology for head loss without debris

In order to accurately isolate and determine loss factors due to floating debris, it is important to understand head losses that occur at the culvert without debris. Presented first are the loss factors that were determined from experiments. Next various loss factors are obtained from literature. This is then followed by a comparison of the measured and theoretical loss factors to validate the results.

The loss factors are defined as the ratio between the total head loss and the velocity head inside the culvert. For engineering purposes it is often more convenient to use Eq. 4.1 instead of loss factors that are defined by the upstream velocity head. This is the case especially for culverts and inverted siphons with pressurised flows, because then U_{cul} is simply the discharge divided by the cross-sectional flow area of the structure (which is fixed with varying discharge, unlike the cross-section of open-channel flows).

$$\Delta H = K \cdot \frac{U_{cul}^2}{2g} \quad (4.1)$$

4.2. Experimental head losses

Head loss over the culvert was measured with various water levels for the discharges of 6, 12, 18, 24, 30, 36 and 42 L/s. Results were obtained with and without presence of the grate at the inlet. Figure 4.1a shows the raw data, and Figure 4.1b shows the same data converted to dimensionless loss factors (K in Eq. 4.1). As expected, the absolute head loss increases with larger discharges. The loss factors give consistent results as well and show only one notable difference: the upward slope in loss factor decreases with increasing discharge.

No accurate loss factors could be determined for the discharge of 6 L/s. With the grate attached the measured head losses ranged from 2 mm to 3 mm, meaning that measurement inaccuracies related to the point gauges (about 0.5 mm for low Froude numbers) have a significant influence. In addition, very low discharge are more affected by scale and model effects. Without the grate the head losses would have been even lower and loss factors more inaccurate, and thus they were not considered in this analysis.

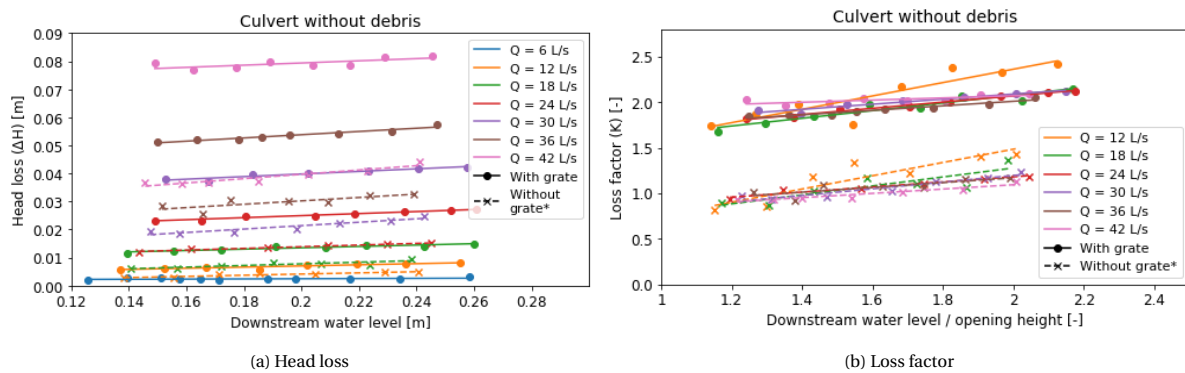


Figure 4.1: Head loss and loss factor over the culvert without debris.

4.3. Theoretical head loss according to literature

The theoretical head loss is determined as the sum of four main loss factors. These are inlet head loss, outlet head loss, friction head loss and grate head loss (including vertical bars supporting the grate).

Inlet head loss

The inlet loss coefficient for outlet-controlled box culverts is about 0.7 for culverts with wingwalls parallel to the direction of flow and with a square-edged crown (U.S. Department of Transportation, 2012, Table C.2). The submergence rate only has a small influence on the inlet loss factor as long as the inlet is fully submerged, especially when the submergence rate is larger than 1.5 (Smith and Oak, 1994).

$$K_{in} = 0.7$$

Outlet head loss

The outlet loss coefficient is often taken as 1.0 without much consideration, but Tullis (2012) found that using the Borda-Carnot expression (equation 4.2) gives much better results for outlet loss. This expression deviated at most 6.2% with experimental results for various test set-ups, whereas the simple approach of assuming 1.0 could deviate as much as 187.3%. While these experiments were performed for projecting pipes, it is safe to assume that also for non-projecting outlets equation 4.2 can be used since this is the situation for which the Borda-Carnot equation is derived.

$$K_{out} = \left(1 - \frac{A_{cul}}{A_{ch}}\right)^2 \quad (4.2)$$

A_{cul} = area of flow inside culvert [m²] = 0.048 m²

A_{ch} = area of flow in channel downstream [m²]

Friction head loss

Friction head loss should be calculated with the Darcy-Weisbach equation for pressurised pipe flow.

$$K_{fric} = f_D \frac{L}{D_{eq}} \quad (4.3)$$

D_{eq} = hydraulic diameter [m] = $4 R_h = 4 A_c / P = 0.185$ m

L = length of culvert [m] = 1.60 m

The Darcy-Weisbach friction factor f_D can be calculated with Eq. 4.4 in case of turbulent flow (Uni-Bell PVC Pipe Association, 2012, Chapter 9). The scale model and flume walls were both made of a smooth material with similar roughness as PVC, so the equivalent roughness ϵ is assumed to be 0.00021 cm (Uni-Bell PVC Pipe Association, 2012, Chapter 9).

$$\frac{1}{\sqrt{f_D}} = 1.14 - 2 \log \left(\frac{\epsilon}{D_{eq}} + \frac{9.35}{Re \sqrt{f_D}} \right) \quad \text{for } Re > 4000 \quad (4.4)$$

The Reynolds number for pipe flow is calculated according to Eq. 4.5. It has been verified that in all experiments the Reynolds number inside the culvert was well above 4000.

$$Re = \frac{D_{eq} U_{cul}}{\nu} \quad (4.5)$$

U_{cul} = average flow velocity inside culvert [m/s] = Q / A_{cul}

ν = kinematic viscosity of water [m²/s] = $1 \cdot 10^{-6}$ m²/s

Grate head loss

Equation 4.6 for the grate loss factor K_g is given by Meusburger (2002). In this equation p is the blockage ratio, which is defined as the area that is blocked by the grate divided by the total cross-sectional area. The bar shape factor k_F takes into account the shape of the grate bars, and is equal to 1.79 for cylindrical bars perpendicular to the direction of flow as reported by Meusburger (2002) after Kirschmer (1926).

$$K_g = k_F \cdot \left(\frac{p}{1-p} \right)^{\frac{3}{2}} \quad (4.6)$$

The grate consisted of 1 mm thick bars with a center-to center distance of 5 mm in both the horizontal and vertical direction. This leads to a blockage ratio p of 0.36 for just the grate itself. When only the four vertical bars that support the grate are considered, the blockage ratio would be 0.08. To compute the combined blockage ratio, the overlap between the grate and vertical bars needs to be taken into account.

$$p = 0.36 + 0.08 \cdot (1 - 0.36) = 0.411$$

The grate loss factor (including vertical bars) then becomes as follows.

$$K_g = 1.79 \cdot \left(\frac{0.411}{1 - 0.411} \right)^{\frac{3}{2}} = 1.045$$

In Section 7.3 an alternative equation in the form of Eq. 7.8 was derived. This equation results in $K_g = 1.183$ for the same blockage ratio ($p = 0.411$), which is 13% higher than the value resulting from Eq. 4.6.

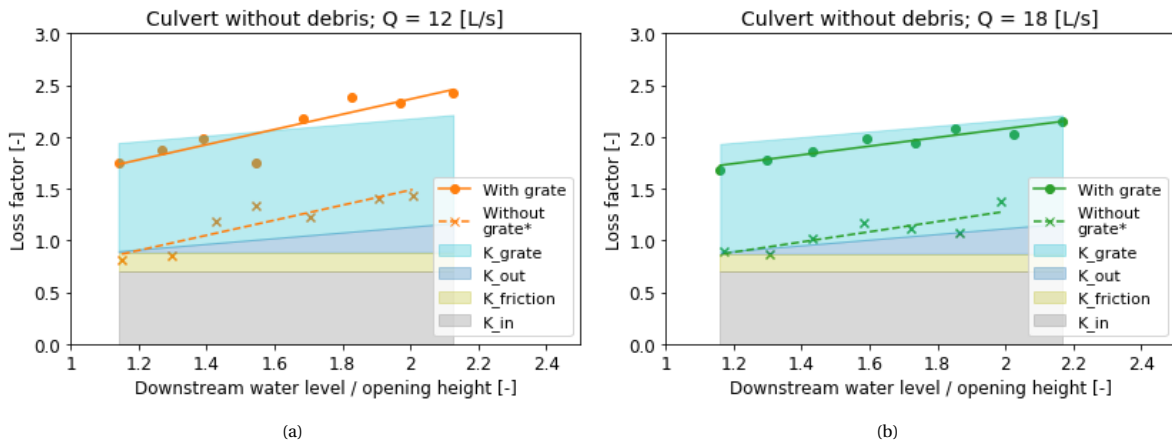
4.4. Comparison of experimental and theoretical head loss

Figure 4.2 show how the experimental and theoretical loss factors compare for the discharges of 12, 18, 24, 30, 36 and 42 L/s. The experimental data has been approximated by a linear best fit.

There is a strong match between the experimental and theoretical values. Methods from literature slightly overestimate the actual loss factor as found from experiments. The only notable mismatch between experiments and literature is found at the discharge of 12 L/s, where literature underestimates the rate in which the loss factor increases for increasing water level.

Note that the experimental results without grate did still include the four supporting bars, so the theoretical K_g should be replaced by the value of 0.046 instead of being disregarded entirely when comparing the experimental and theoretical values without grate. Taking this into account, the difference between the experimental fits with and without grate is approximately equal to the theoretical K_g .

To conclude, the literature review validates the experimental results for flow without debris.



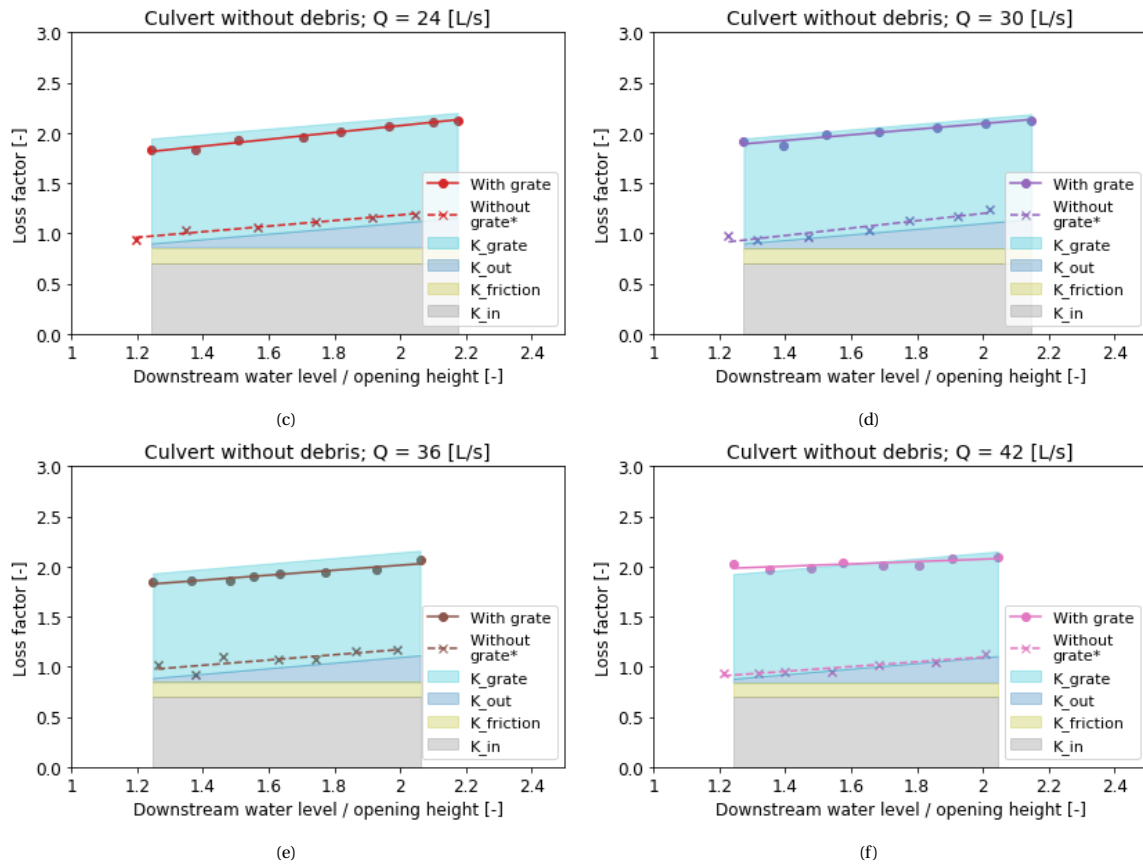


Figure 4.2: Comparison of experimental and theoretical results for head loss at the culvert without debris.

* The measurements without grate did still include the four support bars, meaning that in this case the blockage is 8%.

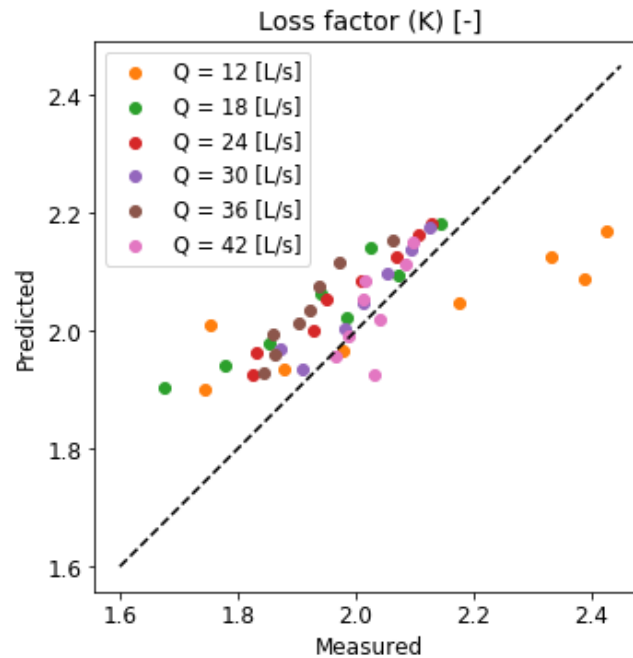


Figure 4.3: Comparison of the measured and predicted loss factors, for the situation with grate. Data points are identical to those in Figure 4.2. Line of equality is plotted in black.

5

Development of floating debris accumulations

5.1. Visual analysis

Figures 5.1, 5.2 and 5.3 show the development of the debris accumulation for one repetition of three different approach flow conditions. They correspond to the upstream Froude numbers of 0.15, 0.25 and 0.35 respectively, all at a submergence rate of 1.4. The subfigures indicate what fraction of the design volume V_d has been added.

Figure 5.1, corresponding to $F_0 = 0.15$, shows clearly how most of the debris remains floating at the surface. The flow is too weak to pull many branches down towards the grate. As a result, adding more debris mainly increases the length of the accumulation, but the backwater rise is small. When $F_0 = 0.15$ is combined with a submergence rate of 2.0, initially more debris is pulled down towards the grate due to a larger discharge and (vertical) flow velocities, but subsequently most debris remains at the surface again.

Figure 5.2, corresponding to $F_0 = 0.25$, shows the development of a more typical accumulation. The flow is sufficiently strong to pull branches down towards the grate, causing some backwater rise. Gradual addition of more debris then causes the accumulation to take on an arched geometry.

Figure 5.3, corresponding to $F_0 = 0.35$, shows an extreme example under higher Froude numbers, where the accumulation grows into a triangular shape. However, most repetitions with $F_0 = 0.35$ developed an arched geometry that was visually more similar to the accumulation in Figure 5.2.

The effects described here are explained from a theoretical perspective in Section 5.2.

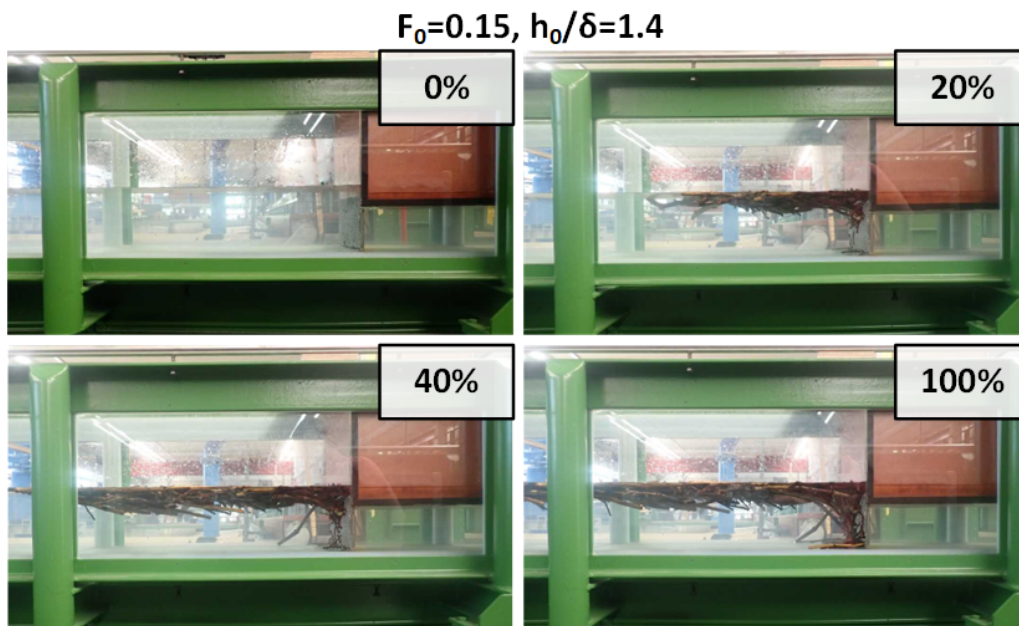


Figure 5.1: Development of the accumulation for $F_0 = 0.15$ and $h_0/\delta = 1.4$. The legend indicates the fraction of the fixed experimental debris volume V_d added.

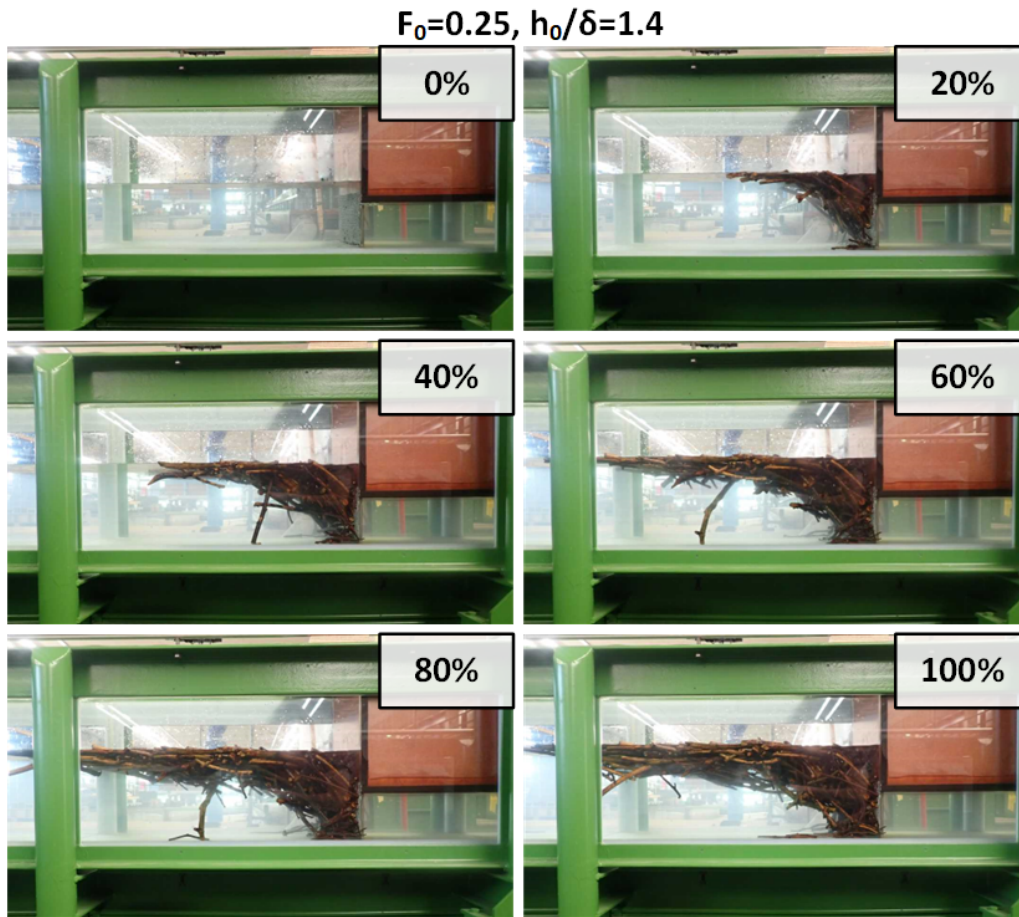


Figure 5.2: Development of the accumulation for $F_0 = 0.25$ and $h_0/\delta = 1.4$. The legend indicates the fraction of the fixed experimental debris volume V_d added.

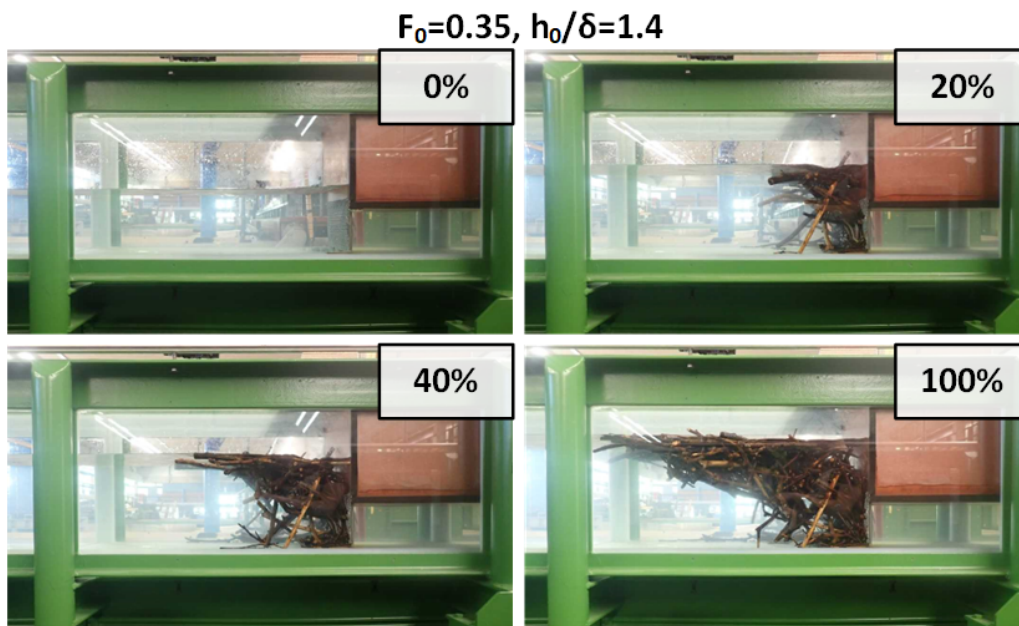


Figure 5.3: Development of the accumulation for $F_0 = 0.35$ and $h_0/\delta = 1.4$. The legend indicates the fraction of the fixed experimental debris volume V_d added.

5.2. Drag and lifting forces

When a floating log comes from upstream and approaches the culvert two things can happen, either it will remain at the surface or it will be pulled down towards the grate at the inlet. This is determined by the ratio between drag and lifting forces. Drag force is determined by the velocity, in this case the vertical component specifically, and lifting force comes from buoyancy. Assuming the vertical velocity to be downward, a submerged log will sink if the following expression holds true.

$$\frac{1}{2} c_D \rho_d u_y^2 L_l d_l > L_l \frac{\pi}{4} d_l^2 (\rho_w - \rho_d) g \quad (5.1)$$

$$u_y^2 > \frac{1}{2} \pi d_l \frac{\rho_w - \rho_d}{\rho_d} \frac{1}{c_D} g \quad (5.2)$$

The expression shows that the required vertical velocity for a cylindrical log to sink depends on its diameter d_l , the debris density ρ_d , the density of water $\rho_w = 1000 \text{ kg/m}^3$, a drag coefficient c_D and the gravitational acceleration $g = 9.81 \text{ m/s}^2$. c_D is about 1.0 for cylindrical logs (Follet et al., 2020), regardless of the orientation (Hygelund & Manga, 2003)

The required vertical velocities for each log size to sink are summarised in Table 5.1. It should be stressed that when mean velocities are below these values, turbulent fluctuations could still exceed these threshold velocities and enable downward transport. Furthermore, note that $d_l = L_l/20$.

L_l [m]	0.05	0.10	0.15	0.20	0.25	0.30
$v_{y,req}$ [m/s]	0.098	0.139	0.170	0.196	0.219	0.240

Table 5.1: Required vertical velocity for different logs to sink. Note that the log diameter $d_l = L_l/20$

Figure 5.4 shows how these required vertical velocities (dashed lines) compare to the measured mean horizontal and vertical velocity components, as well as the 2.5th percentile and 97.5th percentile of velocity fluctuations. All three profiles correspond to the location of 12 cm upstream of the inlet. This location was selected because the largest downward velocities were observed here.

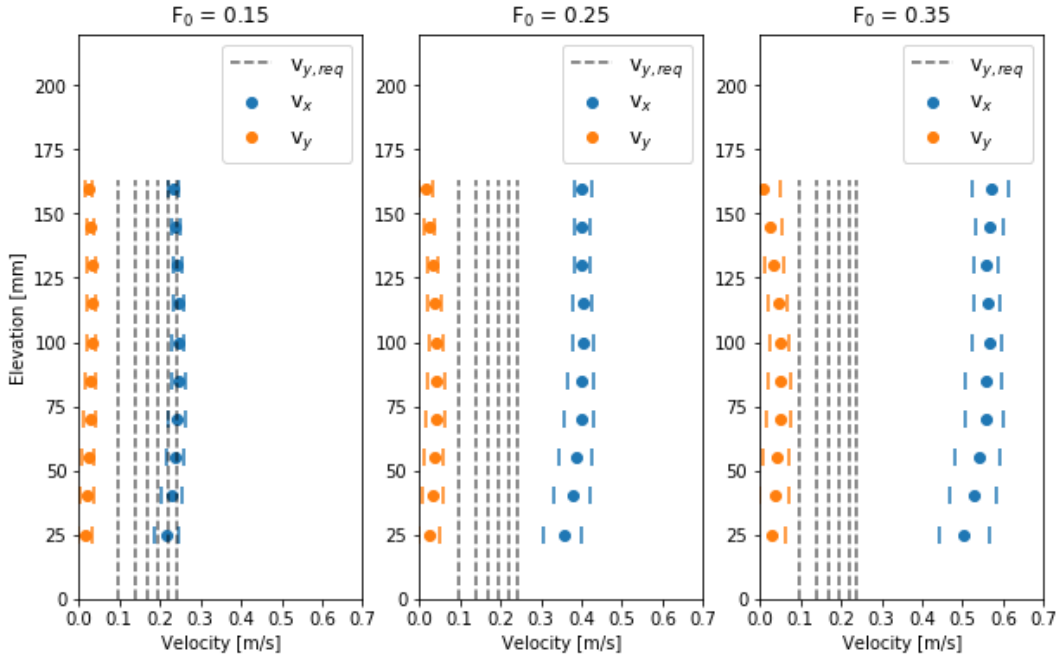


Figure 5.4: Horizontal and vertical velocity components compared to required vertical velocity for log to sink. Profiles are taken at 12 cm upstream of the inlet, without debris. Dots refer to mean values. Vertical line markers give the 2.5th percentile and 97.5th percentile of velocity fluctuations. In this figure positive vertical velocities are oriented downward.

From Figure 5.4 it becomes clear that for all three Froude numbers the downward velocities are not large enough to overcome the buoyancy. This means that except for the few logs with a density much higher than 800 kg/m^3 a different process must pull the logs down towards the inlet. A possible explanation is that as logs approach and collide with the accumulation one of three mechanisms can take place:

1. The log is blocked and becomes part of the floating accumulation.
2. Upon impact with the accumulation the log is deflected downward, transferring part of the log's horizontal kinetic energy to the vertical direction. The vertical kinetic energy then becomes sufficient to overcome the buoyancy effect.
3. Upon impact with the floating accumulation one end of the log sticks into the accumulation, creating a hinging point. If the log has an inclination, a horizontal drag force will make the log rotate around the hinging point. This will further increase the horizontal drag force which can transport the log further towards the inlet.

The above effects are visualised in Figure 5.5.

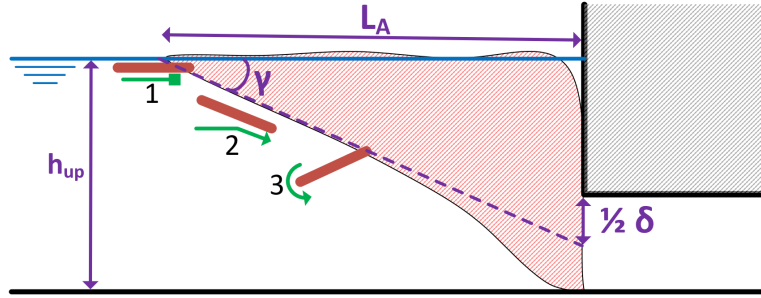


Figure 5.5: Transport mechanisms and parameters in the deflection model

Going back to the accumulations shown in Section 5.1, one can observe that for the case of $F_0 = 0.15$ the flow generally is too weak to pull branches down towards the grate. Figure 5.4 shows that in this case the horizontal velocities are slightly larger than the required vertical velocities, but still nearly all logs remain at the surface. This proves that v_x is not fully converted to a downward momentum. For $F_0 = 0.25$ and $F_0 = 0.35$ the horizontal velocities are at least 2 and 3 times as large as the required vertical velocities, and in these cases logs are transported towards the grate in larger numbers.

5.3. Accumulation length, angle and approach flow velocity

The expression between drag and lifting forces from Eq. 5.2 may be rewritten to Eq. 5.3.

$$\frac{u_y^2}{g d_l} > \frac{1}{2} \pi \frac{\rho_w - \rho_d}{\rho_d} \frac{1}{c_D} \quad (5.3)$$

The left hand side of this equation has the form of a Froude number squared, which implies that a Froude number using the log diameter as length scale (rather than the water depth) could be a more suitable mean to describe the development of debris accumulations.

In the Figures below the accumulation length as well as the angle γ (see Figure 5.5) are expressed as a function of the debris volume. In addition, colormaps are applied to show the initial average inflow velocity U_0 corresponding to each data point. To allow for comparison, identical figures are given using a colormap for initial upstream Froude number instead.

In Figure 5.6 a correlation between the accumulation length and U_0 is clearly visible: higher values of U_0 generate shorter debris accumulations. The colour gradient here is more gradual than that in Figure 5.7 using F_0 , supporting the hypothesis that a dimensionless number between velocity and log diameter would be a more suitable means to describe the accumulation process. Figures 5.8 and 5.9 clearly show larger values of γ with higher velocities, and again the gradient for U_0 is more gradual than that for F_0 .

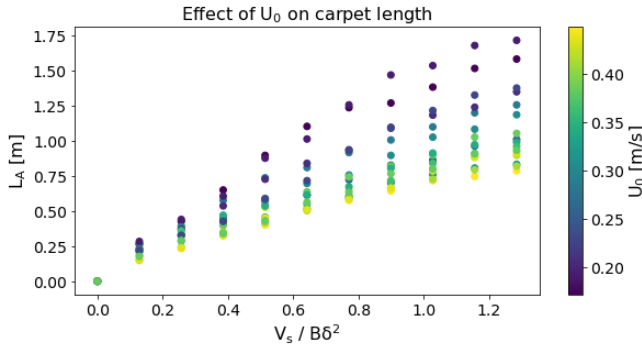


Figure 5.6: Effect of U_0 length. Each dot represents the average of all repetitions of an experiment.

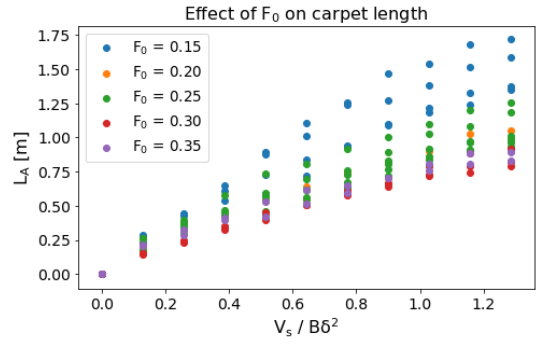


Figure 5.7: Effect of F_0 on carpet length. Each dot represents the average of all repetitions of an experiment.

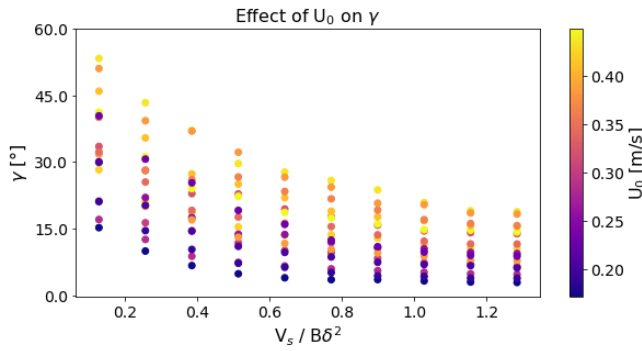


Figure 5.8: Effect of U_0 length. Each dot represents the average of all repetitions of an experiment.

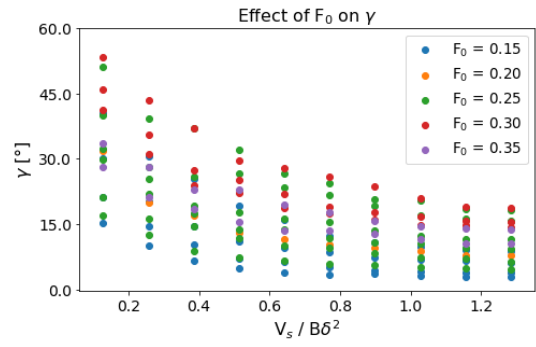


Figure 5.9: Effect of F_0 on carpet length. Each dot represents the average of all repetitions of an experiment.

5.4. Porosity of accumulation

Detailed geometry measurements were taken of the accumulations shown in Section 6.1 to determine the outer volume of the accumulation V_l . The thickness of the accumulation was measured every 5 cm in stream-wise direction, with additional measurements near the inlet. By comparing the outer volume to the solids volume fraction V_s the porosity is then $1 - V_s / V_l$. Another way to express the ratio between outer volume and solids volume is the bulk factor a_b , defined as V_l / V_s .

Table 5.2: Porosity and bulk factor of the three analysed accumulations. The equation by Schalko et al. (2019) is valid for F_0 between 0.3 and 1.5; $F_0 = 0.15$ and $F_0 = 0.25$ are outside of this range and deviate from the measured values.

F_0 [-]	0.15	0.25	0.35
porosity [-]	75.9%	77.4%	77.5%
bulk factor a_b [-] (measured)	4.15	4.43	4.45
bulk factor a_b [-] (Schalko et al., 2019)	4.80	4.66	4.53

Table 5.2 shows that the accumulation porosity hardly changes with Froude number in these ranges that are defined by an outlet-controlled flow. Schalko et al. (2019) found that the bulk factor for accumulations at pile structures can generally be approximated by $5 - 1.35 F_0$ for F_0 between 0.3 and 1.5. In the case of $F_0 = 0.35$ this would amount to $a_b = 4.53$ which is indeed close to the value of 4.45 found during experiments. They also conclude based on a literature review that in real floods a_b is generally between 2 for dense accumulations and 5 for loose accumulations, which is in line with the values in Table 5.2.

5.5. Deflection model

To better understand transport mechanism 2 described in Section 5.2 the following simplified analysis is applied. The deflection model assumes that when a log hits the accumulation and is deflected downward part of the log's momentum in the horizontal direction is transferred to the vertical direction.

First the angle γ is determined from all data. It is approximated as $\gamma = \arctan((h_{up} - \frac{1}{2}\delta)/L_A)$, with L_A being the accumulation length. For a visualisation of symbols, see Figure 5.5. Next, the assumption is made that logs will be deflected parallel to this same angle γ , and will have a downward velocity of $v_{y,deflected} = v_x \cdot \sin(\gamma)$ in addition to the vertical current velocity.

Figure 5.10 presents the velocities obtained from the aforementioned deflection model and compares them to the experimental data. The coloured dotted lines give the vertical current velocity, which is based on the measured vertical velocities at the most upstream location below the debris carpet. Since these measurements are only available for 0% and 100% of debris, interpolation between these points was performed assuming a linear relation with the corresponding backwater rise at that volume. The solid lines are the sum of vertical current velocity and deflected velocity. The grey dashed lines show the required vertical velocities for downward transport per log size, and are identical to the dashed lines in Figure 5.4.

As expected, in case of $F_0 = 0.15$ the deflected velocities are insufficient for the downward transport of all debris sizes. For $F_0 = 0.25$ the deflected velocity is large enough to transport the smallest three debris size classes. In case of $F_0 = 0.35$ vertical transport is possible for all size classes initially, but as the accumulation grows and upstream water level rises, the vertical velocities decrease only the smallest three size classes can be transported downward through mechanism 2.

It should be noted that this analysis is based on several assumptions, and should therefore only be seen as a qualitative assessment. In practice not all logs will be deflected at an angle equal to γ . Furthermore, due to a lack of data it was assumed that v_x is equal to the average upstream velocity, but in practice v_x is higher near the surface. The model assumes this effect to be balanced out by a loss of kinetic energy upon impact.

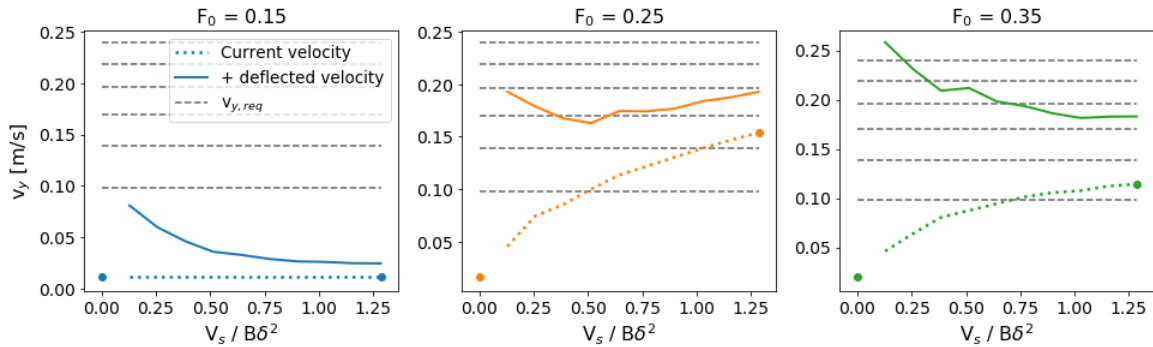


Figure 5.10: Downward velocities according to deflection model. Grey dashed lines give the required downward velocity to enable downward transport of each debris size class. Coloured dotted lines give the measured or interpolated downward flow velocity. Solid coloured lines give the sum of measured or interpolated downward velocity and deflected velocity.

5.6. Summary

In this chapter the development of floating debris accumulations was studied and analysed. Based on a balance equation between drag forcing and buoyancy it was found that for conditions with $F_0 = 0.15$, $F_0 = 0.25$ and $F_0 = 0.35$ the measured downward velocities were insufficient to drag floating logs down towards the inlet. During the experiments it was observed that when $F_0 = 0.15$ only a minor amount of logs was dragged down towards the inlet, but when $F_0 = 0.25$ and $F_0 = 0.35$ large numbers of logs were found at the inlet. This means that a process other than the downward flow velocity brings the logs towards the inlet.

Conform visual observations it was suggested that as a floating log collides with an existing debris accumulation one of three mechanisms may happen: (1) the log is blocked and becomes part of the accumulation, (2) upon impact with the accumulation the log is deflected downward, (3) upon impact one end of the log sticks into the accumulation, creating a hinging point, and the log rotates around this point to move downward and further downstream. The analysis in Section 5.5 proved the feasibility of mechanism 2.

6

Flow and drag analysis

6.1. Debris accumulations and locations of measurements

Figures 6.1, 6.2 and 6.3 show the debris accumulations that were used to obtain velocity data. They correspond to experiments 3A, 3C and 3E respectively. The locations of the velocity profile measurements were given already in the experiment plan in Table 3.2, but here the red lines indicate visually how these locations relate to the accumulation geometry.

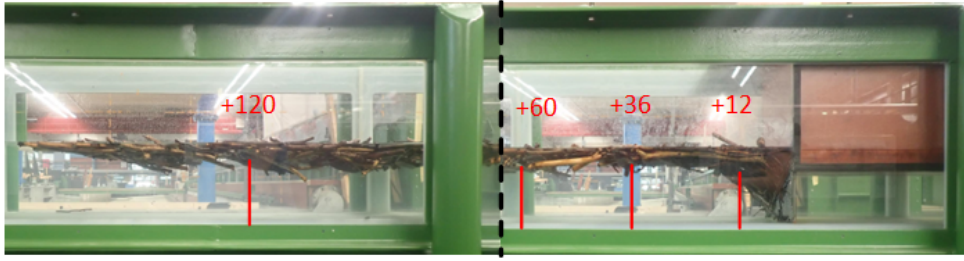


Figure 6.1: Velocity profile measurement locations for conditions of $F_0 = 0.15$ and $h_0/\delta = 1.4$. Values indicate the distance in cm upstream of the inlet.



Figure 6.2: Velocity profile measurement locations for conditions of $F_0 = 0.25$ and $h_0/\delta = 1.4$. Values indicate the distance in cm upstream of the inlet.

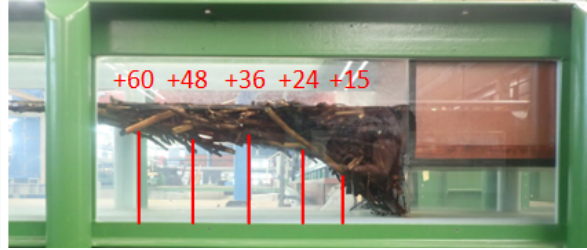


Figure 6.3: Velocity profile measurement locations for conditions of $F_0 = 0.35$ and $h_0/\delta = 1.4$. Values indicate the distance in cm upstream of the inlet.

6.2. Velocity and stress profiles

At the given locations velocity profiles were measured using Laser Doppler Anemometry (LDA). The device was able to measure the horizontal (streamwise) and vertical velocity components, denoted by v_x and v_y respectively. At every point data was sampled for three minutes at a frequency of 1000 Hz. Then Reynolds decomposition was applied to separate the fluctuations from the mean flow values.

$$v_x = \overline{v_x} + v'_x \quad v_y = \overline{v_y} + v'_y$$

Next, Reynolds stresses are obtained from these turbulent fluctuations. Using v'_x and v'_y the normal stresses q_{xx} and q_{yy} as well as the shear stress $s_{xy} = s_{yx}$ can be determined.

$$q_{xx} = \rho \overline{v'_x v'_x} \quad q_{yy} = \rho \overline{v'_y v'_y} \quad s_{xy} = s_{yx} = \rho \overline{v'_x v'_y}$$

Figure 6.4 shows the velocity and stress profiles for the case of $F_0 = 0.35$. Locations correspond to Figure 6.3, and are expressed in cm upstream of the inlet. The subscript 'd' refers to conditions after adding debris, and the subscript '0' to conditions without debris. In case of v_y , negative velocities mean they are downward. In the velocity profile graphs, the continuous lines show the 99% confidence interval of the fluctuations.

Similar figures for $F_0 = 0.15$ and $F_0 = 0.25$ are included in Appendix B. In Appendix C the Prandtl mixing length model is applied to determine the proximity of each highest data point to the debris accumulation.

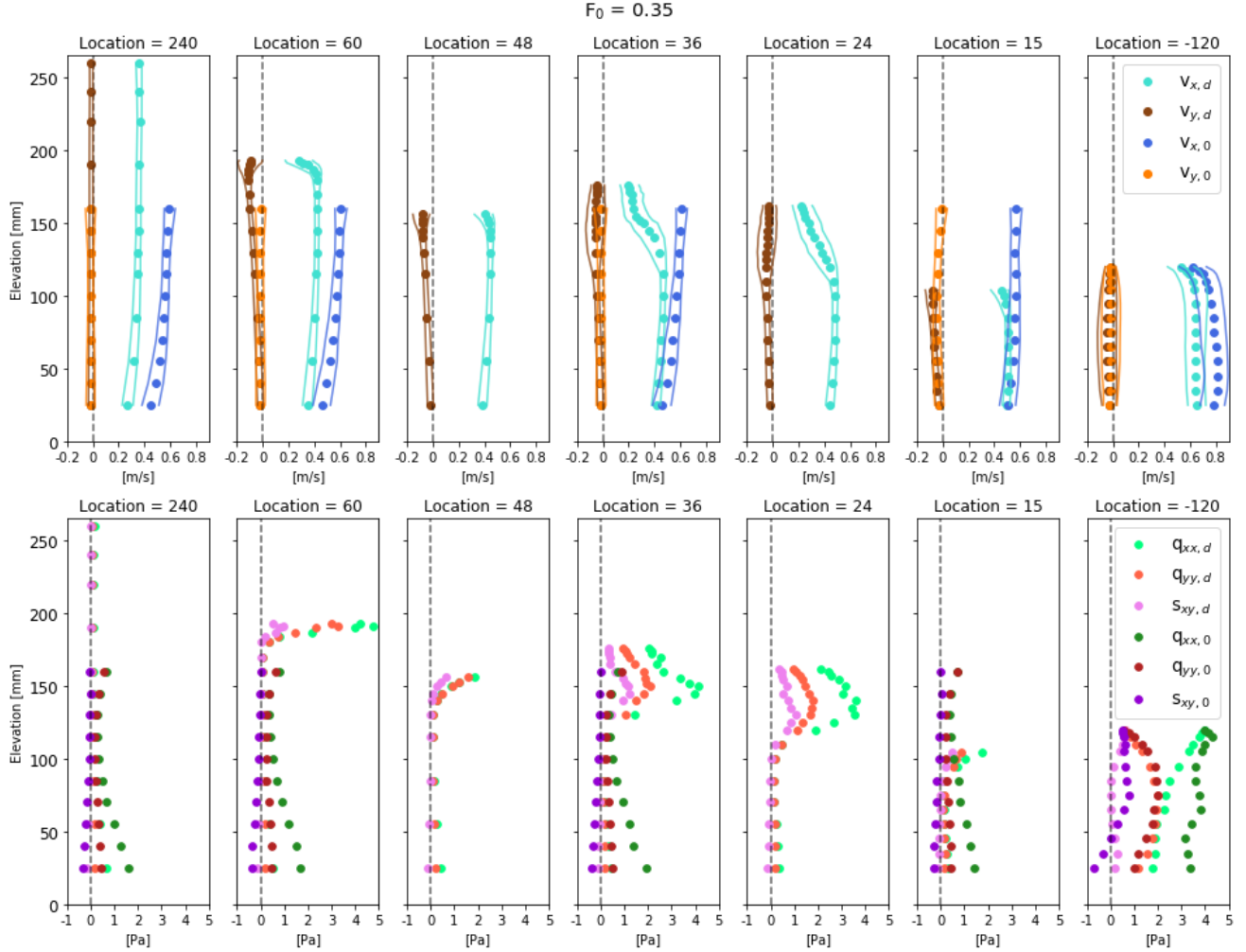


Figure 6.4: Velocity and stress profiles for conditions of $F_0 = 0.35$ and $h_0/\delta = 1.4$. Subscript 'd' refers to debris conditions, subscript '0' to initial conditions. Solid lines show the 99% confidence interval of velocity fluctuations.

The upstream velocities increased after debris was introduced, because the resulting backwater rise means that the same discharge is conveyed with a larger water depth. A sharp increase in Reynolds stresses can be observed just below the debris accumulation, especially at the location 60 cm upstream of the inlet. This can be explained by the rough and irregular surface at the bottom of the accumulation. The figures show that elevation above which Reynolds stresses sharply increase corresponds with the elevation of maximum horizontal velocity. At the locations of 24 and 36 cm upstream of the inlet this sharp increase is visible too, but going further up into the water column the stresses decrease again and the peak in Reynolds stresses is actually a couple of centimeters below the accumulation.

At the lowest data points the Reynolds stresses decreased compared to the situation without debris. This effect was also observed when $F_0 = 0.25$, but in case of $F_0 = 0.15$ the disturbance in the lower half of the water column compared to the situation without debris was minimal.

6.3. Flow through the accumulation

The following steps were taken to estimate the discharge through the debris accumulation.

1] The thickness of the debris accumulation d below the water surface is computed at every location as the difference between the upstream water level h_{up} and the elevation of the highest data point y_{max} . Where no debris is present, y_{max} is assumed to be equal to the upstream water depth.

$$d = h_{up} - y_{max} \quad (6.1)$$

2] Calculate the measured discharge below the accumulation Q_m by numerical integration, from the bed to the highest data point.

$$Q_m = B \cdot \int_0^{y_{max}} v_x(y) dy \quad (6.2)$$

Here v_x is approximated by the log law of the wall below the lowest data points (which are at 25 mm from the bed), using the data points below the maximum horizontal velocity. This is shown in the figures below. For all profiles u_* is around 0.015 m/s, and the roughness k_s has a maximum of 0.0003 m.

$$U = \frac{u_*}{\kappa} \cdot \left(\ln(y) - \ln\left(\frac{k_s}{30}\right) \right) \quad (6.3)$$

3] The discharge Q_m has to be corrected for the fact that flow velocities in the center of the flume are higher than near the walls. At the upstream location, where no debris is present, Q_m corresponds to the entire water column. This can be compared to the measured discharge Q_t to obtain a correction factor ξ , which can then be used to obtain the corrected discharge Q_c .

$$\xi = Q_{m,upstream} / Q_t \quad (6.4)$$

$$Q_c = Q_{m,debris} / \xi \quad (6.5)$$

Table 6.1 gives the values for ξ in this analysis. The measured discharge in the middle of the flume was at most 12% higher than the width-averaged discharge.

Table 6.1: Correction factor values

F_0 [-]	0.15	0.25	0.35
ξ [-]	1.115	1.096	1.121

4] The discharge through the accumulation Q_a is the difference between the total measured discharge and the corrected discharge below the accumulation that follows from the data.

$$Q_a = Q_t - Q_c \quad (6.6)$$

5) The discharge through the accumulation Q_a has to pass through a cross-section of Bd . This gives an averaged velocity.

$$v_{x,a,avg} = Q_a / (B \cdot d) \quad (6.7)$$

The actual flow velocity through the pores of the accumulation, is the average velocity from the equation above divided by the porosity, values of which were found to be around 77% in Section 5.4.

$$v_{x,a,pores} = v_{x,a,avg} / \text{porosity} \quad (6.8)$$

Figures 6.5, 6.6 and 6.7 show the results for experiments 3A, 3C and 3E respectively. The vertical extent of the dashed lines indicates the local thickness of the debris accumulation. The black lines show the averaged velocity through the accumulation, and the blue lines the actual velocity through the pores.

In line with expectations, due to increased friction the flow velocities through the pores are generally lower than the velocities just below the debris. One profile strongly deviates from this: the profile 26 cm upstream in experiment 3C. The accumulation was highly non-uniform across the width just upstream of this cross-section, meaning that the velocities measured in the center of the flume were likely not representative for this cross-section.

The locations 24 and 36 cm upstream in experiment 3E also deviate somewhat from the expected pattern. However, the mixing analysis in Appendix C shows that at these locations the highest data points might not be at the debris boundary. Contrary to this, the profiles at 48 and 60 cm upstream give lower velocities through the accumulation, and there the mixing length analysis also show more clearly that the highest data points are at the debris boundary.

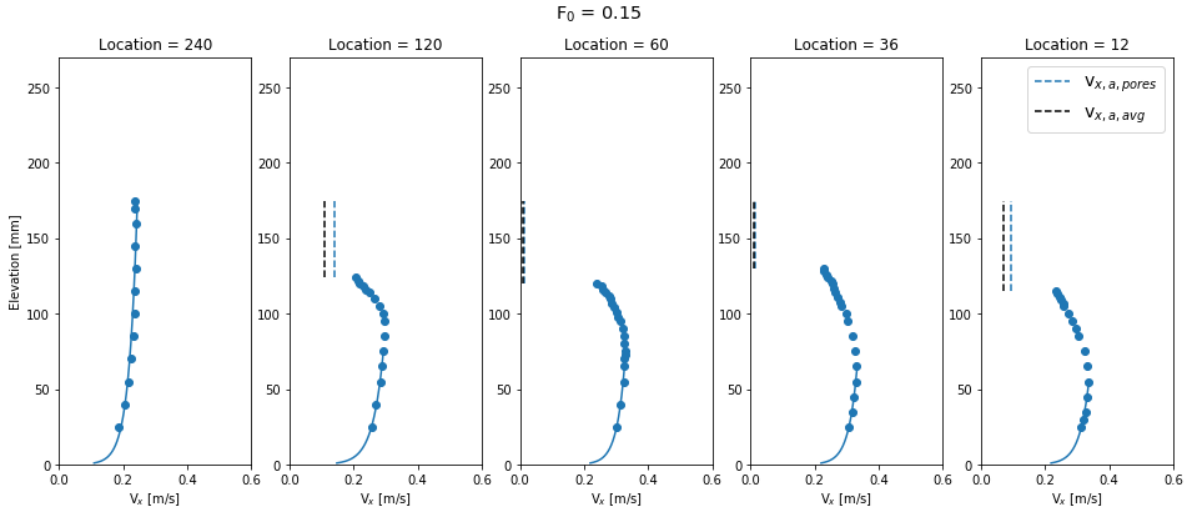


Figure 6.5: Flow velocities through accumulation for conditions of $F_0 = 0.15$ and $h_0/\delta = 1.4$ (experiment 3A).

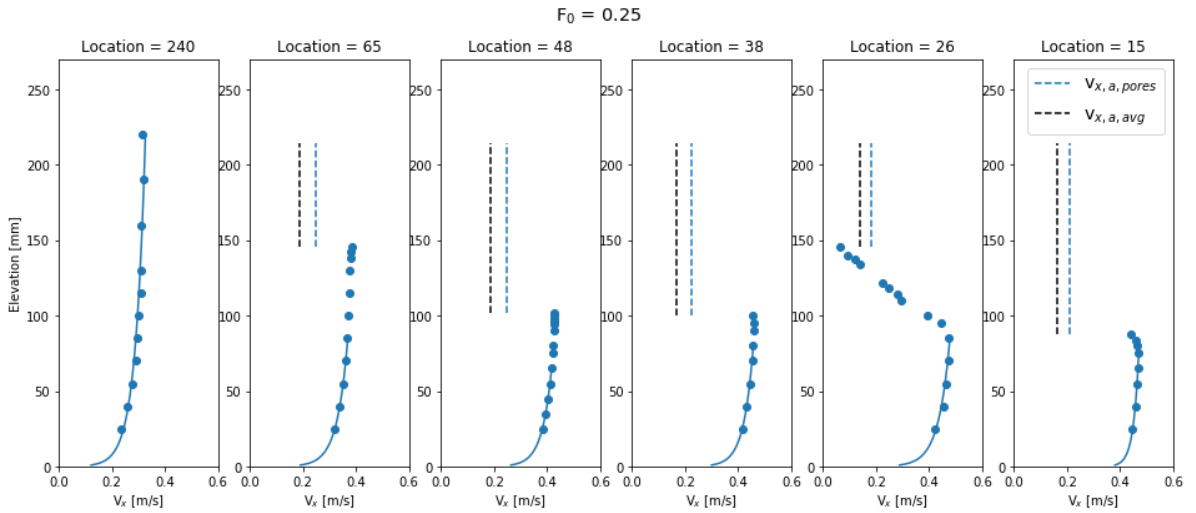


Figure 6.6: Flow velocities through accumulation for conditions of $F_0 = 0.25$ and $h_0/\delta = 1.4$ (experiment 3C).

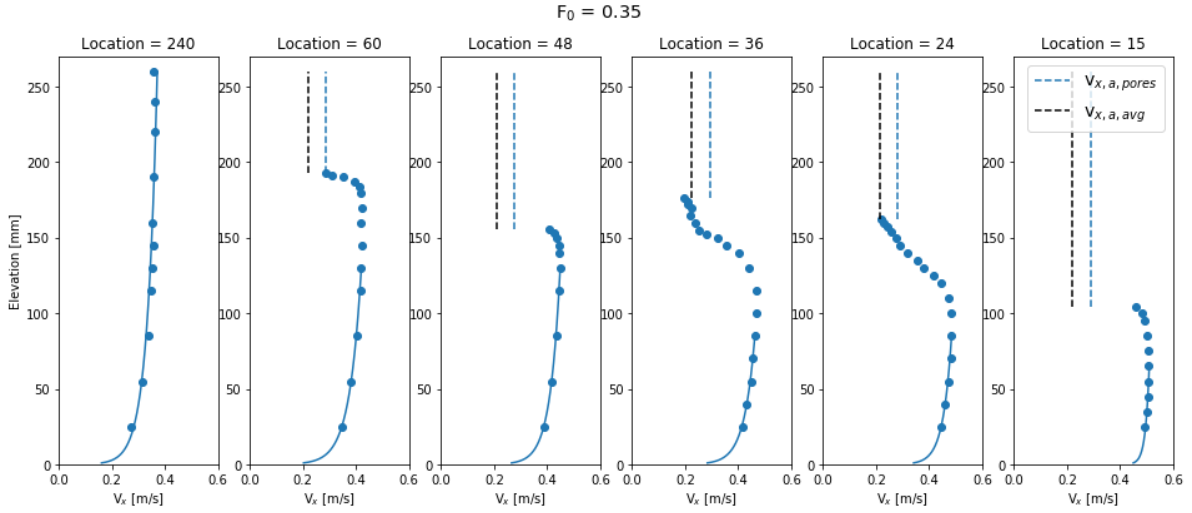


Figure 6.7: Flow velocities through accumulation for conditions of $F_0 = 0.35$ and $h_0/\delta = 1.4$ (experiment 3E).

6.4. Influence of canopy drag and inlet blockage

Introduction

Follet et al. (2020) found an equation that could accurately describe the momentum balance for flow through a debris jam between two rows of pile structures. Their equation consisted of three terms: the net hydrostatic pressure force, the net change in momentum and the drag within the jam. A canopy drag model was selected to describe the drag within the jam. During their work they found that the net change in momentum was on average 13% of the net hydrostatic pressure force, and thus the net change in momentum was found to be negligible. Their final expression was given in Eq. 2.4.

For the experimental setup used in this research a similar equation between the hydrostatic pressure force and drag forces is derived in the following. The control volume is from the upstream boundary to immediately downstream of the grate at the culvert inlet, as indicated by the purple dashed lines in Figure 6.10.

Theory - no debris

When considering a submerged culvert that is clear of any debris, the relevant forces within the control volume are the net hydrostatic pressure force, an equivalent force accounting for inlet loss, and a form drag force resulting from the blockage of the inlet caused by the grate. To obtain the momentum equation, a head loss equation in the form of Eq. 6.9 is used as a reference.

$$\Delta H = K_{gd} \frac{U_{cul}^2}{2g} + K_{in} \frac{U_{cul}^2}{2g} \quad (6.9)$$

Multiplying all terms by $\rho_w g B \delta$ gives a force balance in the form of Eq. 6.10.

$$\rho_w g B \delta \Delta H = \frac{1}{2} \rho_w B \delta K_{gd} U_{cul}^2 + \frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2 \quad (6.10)$$

The expression $\Delta H = \Delta h + \Delta \frac{U^2}{2g}$ is applied to the left hand side of the equation to separate the force terms related to pressure head and velocity head, the latter of which was described as the net change in momentum by Follett et al. (2020). While they determined this term to be negligible, it remains included in this analysis for better accuracy.

$$\underbrace{\rho_w g B \delta \Delta h}_{\text{Net hydrostatic force}} + \underbrace{\frac{1}{2} \rho_w B \delta (U_{up}^2 - U_{cul}^2)}_{\text{Net change in momentum}} = \frac{1}{2} \rho_w B \delta K_{gd} U_{cul}^2 + \frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2 \quad (6.11)$$

The pressure head difference Δh can be specified further as $h_{up} - h_{cul}$, giving Eq. 6.12. The pressure head inside the culvert h_{cul} could not be measured directly, so it is derived following 6.13 from the downstream energy head and expressions for outlet and friction loss introduced in Chapter 4.

One further correction that needs to be made to the forcing term is that the net hydrostatic pressure acting upon the grate bars does not drive the flow, therefore a multiplication by $(1-p)$ to this term is necessary.

$$\rho_w g B \delta (h_{up} - h_{cul}) (1-p) + \frac{1}{2} \rho_w B \delta (U_{up}^2 - U_{cul}^2) = \frac{1}{2} \rho_w B \delta K_{gd} U_{cul}^2 + \frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2 \quad (6.12)$$

$$h_{cul} = H_{down} + \Delta H_{out} + \Delta H_{fric} - \frac{U_{cul}^2}{2g} \quad (6.13)$$

Figure 6.8 further illustrates the derivation of the net hydrostatic pressure force. Only the pressure from the bed to elevation δ (opening height) drives the flow.

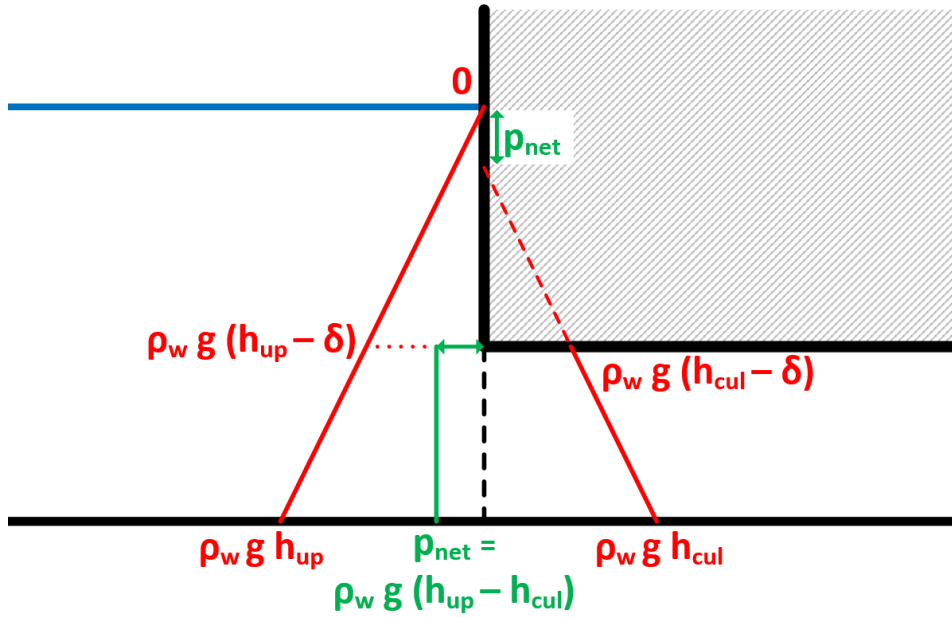


Figure 6.8: Hydrostatic pressure distribution on both sides of the culvert inlet (red), and net hydrostatic pressure in front of the inlet (green). The pressure head inside the culvert h_{cul} is defined by Eq. 6.13.

Next, the resistance term related to blockage of the inlet is considered in more detail. So far it has been described using an undefined loss coefficient K_{gd} . Inlet blockage is modelled here as a form drag, which is typically given by Eq. 6.14. In this equation F_D is the drag force, ρ_w the density of water, c_D a drag coefficient, A the projected area perpendicular to direction of motion and U the flow velocity relative to the object.

$$F_D = \frac{1}{2} \rho_w c_D A U^2 \quad (6.14)$$

The drag coefficient $c_{D,grate}$ is described by Eq. 6.15 as found and verified in Yu et al. (2023) after Milne (1972). It is an empirical relation for drag through rigid nets based on the blockage ratio p .

$$c_{D,grate} = 1 + 1.37p + 0.78p^2 \quad (6.15)$$

The projected area of the grate A is the cross-section of the inlet $B\delta$ multiplied by the blockage ratio p . Because U is the flow velocity perpendicular to the surface area of the bars it can be approximated by U_{cul} . The result is Eq. 6.16.

$$F_D = \frac{1}{2} \rho_w B \delta p c_D U_{cul}^2 \quad (6.16)$$

Combining all these findings gives Eq. 6.17 for the situation of a fully submerged culvert without debris.

$$\underbrace{\rho_w g B \delta (h_{up} - h_{cul}) (1 - p)}_{\text{Net hydrostatic force}} + \underbrace{\frac{1}{2} \rho_w B \delta (U_{up}^2 - U_{cul}^2)}_{\text{Net change in momentum}} = \underbrace{\frac{1}{2} \rho_w B \delta p c_{D,grate} U_{cul}^2}_{\text{Blockage form drag}} + \underbrace{\frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2}_{\text{Inlet loss}} \quad (6.17)$$

Lastly, one adjustment is made to the equation. The correction $(1 - p)$ is applied to the net hydrostatic pressure force, but this causes that the equation cannot simply be converted back to an equation for head loss (similar to Eq. 6.9) through division by $\rho_w g B \delta$. For this reason the correction $(1 - p)$ is applied to the blockage form drag term instead, but as a division because it applies to a resistance term rather than a forcing term. It was found that the adjusted Eq. 6.18 described the experimental data well, whereas Eq. 6.17 systematically underestimated the data for p by up to 25%.

$$\rho_w g B \delta (h_{up} - h_{cul}) + \frac{1}{2} \rho_w B \delta (U_{up}^2 - U_{cul}^2) = \frac{1}{2} \rho_w B \delta \frac{p}{1 - p} c_{D,grate} U_{cul}^2 + \frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2 \quad (6.18)$$

Reserving the steps that were taken to arrive at the momentum balance equation returns it to a head loss equation. Dividing Eq. 6.18 by $\rho_w g B \delta$, and merging the net hydrostatic pressure force and the net change in momentum into $(\Delta H = \Delta h + \Delta \frac{U^2}{2g})$ returns Eq. 6.19. The loss factor for grate and debris blockage K_{gd} is explicitly defined. In addition, K_{gd} is of similar shape to Eq. 4.6 proposed by Meusburger (2002).

$$\Delta H = \underbrace{c_{D,grate} \frac{p}{1 - p}}_{K_{gd}} \frac{U_{cul}^2}{2g} + K_{in} \frac{U_{cul}^2}{2g} \quad (6.19)$$

Validation - no debris

Before extending Eq. 6.18 to account for the presence of debris the equation is compared to the experimental data without debris. There are two sets of data that can be used for this, namely for the situation with grate (p between 0.36 and 0.411, depending on whether the grate and support bars are tightly pressed together), and for the situation with only the support bars ($p = 0.08$). Using the measurements for flume discharge and upstream and downstream water level the inlet blockage was computed from Eq. 6.18. There is a strong match for the measurements with grate. The predicted values without grate have an average blockage ratio of 0.056, which is a slight underestimation compared to the measured value of 0.08. In addition, there is a larger spread in the values without grate which can be attributed to measurement inaccuracies. Because head losses without grate were in the order of millimeters to centimeters, measurement inaccuracies had a large influence. With grate the head losses were in the order of centimeters, and thus measurement inaccuracies only had a minor effect.

In conclusion, the measurements validate Eq. 6.18 for fully submerged culverts without debris, especially when a grate is present.

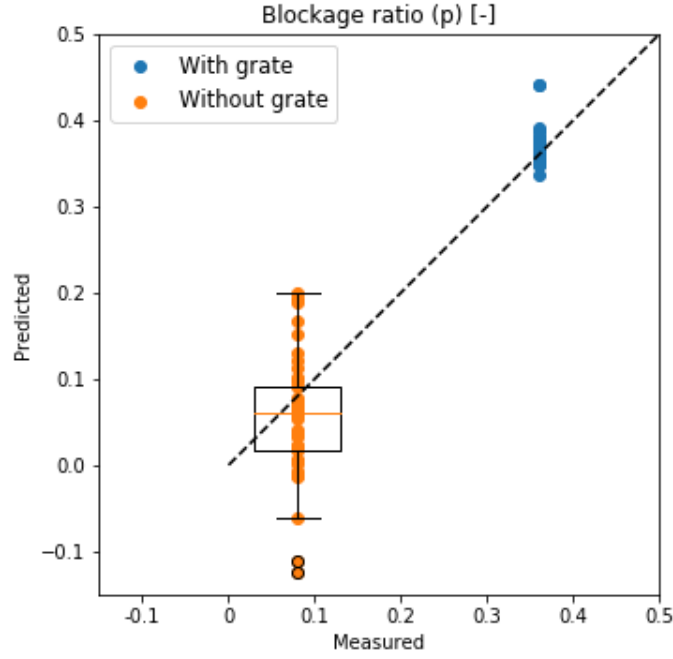


Figure 6.9: Comparison of the measured values for blockage ratio, and those predicted from Eq. 6.18. All data points correspond to a situation without debris. Blue dots ($n=48$) refer to the measurements with the grate ($p = 0.36$, assuming grate and support bars are not tightly pressed together), and orange dots ($n=41$) refer to measurements where the only blockage is generated by the support bars ($p = 0.08$). The box extends from the first quartile (Q1) to the third quartile (Q3) of the data, with a line at the median. The whiskers extend from the box to the farthest data point lying within 1.5x the inter-quartile range (IQR) from the box.

Theory - including debris

When debris is included in the setup two changes to Eq. 6.18 are required. First, the blockage ratio p is now defined as the inlet blockage associated with the grate as well as debris that is attached to it. Second, a canopy drag term must be introduced to account for skin friction drag at the accumulation. Eq 6.20 proposed by Follett et al. (2020) is applied for this term.

$$F_{canopy} = \frac{1}{2} \rho_w B d L a_f c_{D,canopy} U^2 \quad (6.20)$$

The parameters d and L are the thickness and length of an accumulation (section) respectively. U is the flow velocity through the accumulation. Follett et al. (2020) applied $c_{D,canopy} = 1.0$, with good results. Hygelund & Manga (2003) found that the drag coefficient for cylindrical logs is generally independent on orientation, so $c_{D,canopy} = 1$ may be used here as well.

The parameter a_f is the spatially averaged frontal area of debris per jam volume. It can also be explained as the number of 'layers' of frontal area that are on average present within a control volume with cross-section Bd and length of 1 meter. Follett et al. (2020) define a_f as $\phi / (\frac{\pi}{4} \cdot d_l)$ (with ϕ being the solid volume fraction), assuming all logs to be perpendicular to the flow direction and having a diameter d_l . However, in these tests most of the logs were parallel to the flow direction, in which case a_f is equal to $\phi / L_{l,avg}$. In the upcoming analysis a_f is calculated as the average between perpendicular and parallel logs, weighed by volume fraction.

The momentum balance equation for fully submerged culverts including debris accumulation is then given by Eq. 6.21.

$$\underbrace{\rho_w g B \delta (h_{up} - h_{cul}) (1 - p)}_{\text{Net hydrostatic force}} + \underbrace{\frac{1}{2} \rho_w B \delta (U_{up}^2 - U_{cul}^2)}_{\text{Net change in momentum}} = \underbrace{\frac{1}{2} \rho_w B d L a_f c_{D,canopy} U^2}_{\text{Canopy drag}} + \underbrace{\frac{1}{2} \rho_w B \delta p c_{D,grate} U_{cul}^2}_{\text{Blockage form drag}} + \underbrace{\frac{1}{2} \rho_w B \delta K_{in} U_{cul}^2}_{\text{Inlet loss}} \quad (6.21)$$

Application - including debris

The parameters in Eq. 6.21 that still have to be determined are L , d , U and a_f . The accumulation shown in Figure 6.2 corresponding to the Froude number of 0.25 is used as a case study to compare the effect of canopy drag to inlet blockage. Five velocity profiles were measured below this accumulation, at the locations of 15, 26, 38, 48 and 65 cm upstream of the inlet. A schematic is given in Figure 6.10. The dashed red lines show a discretised geometry of the accumulation, with one 'cell' per measured velocity profile.

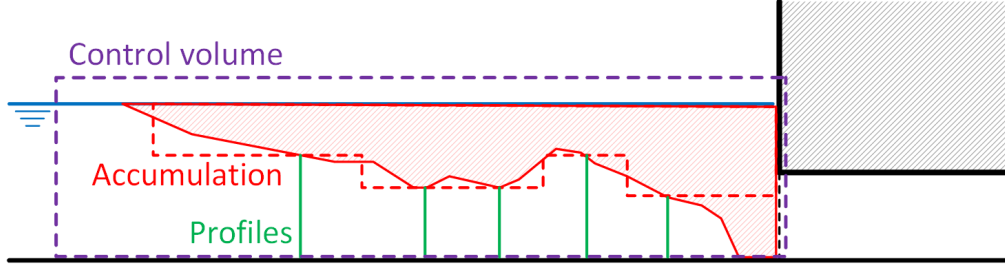


Figure 6.10: Schematisation of the accumulation for conditions of $F_0 = 0.25$ and $h_0/\delta = 1.4$. The control volume is indicated in purple. Green lines show the locations where velocity profile measurements were taken. Solid red lines show the accumulation geometry, and the dashed red lines the discretised accumulation geometry with one 'cell' per measured velocity profile.

In Table 6.2 the length, thickness and flow velocity of each cell are summarised. The flow velocities are the velocities through the pores taken from Section 6.3. At the locations where the highest data point might be below the debris boundary (see Appendix C) the velocity might be slightly overestimated, but since in those cases the thickness will be overestimated as well its effect on canopy drag turns out to be negligible. L is approximated here by the length in the horizontal direction.

Table 6.2: Discretised canopy drag properties. L refers to the length of each cell, d to the thickness and U to the average flow velocity through that cell.

Location	L [m]	d [m]	U [m/s]
+15cm	0.180	0.126	0.211
+26cm	0.115	0.068	0.182
+38cm	0.110	0.114	0.222
+48cm	0.135	0.112	0.248
+65cm	0.295	0.068	0.247

To compute the parameter a_f the representative fraction of logs that are parallel and perpendicular to the flow direction needs to be determined. Looking at the pictures in Section 6.1 most of the larger logs are parallel to the flow or deviate from this only by a small angle, whereas the smallest logs have a more random orientation. A representative value for a_f can be obtained by assuming that all logs in the smallest size class are perpendicular to the flow direction, and that all other size classes are parallel to the flow direction.

Furthermore, the log volume that contributes to the canopy drag term should be reduced by the volume of logs that contributes to the inlet blockage. Because the smallest logs are most likely to be found at the grate (see Section 5.2), this volume is assumed to be a fraction of the smallest size class. Based on observations during the experiments it is concluded that about 20% of the smallest logs represents this volume well.

Following the reasoning above Eq. 6.22 can be formulated. For the case of $F_0 = 0.25$, $a_f = 2.58$ is representative for the accumulation. The solids fraction ϕ follows from Section 5.4. Subscript "1" refers to the smallest log size class, and subscript "2-6" refers to the remaining log size classes as given in Section 3.7. Only 80% of $V_{s,1}$ is included, because the remaining 20% was determined to contribute towards inlet blockage instead.

$$a_f = \underbrace{\frac{\phi}{L_{l,avg,2-6}}}_{\text{parallel}} \cdot \frac{V_{s,2-6}}{V_s} + \underbrace{\frac{4\phi}{\pi d_{l1}}}_{\text{perpendicular}} \cdot \frac{0.8V_{s,1}}{V_s} = 2.58 \quad (6.22)$$

With this Eq. 6.21 can be solved for p . The solution is an equivalent inlet blockage p of 67.3%, compared to 39.5% without debris. Using these parameters, F_{canopy} amounts to 7.4% of the total drag force, and $F_{blockage}$ amounts to 80.5%. The remainder of the resisting force comes from inlet loss.

Applying the same analysis to the accumulations corresponding to the Froude numbers of 0.15 and 0.35 shows that in these cases the canopy drag amounts to 9.5% and 5.9% of the total drag force.

Table 6.3: Influence of canopy drag and inlet blockage as a fraction of the total drag forcing. a_f is the spatially averaged frontal area of debris per jam volume.

F_0 [-]	a_f [m ⁻¹]	Canopy drag [%]	Inlet blockage [%]	Inlet loss [%]
0.15	2.75	9.5	70.4	20.1
0.25	2.58	7.4	80.5	12.0
0.35	2.57	5.9	82.2	11.9

Summary

The analysis shows that the backwater rise due to debris can well be described by neglecting the canopy drag and only considering inlet blockage, especially for engineering purposes. To put this into perspective, if the canopy drag is neglected for the case of $F_0 = 0.25$ the inlet blockage amounts to 68.9% instead of 67.3%.

During experiments it was verified that canopy drag has a much smaller influence than inlet blockage. Removing the debris carpet (but not removing debris from the grate) did indeed cause only a minor reduction in backwater rise. However, it was not possible to obtain measurements that were accurate enough to allow for detailed comparison, as in practice removing the debris carpet always caused some disturbance to the blockage at the grate.

Comparison to literature

This chapter shows that a momentum balance equation, similar to Eq. 2.4 by Follet et al. (2020), can be applied to describe head loss at culverts and inverted siphons.

An alternative for ice covered channels at inverted siphon inlets was proposed in the form of Eq. 2.6 by Fu et al. (2020). However, this equation is based on the condition of a densely packed floating ice layer, which is then modelled as a solid wall. Within the flow regimes considered in this research Section 5.4 showed that the floating debris accumulations were quite porous at 76% to 78%. Section 6.3 proved considerable flow velocities through the accumulations, and verified that the debris accumulations cannot be modelled as a solid wall. As a result, it must be concluded that Eq. 2.6 cannot be applied to the porous debris accumulations observed in the experiments of this research. That being said, it could still have merit for more densely packed debris accumulations, for which dedicated experiments are recommended.

Backwater rise

7.1. Experimental data

Figure 7.1 shows the results of all backwater rise experiments. Each subfigure corresponds to one submergence rate, and the colours refer to different Froude numbers. The fainter symbols visualise individual repetitions, and the squares give the averages of all three repetitions. In this chapter a theoretical framework and design equation are derived to describe the data shown here.

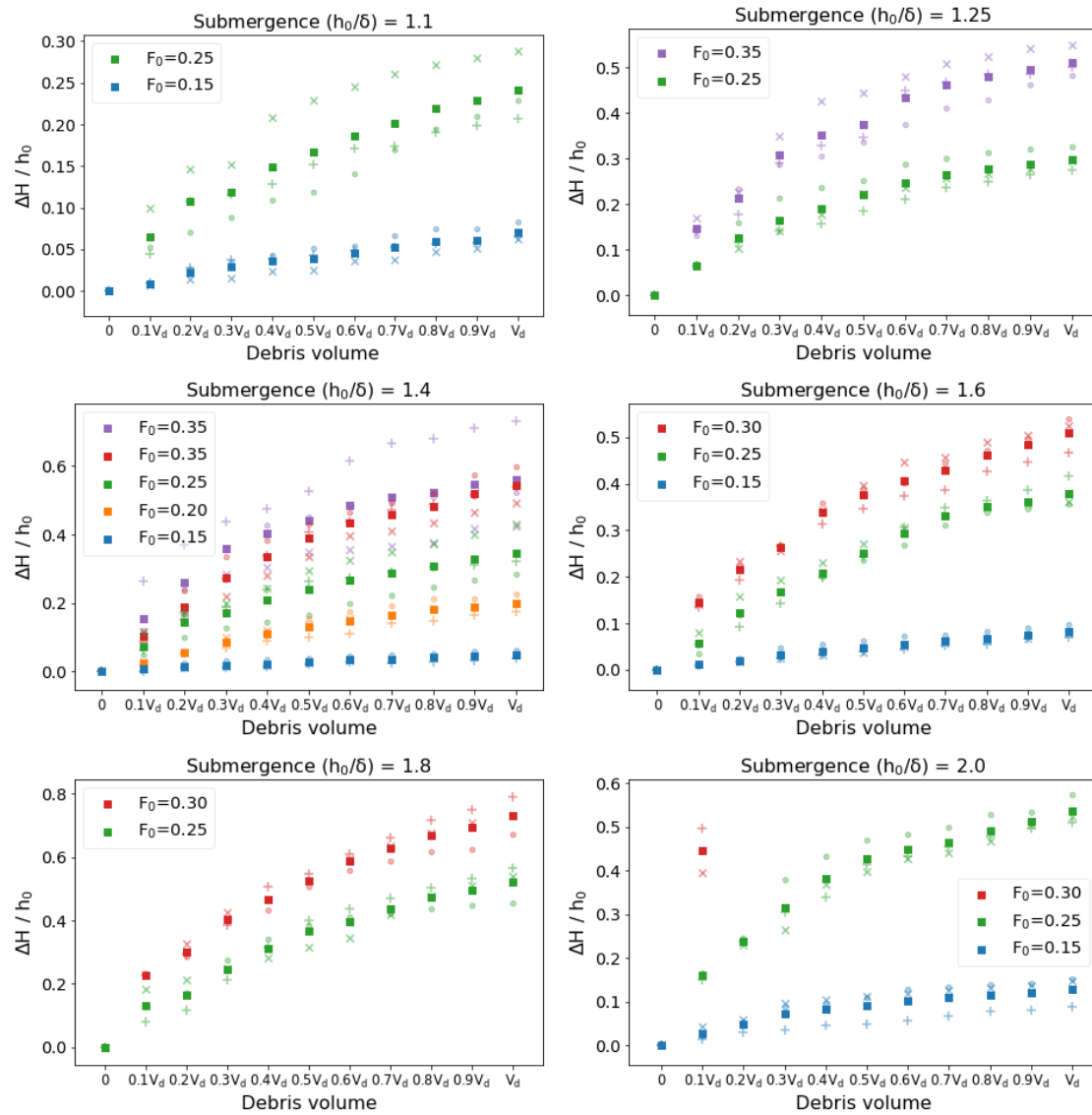


Figure 7.1: Experimental results for backwater rise experiments. Squares give averages of all three repetitions, and fainter symbols visualise individual repetitions.

7.2. Comparison to literature

Schalko et al. (2019) proposed Eq. 2.2 to describe backwater rise associated with floating wooden debris at vertical pile structures. It is repeated here as Eq. 7.1, excluding the fine material term, which is in line with the present work.

$$\frac{\Delta h}{h_0} = 5.4 \cdot f_A \cdot \frac{F_0 (V_s / (h_0 B d_{lm}))^{1/3}}{a_b} \quad (7.1)$$

Δh = backwater rise; h_0 = approach flow depth; f_A = accumulation type factor; F_0 = approach flow Froude number; V_s = debris volume; d_{lm} = mean log diameter; a_b = bulk factor = $\frac{1}{1-\text{porosity}}$.

The measured values for $\Delta h / h_0$ of the present study can be compared to Eq. 7.1. In Figure 7.2 this comparison is made for the developed accumulations corresponding to the total debris volume V_d , meaning $V_s = V_d = 0.0074 \text{ m}^3$. The parameters F_0 and h_0 defined the different experiments, B and d_{lm} were fixed values of 0.40 m and 0.12 m respectively. The bulk factor a_b was determined from the porosity of the accumulation, here approximated by 4.43 following Section 5.4 where differences in porosity were found to be small for developed accumulations within the range of tested Froude numbers. The accumulation type factor f_A is 0.55 for natural accumulation processes with a fixed bed.

Figure 7.2 shows the outcome of this comparison between Eq. 7.1 and the experimental data of the present work. The equation severely underestimates the measured head loss in case of fully submerged culverts, in case of this particular setup approximately by a factor of 3.

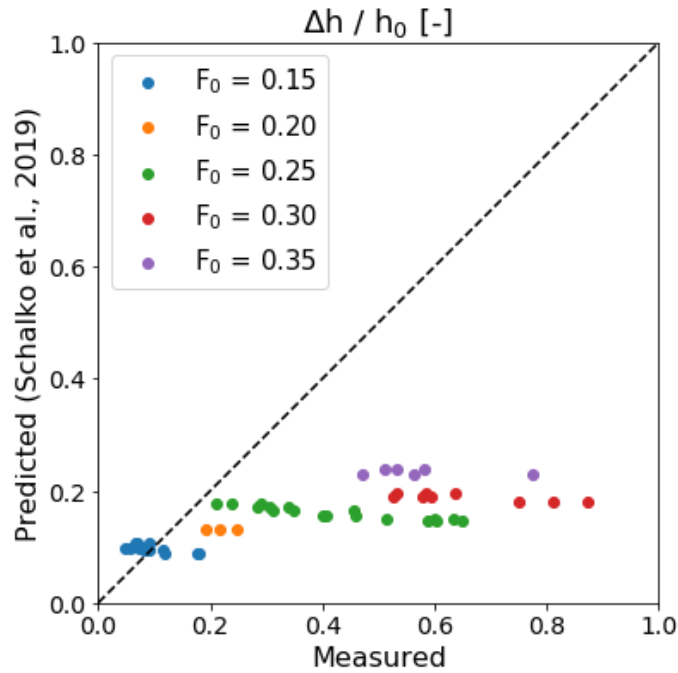


Figure 7.2: Comparison of the measured values for $\Delta h / h_0$ and the predicted values obtained from Eq. 7.1 after Schalko et al. (2019).

In addition, one important remark needs to be made regarding Eq. 7.1. The experiments were performed with a fixed grate in place at the inlet, but the equation is unable to take the presence of such a grate into account. If the experiments were performed with a different grate, or no grate at all, the measured backwater rise could change drastically whereas this would not influence the values predicted by Eq. 7.1.

Furthermore, in Section 6.4 it was found that drag due to inlet blockage is dominant over canopy drag when considering submerged culverts, yet Eq 7.1 for pile structures is mostly based on canopy drag. A possible explanation for this difference is as follows. When the debris volume at a pile structure becomes larger, the cross-sectional blockage at the structure is increased and together with a larger canopy drag this increases

backwater rise. That increase in water depth then reduces the cross-sectional blockage at the structure. At submerged culverts a larger debris volume also increases the cross-sectional blockage of the inlet and thereby backwater rise, but that backwater rise hardly reduces the cross-sectional blockage in turn as the inlet was already fully submerged.

Based on the findings above it was concluded that Eq. 7.1 is not suitable to describe backwater rise at submerged culverts, especially when the inlet is covered by a grate. This points out the need for an additional methodology to predict water level rise at culverts and submerged structures.

7.3. Theoretical framework

Overview of the theoretical framework

In this section a theoretical framework is derived to describe the head loss associated with the presence of debris. In Section 6.4 it was found that drag forcing, and thereby head loss, can well be described by the blockage of the inlet p caused by a combination of debris and a grate. This parameter p is defined as the blocked area fraction of the inlet. The experiments were performed with one fixed grate, so to generalise the results they were separated into separate components describing the effect of grate and debris: p_g and p_d respectively.

The experimental results consisted of values for backwater rise expressed in meter, so first this data was converted to a dimensionless loss factor K_{gd} , which takes into account the head loss associated with debris as well as the grate at the inlet. Next the blockage of the inlet p due to grate and debris combined is calculated according to the methods in Section 6.4. Then p is divided into separate components for the grate blockage p_g and debris blockage p_d .

Following this, a regression analysis is applied in Section 7.4 to find a design equation for p_d .

Figure 7.3 summarises this theoretical framework, and approach to arrive at the design equation. The blue cells show the steps that were taken in the data analysis. The green cells show how design equation can be used to obtain a prediction for the head loss due to grate and debris.

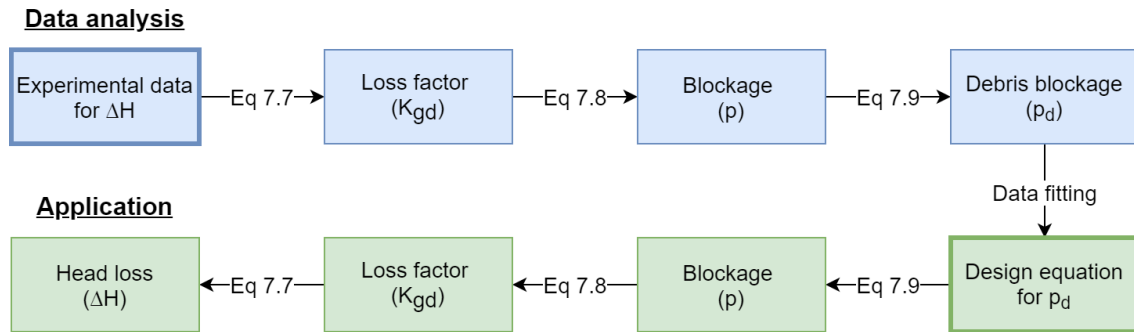


Figure 7.3: Theoretical framework and approach

Reformulating the head loss equation

The head loss over a culvert is described by a loss factor K multiplied by the velocity head, see Eq. 7.2.

$$\Delta H = K \cdot \frac{U_{cul}^2}{2g} \quad (7.2)$$

The upstream velocity u_0 is obtained from the definition of Froude F_0 number in open channel flow, and thus is dependent on F_0 and the upstream water depth h_0 . see Eq. 7.3.

$$F_0 = \frac{U_0}{\sqrt{gh_0}} \Rightarrow U_0 = F_0 \sqrt{gh_0} \quad (7.3)$$

Under the conditions of pressurised flow inside the culvert, as well as equal width between channel and culvert, continuity is described by Eq. 7.4. This means that U_{cul} can be expressed as the product of U_0 and the submergence rate.

$$U_{cul} = U_0 \cdot \left(\frac{h_0}{\delta} \right) \quad (7.4)$$

By taking Eq. 7.4, and then substituting u_0 by Eq. 7.3, the following expression can be obtained:

$$U_{cul} = F_0 \sqrt{g h_0} \cdot \left(\frac{h_0}{\delta} \right) \quad (7.5)$$

Next the expression above is used to substitute U_{cul} in Eq. 7.2, giving the following expression:

$$\Delta H = K \cdot \frac{\left(F_0 \sqrt{g h_0} \cdot \left(\frac{h_0}{\delta} \right) \right)^2}{2g} \quad (7.6)$$

Finally, this can then be rewritten into Eq. 7.7.

$$\frac{\Delta H}{h_0} = \frac{1}{2} \cdot K \cdot F_0^2 \cdot \left(\frac{h_0}{\delta} \right)^2 \quad (7.7)$$

The derived Eq. 7.7 is equal to Eq. 7.2 (under the conditions of pressurised flow, with the same width between channel and culvert), but it shows more explicitly how the initial upstream Froude number, submergence rate and water depth influence the head loss even when K is assumed to be a constant.

Definition of the loss factor

In Section 6.4 it was concluded that head losses associated with a grate and debris can well be described by only considering drag due to inlet blockage and neglecting canopy drag when dealing with fully submerged culverts. The loss factor is then defined by Eq. 7.8 as determined in Section 6.4. Also note that $c_{D,grate}$ was defined in Eq. 6.15 as $c_{D,grate} = 1 + 1.37p + 0.78p^2$, and thus only depends on the blockage p . This means that the loss factor K_{gd} only depends on the blockage p .

$$\Delta H_{gd} = \underbrace{c_{D,grate} \frac{p}{1-p}}_{K_{gd}} \frac{U_{cul}^2}{2g} \quad (7.8)$$

Definition of the blockage ratio

Across all experiments one grate with a fixed blockage ratio was used. However, the results need to be generalised such that they can also be applied to cases where a different grate is used. Therefore the combined blockage p is considered to be made up of two components: the grate blockage p_g and debris blockage p_d . Eq. 7.9 gives the relation between these blockage parameters.

$$p = p_g + p_d \cdot (1 - p_g) \quad (7.9)$$

The grate blockage p_g is the frontal area fraction of the inlet that is blocked by bars. The debris blockage p_d is the frontal area fraction of the inlet that is blocked by debris, neglecting the presence of a grate.

7.4. Design equation for debris blockage

Data converted to debris blockage

In the figures below the experimental data expressed in head loss has been converted to values for debris blockage p_d , following the theoretical framework from Section 7.3. The data fitting curves are derived throughout this section.

Starting with Figure 7.4, the role of the Froude number becomes more clear by considering one specific submergence rate, in this case $h_0/\delta = 1.4$ as it has the most detailed data on Froude number. With the exception of $F_0 = 0.15$ the data points corresponding to various Froude numbers are consistent and no clear trend as a function of Froude number can be found. In Sections 5.2 and 5.5 it was concluded that in case of $F_0 = 0.15$ a certain threshold velocity for downward transport of the smallest logs was not exceeded, hence explaining the different behavior shown in Figure 7.3.

In Figure 7.5 the effect of submergence rate becomes more apparent by considering one specific Froude number, in this case $F_0 = 0.25$ as it has the most detailed data on submergence rate. One can clearly observe that debris blockage decreases with increasing submergence rate. This is in line with expectations, as when submergence rates are higher more debris will float above the inlet rather than directly blocking it.

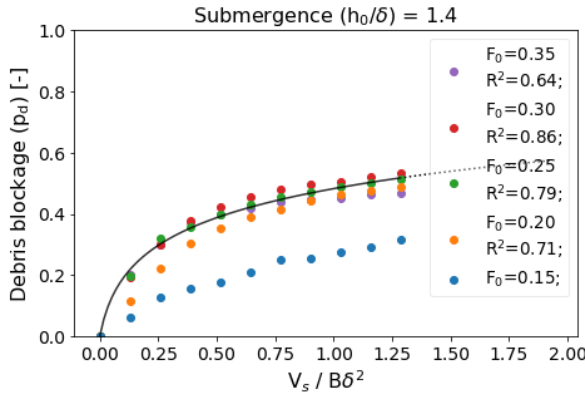


Figure 7.4: Debris blockage results for $h_0/\delta = 1.4$

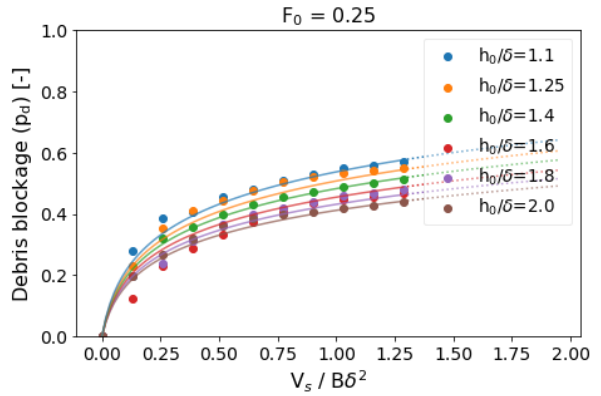


Figure 7.5: Debris blockage results for $F_0 = 0.25$

Effect of submergence rate

The debris blockage p_d is assumed to be a function of the submergence rate to the power β . To determine this power β , all data corresponding to $F_0 = 0.25$ was selected, as this Froude number has the most detailed data for varying submergence rate. Additionally, by using this approach any potential effects related to the Froude number are excluded from the data. Eq. 7.10 is then used as the fitting equation, with k_{sub} being a catch-all term that is broken down further in this section.

$$p_d = k_{sub} \cdot \left(\frac{h_0}{\delta} \right)^\beta \quad (7.10)$$

The data fitting is initially performed for each debris volume individually. The resulting values for the power β are shown in Figure 7.6. As expected from Figure 7.5, β is negative, because larger submergence rates correspond to lower debris blockage. Initially the submergence rate has a stronger effect, but after about 30% of the volume is added an equilibrium is quickly reached around $\beta = -0.45$.

The reason for which β initially has a stronger effect is likely associated with the stage of initial formation of the debris accumulation. Generally two or three batches of debris were required obtain the typical arch-shaped accumulation. Further debris being added then contributes to an expansion of this already developed accumulation, as discussed in Section 5.1.

As mentioned before Figure 7.6 is based on the data corresponding to $F_0 = 0.25$. The remaining tested Froude numbers are not suitable to determine the effect of submergence rate. $F_0 = 0.15$ showed a different physical process compared to larger Froude numbers. $F_0 = 0.20$ was only tested for one submergence rate. $F_0 = 0.30$

was only tested between submergence rates of 1.4 and 1.8, and $F_0 = 0.35$ could not be created for submergence rates larger than 1.4. Only $F_0 = 0.25$ was tested for the full range of submergence rates between 1.1 and 2.0 at regular intervals.

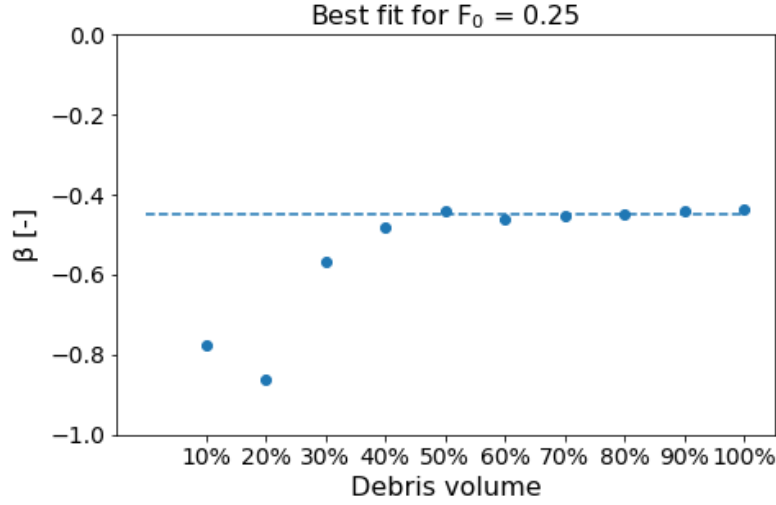


Figure 7.6: Best fit values for β in Eq. 7.10, using data corresponding to $F_0 = 0.25$.

It can be concluded that p_d is a function of the submergence rate to the power $\beta = -0.45$, leading to:

$$p_d \sim \left(\frac{h_0}{\delta} \right)^{-0.45}$$

Effect of Froude number

Next, a similar approach is used to better understand the effect of Froude number. Eq. 7.11 is used as the fitting equation, and assumes that p_d is a function of the Froude number to a power b . The data corresponding to the submergence rate of 1.4 was selected as it has the most detailed data on Froude number. However, based on Figure 7.4 it was decided to exclude $F_0 = 0.15$ in this analysis because it showed a different physical process compared to larger Froude numbers (as was investigated in Section 5.5).

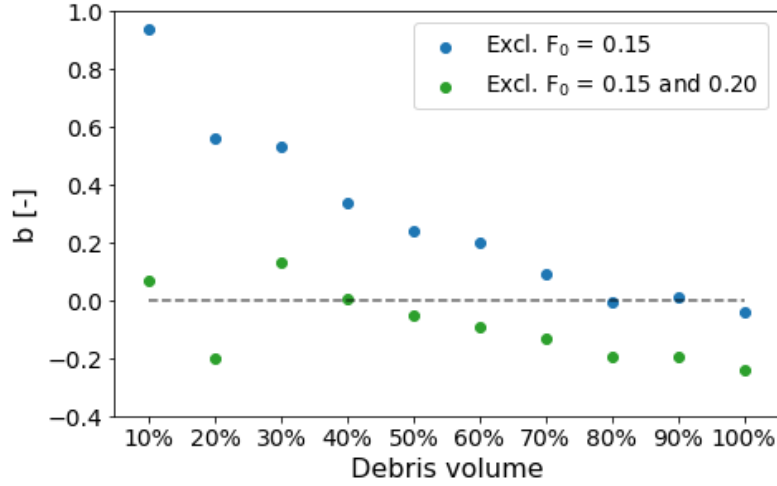
$$p_d = k_{F_0} \cdot F_0^b \quad (7.11)$$

Figure 7.7 shows the values of b (in blue) that were obtained from applying Eq. 7.11 to the experimental data. It turns out that for higher debris volumes the Froude number indeed seems to have no influence on p_d , as the power b approaches an equilibrium at 0.

Looking at the lower volumes however some influence of Froude number is observed. In Section 5.5 it was found that a threshold velocity for downward transport of logs was not exceeded for $F_0 = 0.15$, but for $F_0 = 0.25$ and $F_0 = 0.35$ this was the case for at least the three smallest log sizes. Going back to Figure 7.4 it can be observed that the data points corresponding to $F_0 = 0.25$, 0.30 and 0.35 show consistent behaviour. Until about 50% of the volume is added the data points corresponding to $F_0 = 0.20$ are somewhat lower than those corresponding to the higher Froude numbers. The reason for this may be that for $F_0 = 0.20$ the threshold velocity may be exceeded for one size class less than for the higher Froude numbers.

For this reason the same analysis is applied excluding $F_0 = 0.20$ in addition to $F_0 = 0.15$, and Figure 7.7 shows (in green) that this indeed largely eliminates the dependency of p_d on Froude number.

To conclude, no clear dependency of p_d on F_0 can be found within $F_0 = 0.20$ - 0.35 .

Figure 7.7: Best fit values for b in Eq. 7.11, using data corresponding to $h_0/\delta = 1.4$.

Effect of volume and combined equation

Lastly, the effect of volume needs to be described. So far it was established that p_d is a function of the submergence rate to the power -0.45, but does not depend on F_0 within $F_0 = 0.20$ -0.35. Therefore a combined fitting equation in the form of Eq. 7.12 is applied incorporating the effect of volume, submergence rate and a general correction factor k . Note that c and k are fitting constants.

$$p_d = k \cdot \ln(1 + c \cdot V_s / (B\delta^2)) \cdot \left(\frac{h_0}{\delta}\right)^{-0.45} = 0.17 \cdot \ln(1 + 27.8 \cdot V_s / (B\delta^2)) \cdot \left(\frac{h_0}{\delta}\right)^{-0.45} \quad (7.12)$$

Applying this fitting equation to the data results in best fit values of $c = 27.8$ and $k = 0.17$. Note however that only the data corresponding to $F_0 = 0.20$ -0.35 was selected as this showed consistent results in terms of p_d . Applying Eq. 7.12 to cases where $F_0 < 0.20$ will generally lead to an overestimation of p_d and therefore higher head loss, meaning that resulting values can be applied in practice but will be conservative.

The lines in Figures 7.4 and 7.5 were plotted using Eq. 7.7. Similar figures for all tested submergence rates can be found in Appendix D.

In engineering applications the table in Figure 7.8 may be applied to determine the debris blockage p_d .

Vs / (B δ^2)	Submergence rate		Within tested range			Extrapolated				
	1.2	1.4	1.6	1.8	2	2.2	2.4	2.6	2.8	3
0.1	20.8%	19.4%	18.3%	17.3%	16.5%	15.8%	15.2%	14.7%	14.2%	13.8%
0.25	32.5%	30.3%	28.5%	27.0%	25.8%	24.7%	23.8%	22.9%	22.2%	21.5%
0.5	42.3%	39.5%	37.2%	35.2%	33.6%	32.2%	31.0%	29.9%	28.9%	28.0%
0.75	48.3%	45.1%	42.4%	40.2%	38.4%	36.8%	35.3%	34.1%	33.0%	32.0%
1	52.6%	49.1%	46.2%	43.8%	41.8%	40.1%	38.5%	37.2%	35.9%	34.8%
1.25	56.0%	52.2%	49.2%	46.7%	44.5%	42.6%	41.0%	39.5%	38.2%	37.1%
1.5	58.8%	54.8%	51.6%	49.0%	46.7%	44.7%	43.0%	41.5%	40.1%	38.9%
1.75	61.1%	57.0%	53.7%	50.9%	48.6%	46.5%	44.8%	43.2%	41.8%	40.5%
2	63.2%	59.0%	55.5%	52.7%	50.2%	48.1%	46.3%	44.6%	43.2%	41.8%
2.25	65.0%	60.7%	57.1%	54.2%	51.7%	49.5%	47.6%	45.9%	44.4%	43.0%
2.5	66.6%	62.2%	58.5%	55.5%	53.0%	50.7%	48.8%	47.1%	45.5%	44.1%
2.75	68.1%	63.5%	59.8%	56.7%	54.1%	51.8%	49.9%	48.1%	46.5%	45.1%
3	69.5%	64.8%	61.0%	57.9%	55.2%	52.9%	50.8%	49.0%	47.4%	46.0%

Figure 7.8: Values for debris blockage p_d

Design equation converted to head loss

Once the debris blockage p_d has been determined using Eq. 7.12, the theoretical framework summarised in Figure 7.3 can be applied to convert this to a value for backwater rise. First, the combined blockage p as a result of both the grate and debris calculated using Eq. 7.9. Next, the loss factor K_{gd} due to grate and debris combined is computed using Eq. 7.8. The head loss can then be found by $\Delta H = K_{gd} \cdot U_{cul}^2 / (2g)$.

A step-by-step guide on how to calculate the debris blockage p_d and how to convert it to a value for backwater rise is given in Appendix E.

The results are visualised in Figures 7.9 and 7.10 below. The fainter symbols visualise individual repetitions, and the squares give the averages of all repetitions. Similar figures for all tested submergence rates can be found in Appendix D.

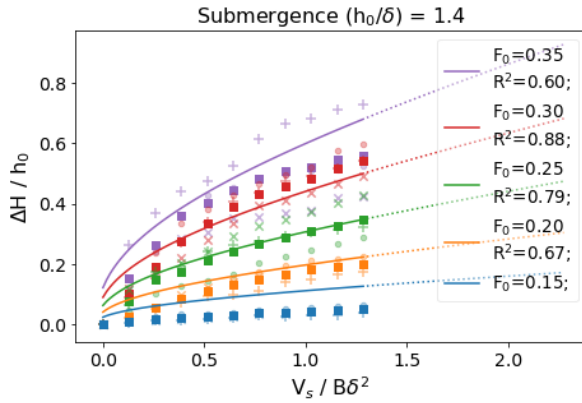


Figure 7.9: Backwater rise results for $h_0/\delta = 1.4$. Squares give averages of all three repetitions of measurements, fainter icons indicate individual repetitions. Lines visualise the outcome of the design equation.

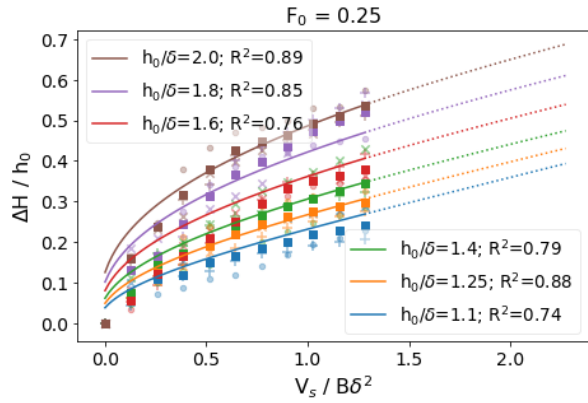


Figure 7.10: Backwater rise results for $F_0 = 0.25$. Squares give averages of all three repetitions of measurements, fainter icons indicate individual repetitions. Lines visualise the outcome of the design equation.

Discussion

8.1. Reflection on results

In Chapter 6 it was determined through a momentum balance equation that backwater rise associated with debris can well be described by neglecting the canopy drag and only considering inlet blockage. This was verified within the experimental range of $V_s / (B\delta^2) < 1.28$. At the upper end of this range the debris blockage p_d was observed to be steadily increasing still, but eventually when the debris volume becomes sufficient large p_d will reach a constant value. As this happens form drag due to inlet blockage will follow a similar trend, but canopy drag will persist to increase. As a result, the assumption that canopy drag may be neglected might not be valid for $V_s / (B\delta^2)$ well above 1.28.

The momentum balance analysis mentioned above followed the same methodology as proposed by Follett et al. (2020) for debris accumulations at pile structures. Interestingly, they found a momentum balance equation that was dominated by canopy drag, whereas in this work inlet blockage was found to be dominant for fully submerged culverts. A possible explanation for this difference was suggested in Section 7.2, being as follows: "When the debris volume at a pile structure becomes larger, the cross-sectional blockage at the structure is increased and together with a larger canopy drag this increases backwater rise. That increase in water depth then reduces the cross-sectional blockage at the structure. At submerged culverts a larger debris volume also increases the cross-sectional blockage of the inlet and thereby backwater rise, but that backwater rise hardly reduces the cross-sectional blockage in turn as the inlet was already fully submerged."

In Section 7.2 the experimental results were compared to the design Equation 7.1 proposed by Schalko et al. (2019) for debris accumulations at pile structures. It was found that this equation severely underestimated the measured head loss at fully submerged culverts. Besides the fundamental difference between pile structures and culverts explained in the previous paragraph, another main reasons due to which Eq. 7.1 does not match the data is because it is unable to take the presence of a grate into account. If the experiments were performed with a different grate, or no grate at all, the measured backwater rise could change drastically whereas this would not influence the values predicted by Eq. 7.1.

The loss factor $K_{gd} = c_{D,grate} \left(\frac{p}{1-p} \right)$ for inlet blockage associated with grate and debris is of similar form to $K_g = k_F \left(\frac{p}{1-p} \right)^{3/2}$ proposed by Meusburger (2002), or $K_g = \frac{p}{(1-p)^{7/3}}$ by Osborn (1968) as documented by Raynal et al. (2013), both of which apply to vertical trash racks. In addition, Raynal et al. (2013) found $K_g = k_F \left(\frac{p}{1-p} \right)^{1.65}$ as a best fit to describe their experimental data. It is important to note that defining $c_{D,grate}$ as a function increasing with p rather than using a constant k_F produces similar effects on K_{gd} as the power terms have in the equations offered in literature.

8.2. Reflection on methods used

As part of this research experiments were performed in which the initial upstream Froude number, submergence rate and debris volume were governing parameters. The data analysis showed that backwater rise was primarily caused by debris blocking the inlet. Consequentially, backwater rise is largely governed by the ability of the flow to drag floating debris down towards the inlet. Section 5.3 hypothesised that a Froude number using the log diameter as length scale (rather than the upstream water depth) could be a more suitable means to describe the accumulation process when dealing with cylindrical floating debris.

It is possible that if this alternative Froude number was systematically varied throughout the experiments a clear dependency of backwater rise on Froude number would have been found. Both the velocity scale and log diameter should be systematically varied to get the best results. Experiments with one log size at a time are deemed essential to understand the conditions exact for downward transport towards the inlet.

Eq. 6.15 stated that the drag coefficient for rigid grates can be approximated by $c_{D,grate} = 1 + 1.37p + 0.78p^2$. However, this equation was verified by Yu et al. (2023) for blockage ratios of up to 17% which is significantly lower than values found in this research. The validity of this equation for significantly larger blockage ratios is uncertain, especially when full blockage is approached.

8.3. Validity range of design equation

The design equation given as Eq 7.12 was derived from experiments under certain conditions. The submergence rate was varied from 1.1 to 2.0. Debris was always added up to a dimensionless volume $V_s/(B\delta^2)$ of 1.28. This means that the design equation is most reliable within these ranges, and while extrapolation of the results is possible it should be done with caution.

The submergence rate should always be > 1 , as the equation is only valid for fully submerged culverts and inverted siphons. It could be possible that submergence rates are larger than 2.0 in real applications. However, a case study (in Appendix A) of three inverted siphons in The Netherlands shows that typically the embankments of the upstream river section are in the order of 2.5 times as high as the structure's inlet (relative to bed level). Once the upstream water level exceeds the embankment level the river will start to overflow, and water levels are unlikely to rise much further. Taking this into consideration, the submergence rate should seldom have to be extrapolated by large amounts.

The experiments were performed up until the largest initial upstream Froude numbers that could be generated in the setup, while maintaining fully submerged and outlet-controlled conditions. Given that the setup was based on real structures and Froude similarity was maintained, it is likely that Froude numbers in real applications have similar maxima to those observed in the experiments. For these reasons approximately the full range of Froude numbers that will occur in practice should be covered by the experiments.

The debris volume that will accumulate at a structure can vary greatly depending on flood intensity and duration, amount of vegetation, seasonal variability and river slope. That being said, in conditions typical for the south of The Netherlands where the inverted siphons of Appendix A are located, dimensionless debris volumes are unlikely to exceed 0.5 based on past flood events. In mountainous areas dimensionless debris volumes larger than 1.28 are plausible, but on the other hand a larger channel slope makes outlet-controlled conditions less likely. Furthermore, if debris volumes are much larger the ratio of skin friction drag to form drag due to inlet blockage might increase. For these reasons the recommended range is $V_s/(B\delta^2) < 3$.

Conclusions and recommendations

9.1. Conclusions

Accumulations of floating debris at culverts and inverted siphons cause an increase in head loss that is also referred to as backwater rise. This effect is important for flood management purposes, but a lack of fundamental understanding prevents this backwater rise term from being addressed adequately in practice or design guidelines. Therefore this thesis studied the development of floating debris accumulations at fully submerged and outlet-controlled culverts and inverted siphons, and analysed the effect that floating debris accumulations have on backwater rise. This backwater rise originates from a canopy drag at the accumulation, and a form drag due to inlet blockage. Existing literature offered little knowledge on the influence of each of these terms for the case of fully submerged culverts and inverted siphons, making it difficult to derive accurate design equations.

For these reasons the following three research objectives were stated in the first chapter of this report:

1. Understand the mechanisms that govern the development of floating debris accumulations at fully submerged culverts and inverted siphons.
2. Determine the influence of canopy drag and form drag due to inlet blockage on debris head loss at fully submerged culverts and inverted siphons.
3. Develop a method to predict backwater rise due to floating debris accumulations at fully submerged culverts and inverted siphons.

To meet these objectives experiments were performed using a scale model of a culvert in a flume. First, the measured backwater rise without debris was compared to theoretical methods and a strong agreement was found, validating the applicability of relevant theories. Next, experiments were performed where debris was gradually added under varied upstream Froude numbers and submergence rates. Based on a visual analysis and a feasibility analysis three outcomes for floating logs colliding with an existing debris accumulation were suggested: (1) the log is blocked and becomes part of the accumulation, (2) upon impact with the accumulation the log is deflected downward, (3) upon impact one end of the log sticks into the accumulation, creating a hinging point, and the log rotates around this point to move downward and further downstream. The possible occurrence of these mechanisms was investigated and confirmed using simple models based on drag forcing and buoyancy (objective 1). Only mechanisms 2 and 3 actively contribute towards inlet blockage, the relevance of which is explained below.

To determine the influence of canopy drag and inlet blockage on debris head loss (objective 2) a momentum balance equation was derived between the hydrostatic forcing and drag forcing. The latter was divided into two components: a term for form drag caused by inlet blockage and a term for canopy drag caused by skin friction at the debris accumulation. First, this equation was verified using the data without debris and good agreement was found between the calculated blockage of the inlet and the directly measured blockage due to the grate. Following this, flow velocities through the debris accumulations were derived, as the measurement device was only able to measure flow velocities below the debris accumulation. Generally the derived flow velocities through the accumulation were lower than those at the highest data point below the accumulation because of increased friction, and results were deemed realistic. At a few locations non-uniformity of the accumulation across the width resulted in slightly higher velocities through the accumulation. This data was then combined with the momentum balance equation to calculate the influence of the canopy drag and inlet blockage on the total drag forcing. It was found that the influence of canopy drag was only between 5.9% and 9.5% (inlet blockage amounted between 70.4% and 82.2%) for a dimensionless debris volume $V_s/(B\delta^2)$ of

1.28 and initial upstream Froude numbers of 0.15, 0.25 and 0.35. Therefore it was concluded that backwater rise can well be described by excluding the canopy drag term in the case of fully submerged culverts. As discussed in Chapter 8, this assumption is considered valid for debris volumes $V_s / (B\delta^2) < 3$.

Using the knowledge that drag as a result of inlet blockage is the main cause of head loss, a theoretical framework was derived that relates the inlet blockage ratio to backwater rise. To generalise the results the inlet blockage ratio was separated into different components for blockage as a result of the grate and debris. A regression analysis was then applied to find a design equation for the debris blockage (objective 3). The analysis showed that debris blockage decreases with submergence rate to the power -0.45, but no clear dependency on the upstream Froude number was observed.

It was found that a Froude number using the upstream water depth as length scale is not an effective dimensionless parameter to describe the development of floating debris accumulations. The data analysis showed that this Froude number has some influence on the debris blockage, as the data corresponding to $F_0 = 0.15$ (and 0.20 to a small degree) showed different behaviour than the higher Froude numbers. However, beyond this no clear dependency of the debris blockage on Froude number could be found. From a balance between drag forcing and buoyancy it was found that a Froude number using the log diameter as length scale would likely be more effective in describing the accumulation process. Preliminary investigations in this research show that the accumulation length reduces more gradually with increasing inflow velocity than with upstream Froude number, and thereby support this hypothesis.

9.2. Recommendations

The results of the present work may be used for scientific purposes and in practical applications. The design equation offers valuable insights into the backwater level increase associated with floating wooden debris that is plausible to occur during extreme events. This information is essential for accurate predictions of water levels during floods, and allows engineers to quantitatively take this backwater rise term into account in hydraulic designs of culverts and inverted siphons. Appendix E gives a step by step guide on how the design equation may be applied.

The main recommendation for future research is to perform similar experiments with one log size at a time, and systematically vary the inflow velocity and log diameter. This will provide better insights into the exact conditions that allow for downward transport of a log towards the inlet, and could result in a clear dependency of debris blockage on Froude number.

A second recommendation for future research is to validate the empirical equation for the drag coefficient of rigid grates for blockage ratios above 17%. This will improve the reliability of the design equation, especially for higher debris volumes.

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Appendices

A

Reference inverted siphons

A.1. Tungalroyse beek inverted siphon

The Tungalroyse beek inverted siphon is located east of Weert, The Netherlands, at the canal Wessem-Nederweert (coordinates: 51.236851666768395° N, 5.795904760585382° E). It consists of 3 round pipes with an internal diameter of 1.65 m and a center-to-center distance of 3.30 meter. The downward and upward sloping parts of the barrel are relatively steep at 18 degrees, corresponding to a bed level drop of 4 meter over a horizontal length of 12 meter. The total length is 90 meter. In front of the inlet a debris rack is placed, as well as a claw machine to grab and remove debris that has accumulated at the aforementioned rack. The upstream river approaches the inverted siphon with a mild bend, and the downstream river section is straight.



Figure A.1: Inlet of the Tungalroyse beek inverted siphon



Figure A.2: Outlet of the Tungalroyse beek inverted siphon

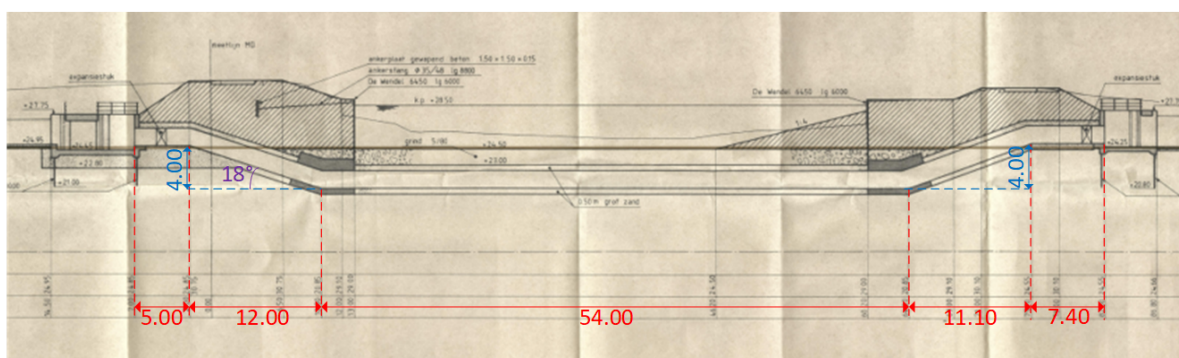


Figure A.3: Side view of the Tungalroyse beek inverted siphon (technical drawing obtained from Waterschap Limburg)

A.2. Geul inverted siphon

The Geul inverted siphon is located west of Bunde, The Netherlands, at the Juliana canal (coordinates: 50.89707806809559° N, 5.7172944268918275° E). It consists of 5 rectangular pipes of 2.50 meter wide and 2.50 meter high, divided by 0.20 meter thick walls. It has a relatively large bed level drop of 5.50 meter. The slope angle is 11 degrees. The horizontal section of 16 meter is short compared to the total length of 70 meter. Both the upstream and downstream river sections are straight.



Figure A.4: Inlet of the Geul inverted siphon



Figure A.5: Outlet of the Geul inverted siphon

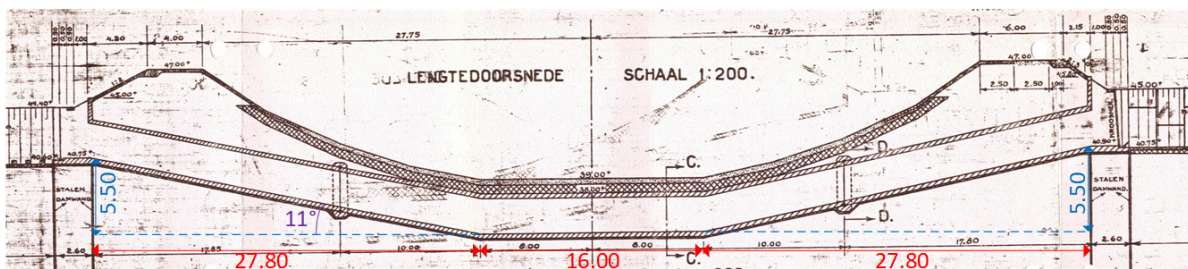


Figure A.6: Side view of the Geul inverted siphon (technical drawing obtained from Waterschap Limburg)

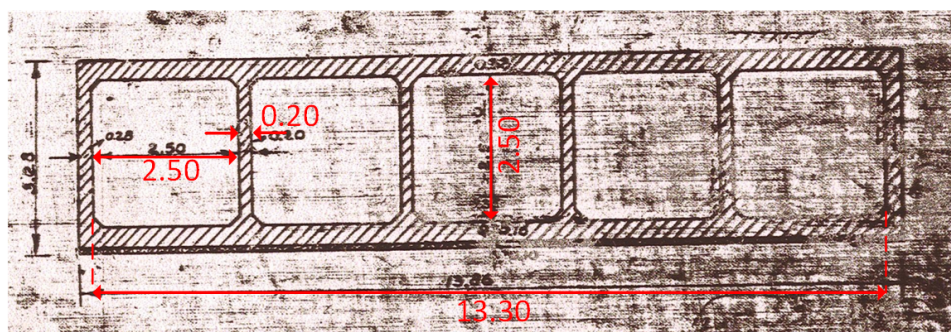


Figure A.7: Cross section of the Geul inverted siphon (technical drawing obtained from Waterschap Limburg)

A.3. Geleenbeek inverted siphon

The Geul inverted siphon is located south of Echt, The Netherlands, at the Juliana canal (coordinates: 51.09348026452907° N, 5.837727853374077° E). It consists of 4 rectangular pipes of 2.35 meter wide and 1.90 meter high, divided by 0.20 meter thick walls. It has a bed level drop of 3.50 meter. The slope angle is 11 degrees. The horizontal section is 53 meter long. The total length is 102 meter. In front and above of the inlet a highway bridge was constructed after the initial construction of the inverted siphon, and as a result the channel geometry directly upstream is nearly equal to the inlet cross section. The upstream river section arrives through a bend, and the downstream river section is straight.



Figure A.8: Inlet of the Tungalroyse beek inverted siphon



Figure A.9: Outlet of the Tungalroyse beek inverted siphon

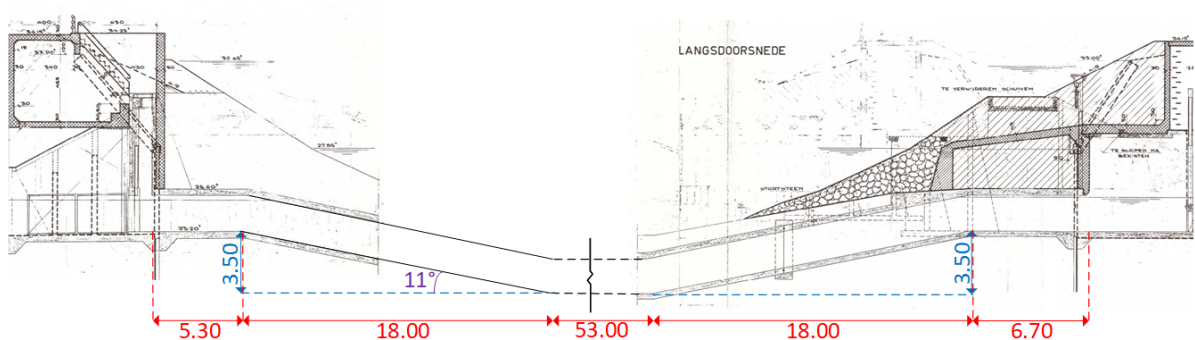


Figure A.10: Side view of the Geleenbeek inverted siphon (technical drawing obtained from Waterschap Limburg)

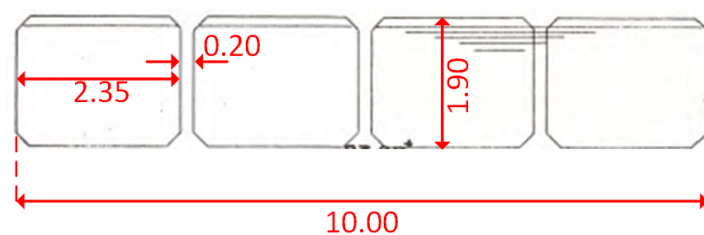


Figure A.11: Cross section of the Geleenbeek inverted siphon (technical drawing obtained from Waterschap Limburg)

A.4. Representative prototype

Figure A.12 gives a prototype inverted siphon that is representative of the Tungelroyse beek, Geul and Geleenbeek inverted siphons. Initially a 1:25 scale model of this prototype was going to be used for the experiments. However, it turned out this was too difficult to construct with the resources available, and it was decided to use the simpler prototype as given in Section 3.1.

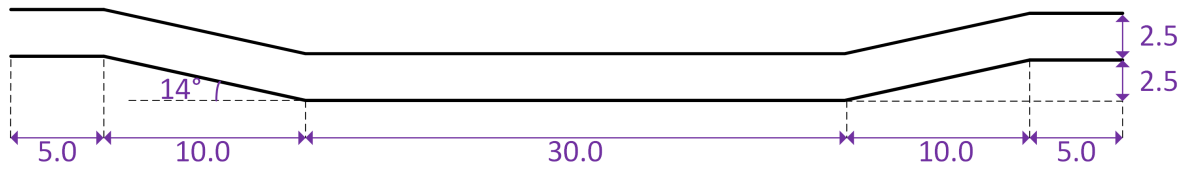


Figure A.12: Dimensions of the inverted siphon prototype, expressed in meter.

Table A.1 compares the prototype to the reference inverted siphons based on ten properties. The percentages in rows 1 to 8 indicate the prototype dimensions relative to each reference structure. The marks in rows 9 and 10 indicate whether or not the prototype shares this property with the reference structure. Most properties of the prototype are some kind of average across the reference structures, but the length and excavation depth were reduced to allow a larger scale to fit in the flume. In reducing the length and excavation depth, the representative angle was deliberately maintained.

		Tungelroyse beek	Geul	Geleenbeek	Prototype
1.	Horizontal section at inlet and outlet [m]	6.0 -17%	0 N/A	6.0 -17%	5.0
2.	Angle [degrees]	18 -22%	11 +27%	11 +27%	14
3.	Excavation depth [m]	4.0 -38%	5.5 -55%	3.5 -29%	2.5
4.	Sloped length [m]	12.0 -17%	27.8 -64%	18.0 -44%	10.0
5.	Horizontal length [m]	54.0 -44%	16.0 +88%	53.0 -43%	30.0
6.	Total length [m]	89.5 -33%	71.6 -16%	101.0 -41%	60.0
7.	Opening height [m]	1.65 (diameter) N/A	2.5 ±0%	1.9 +32%	2.5
8.	Flow width [m]	3 × 1.65 (diameter) N/A	12.5 -20%	10.0 ±0%	10.0
9.	Opening shape	round ✗	rectangular ✓	rectangular ✓	rectangular
10.	Debris rack	yes ✓	no ✗	no ✗	yes

Table A.1: Inverted siphon properties. Percentages indicate how the prototype relates to each of the reference inverted siphons.

B

Velocity and stress profiles

This appendix gives the velocity and stress profiles for the data corresponding to $F_0 = 0.15$ and $F_0 = 0.25$. The profiles corresponding to $F_0 = 0.35$ can be found in Figure 6.4.

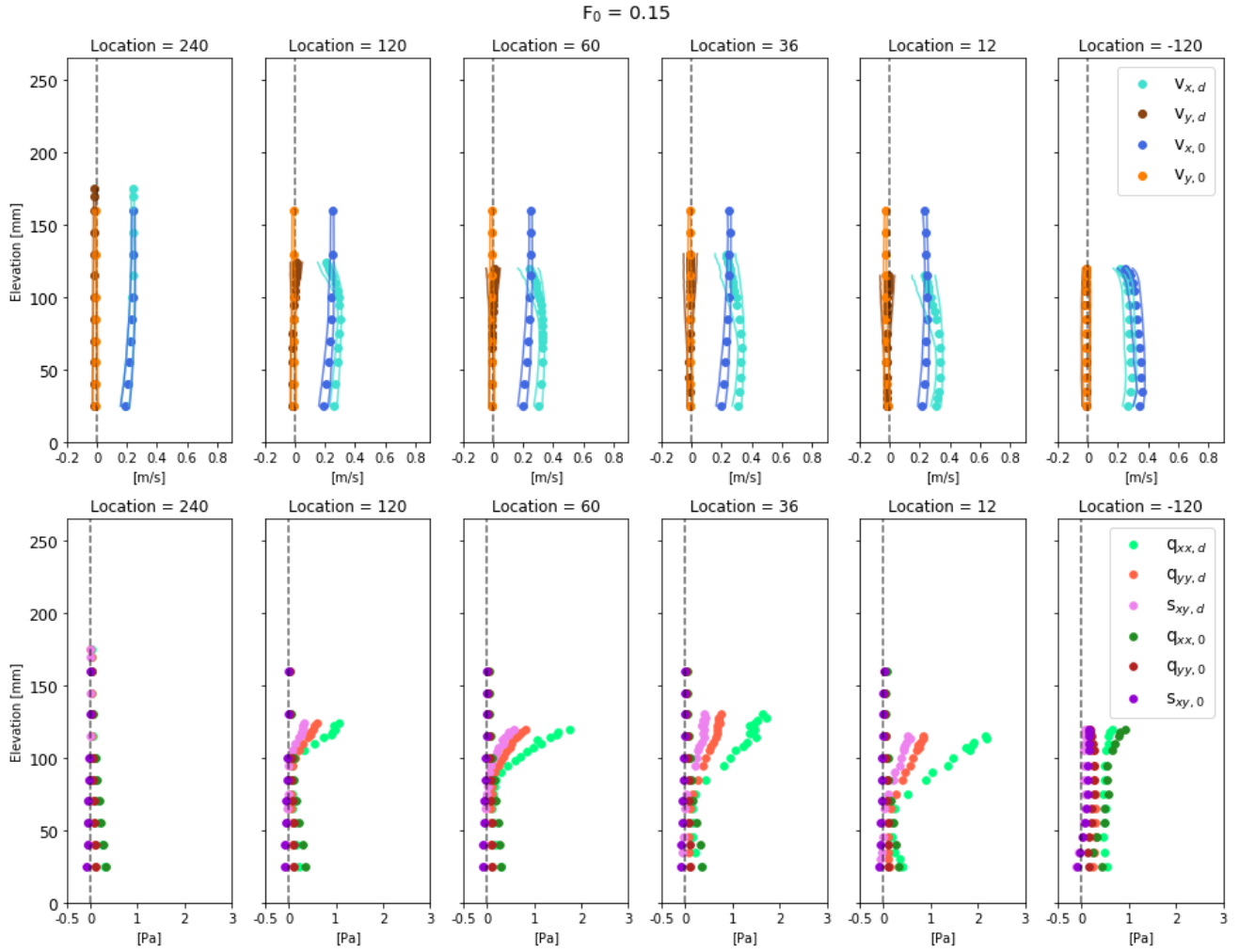


Figure B.1: Velocity and stress profiles for conditions of $F_0 = 0.15$ and $h_0/\delta = 1.4$. Subscript 'd' refers to debris conditions, subscript '0' to initial conditions. Solid lines show the 99% confidence interval of velocity fluctuations.

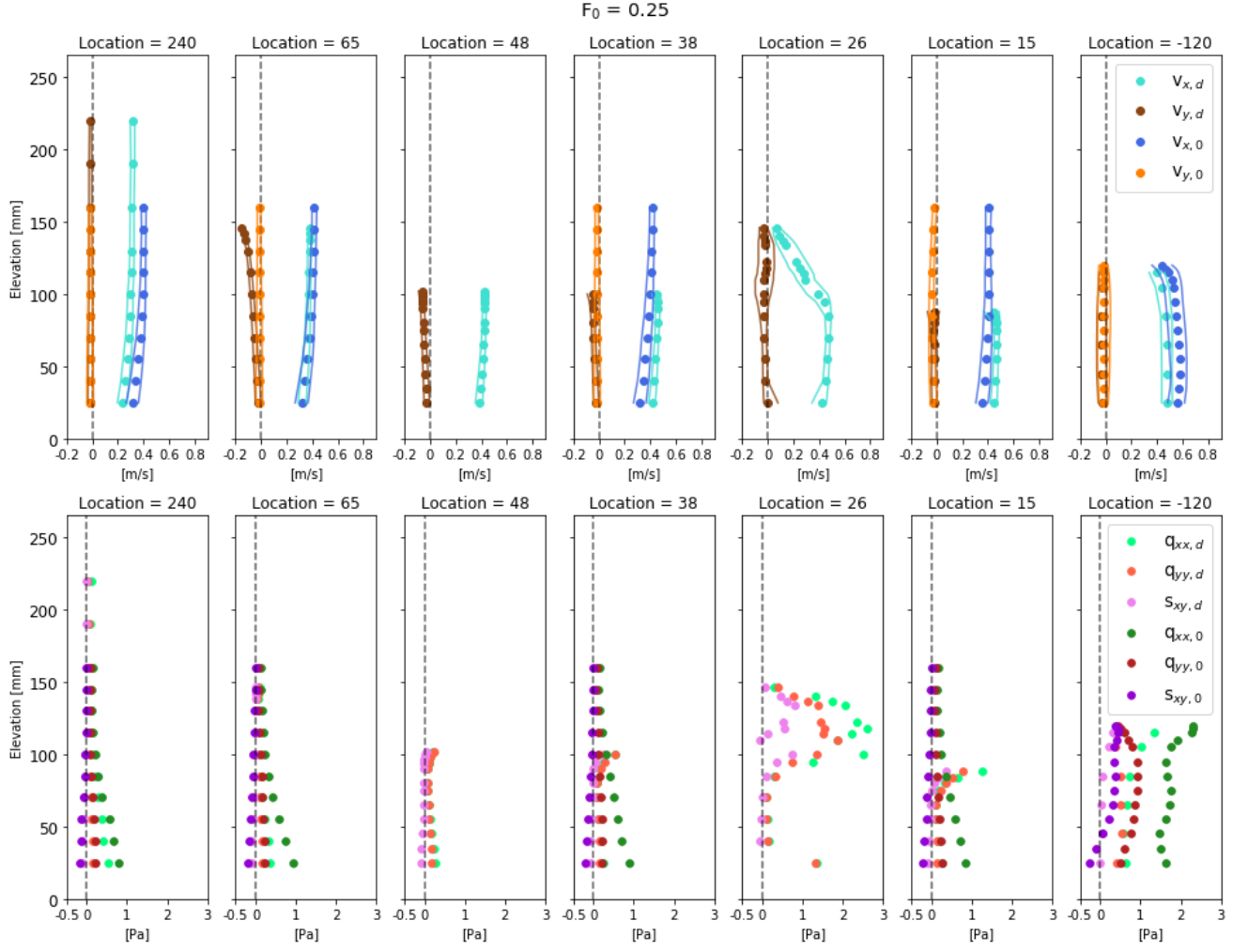


Figure B.2: Velocity and stress profiles for conditions of $F_0 = 0.25$ and $h_0/\delta = 1.4$. Subscript 'd' refers to debris conditions, subscript '0' to initial conditions. Solid lines show the 99% confidence interval of velocity fluctuations.

C

Application of the Prandtl mixing length model

The Prandtl mixing length model can be applied to gain better insights into the proximity of the highest measurements to the debris accumulation.

A limitation of the LDA measurements is that while it measures the horizontal- and vertical velocity components at one point in the middle of the flume, it requires a clear line of sight over the entire width of the flume. If there are irregularities in the bottom of the debris accumulation, which is usually the case, the highest measurement point is only as high as the lowest irregularity in that cross-section. In practice, this means that the highest measurement at each location can be a few millimeters below the debris.

Using Eq. C.1 and the experimental data, the characteristic mixing length l_m corresponding to each data point can be obtained.

$$\overline{\rho v'_x v'_y} = -\rho l_m^2 \left| \frac{\delta v_x}{\delta y} \right| \frac{\delta v_x}{\delta y} \quad (\text{C.1})$$

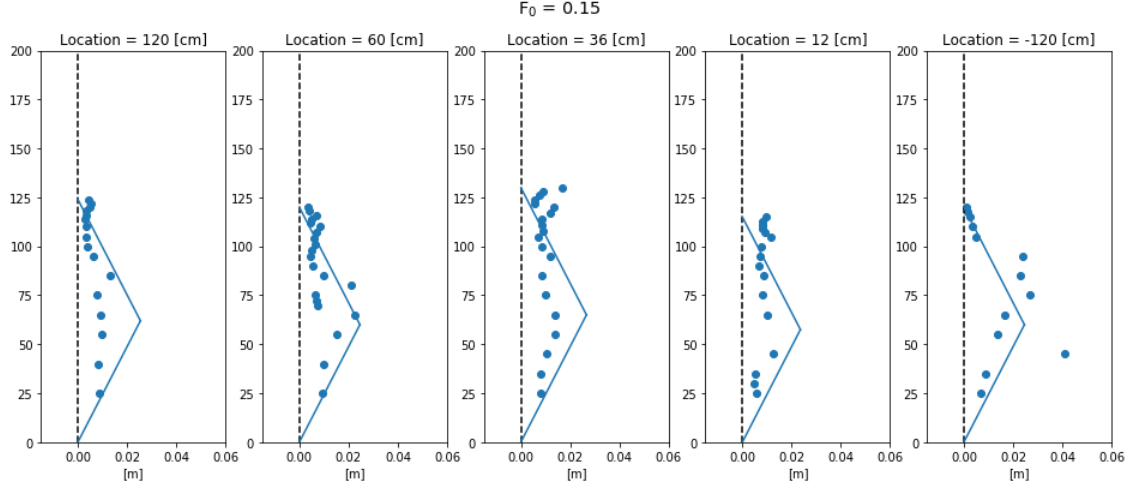
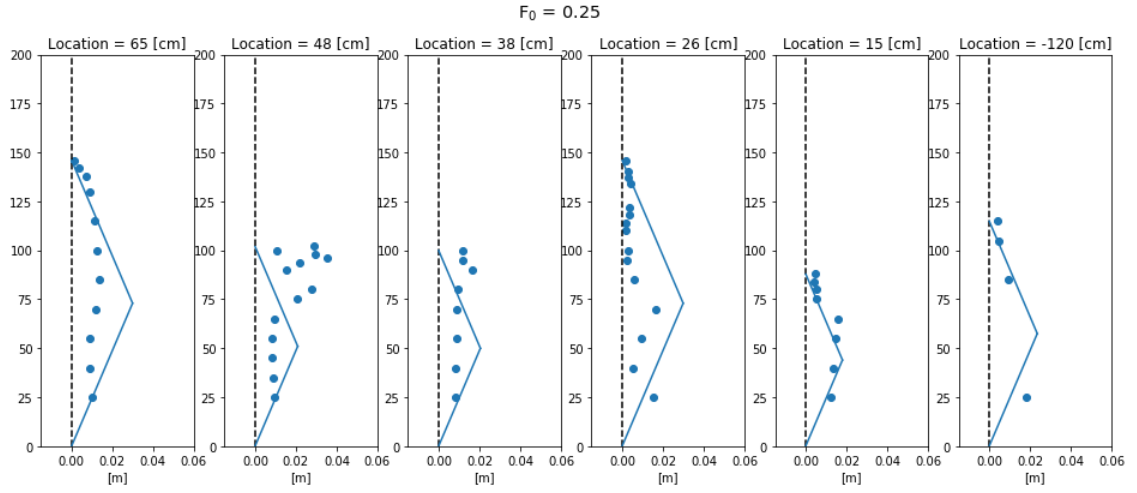
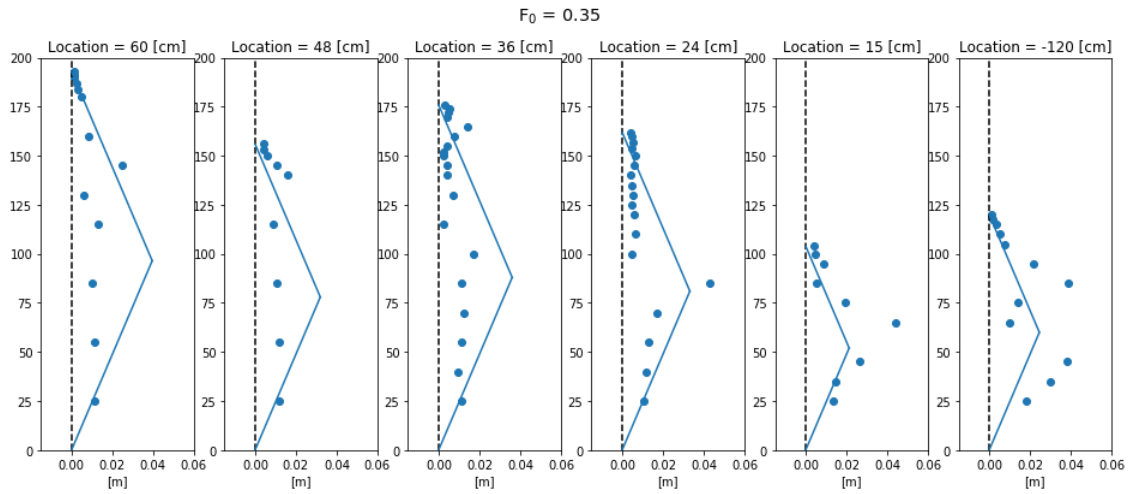
Near a wall the mixing length is often assumed to be equal to the distance to the wall multiplied by the Von Karman constant ($\kappa \approx 0.41$). By comparing the experimentally obtained mixing length to the mixing length according to the Von Karman constant, an assessment can be made whether the data point is indeed taken close to the debris boundary.

In Figures C.1, C.2 and C.3 mixing lengths are shown for various locations below the debris accumulations. Each plot gives its location as a distance upstream of the inlet. The dots are obtained from Eq. C.1, and the lines are computed by multiplying the Von Karman constant by the distance to the nearest wall. As the elevation of the debris boundary is not known exactly, it is approximated by the highest data point.

In Eq. C.1, the velocity derivative is computed using Eq. C.2 (Wang et al., 2011). As the vertical spacing between data points decreased towards the debris boundary, an equation allowing for non-uniform grids was required. This central-difference scheme gave significantly better results than a forward- or backward scheme, especially near the elevation of maximum horizontal velocity.

$$\left(\frac{\delta v_x}{\delta y} \right)_i = \left(\frac{1}{\Delta y_{i+1}} - \frac{1}{\Delta y_{i+1} + \Delta y_i} \right) v_{x_{i+1}} + \left(\frac{1}{\Delta y_i} - \frac{1}{\Delta y_{i+1}} \right) v_{x_i} + \left(\frac{1}{\Delta y_{i+1} + \Delta y_i} - \frac{1}{\Delta y_i} \right) v_{x_{i-1}} \quad (\text{C.2})$$

Taking Figure C.2 as an example, there is good agreement between the upper data points and the Von Karman approximation at the locations of 15, 26 and 65 cm upstream, indicating the measurements are taken close to the debris boundary. At the locations of 38 and 48 cm upstream there are much larger deviations, indicating a larger distance to the debris boundary. Visual observations are in line with these assessments.


 Figure C.1: Mixing lengths for conditions of $F_0 = 0.15$ and $h_0/\delta = 1.4$. Lines indicate mixing length according to Von Karman constant.

 Figure C.2: Mixing lengths for conditions of $F_0 = 0.25$ and $h_0/\delta = 1.4$. Lines indicate mixing length according to Von Karman constant.

 Figure C.3: Mixing lengths for conditions of $F_0 = 0.35$ and $h_0/\delta = 1.4$. Lines indicate mixing length according to Von Karman constant.

D

Backwater rise figures

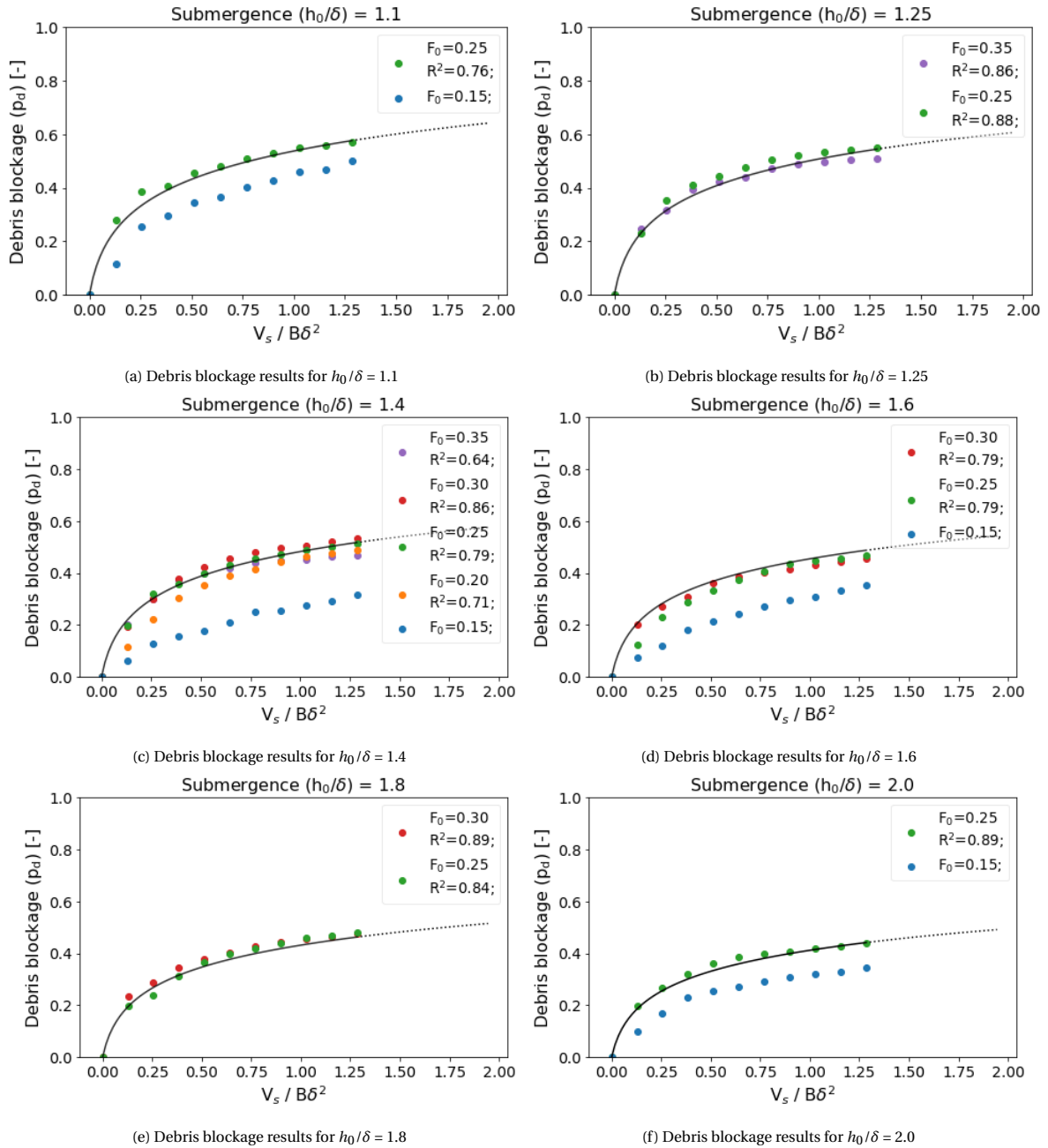


Figure D.1: Debris blockage results. Dots give averages of all three repetitions of measurements. Lines visualise the design equation.

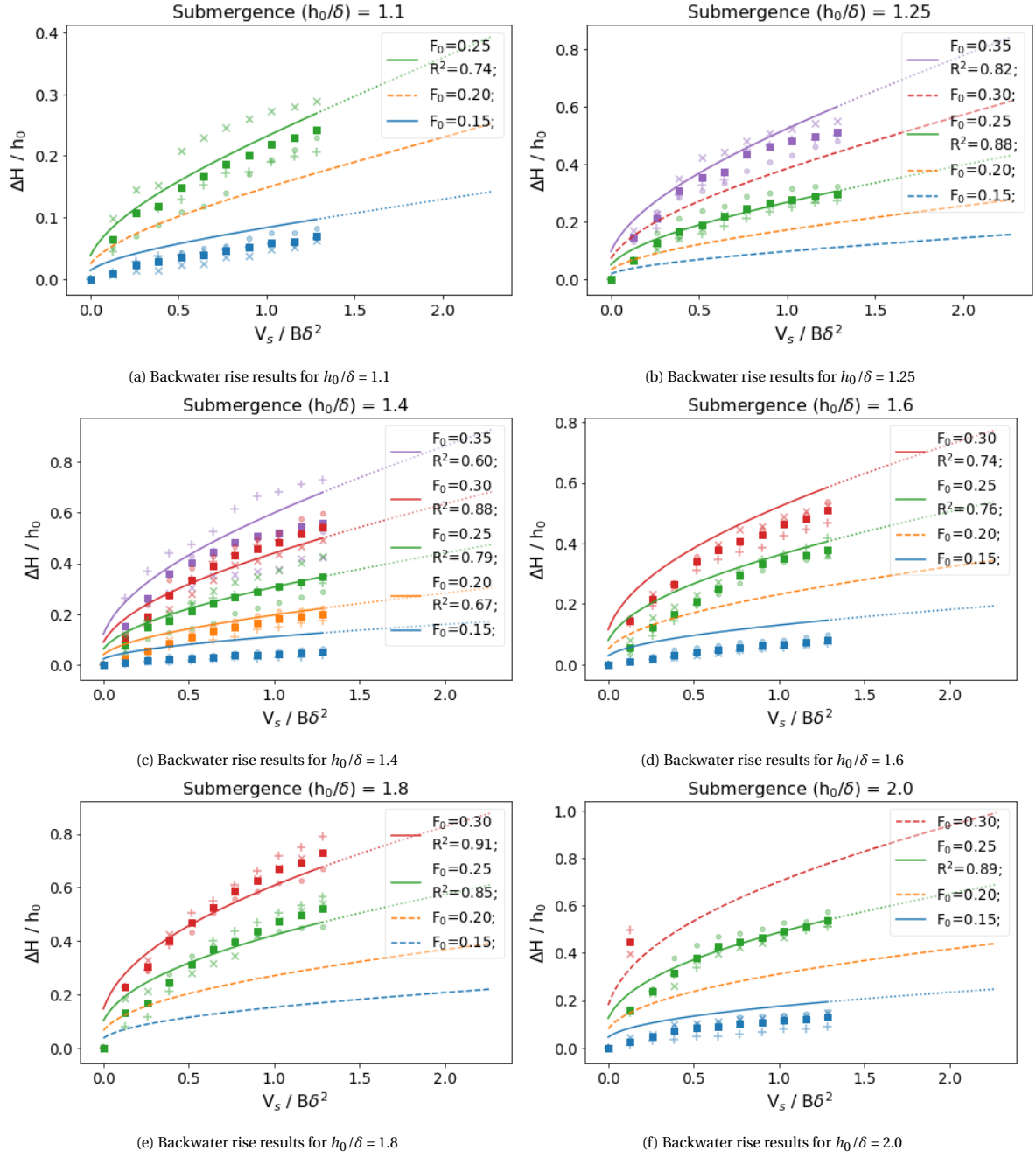


Figure D.2: Backwater rise results. Squares give averages of all three repetitions of measurements, fainter icons indicate individual repetitions. Lines visualise the outcome of the design equation.

Engineering application

1. Calculate the average flow velocity inside the culvert or inverted siphon U_{cul} , and the submergence rate (defined as the initial upstream water level h_0 divided by the height of the structure's opening δ).
2. Verify that the culvert or inverted siphon is fully submerged and has an outlet-controlled flow regime.
3. Determine the expected debris solids volume V_s . This is equal to the outer volume of the accumulation divided by (1 - porosity). One may assume the porosity to be about 76% under this flow regime.
4. Calculate the dimensionless debris volume, defined as $V_s / (B\delta^2)$ (with B being the width of the structure and approaching channel).
5. Use the table in Figure E.1, or Eq. E.1, to determine the expected debris blockage p_d .
6. Calculate the blockage p_g of the grate at the inlet (if present). This is the frontal area fraction of the inlet that is blocked by bars.
7. Use Eq. E.2 to calculate the combined blockage p . If no grate is present, p is equal to p_d .
8. Use Eq. E.3 to calculate the head loss factor K_{gd} for grate and debris combined.
9. Use Eq. E.4 to calculate the expected backwater rise.

	Submergence rate		Within tested range			Extrapolated				
$V_s / (B\delta^2)$	1.2	1.4	1.6	1.8	2	2.2	2.4	2.6	2.8	3
0.1	20.8%	19.4%	18.3%	17.3%	16.5%	15.8%	15.2%	14.7%	14.2%	13.8%
0.25	32.5%	30.3%	28.5%	27.0%	25.8%	24.7%	23.8%	22.9%	22.2%	21.5%
0.5	42.3%	39.5%	37.2%	35.2%	33.6%	32.2%	31.0%	29.9%	28.9%	28.0%
0.75	48.3%	45.1%	42.4%	40.2%	38.4%	36.8%	35.3%	34.1%	33.0%	32.0%
1	52.6%	49.1%	46.2%	43.8%	41.8%	40.1%	38.5%	37.2%	35.9%	34.8%
1.25	56.0%	52.2%	49.2%	46.7%	44.5%	42.6%	41.0%	39.5%	38.2%	37.1%
1.5	58.8%	54.8%	51.6%	49.0%	46.7%	44.7%	43.0%	41.5%	40.1%	38.9%
1.75	61.1%	57.0%	53.7%	50.9%	48.6%	46.5%	44.8%	43.2%	41.8%	40.5%
2	63.2%	59.0%	55.5%	52.7%	50.2%	48.1%	46.3%	44.6%	43.2%	41.8%
2.25	65.0%	60.7%	57.1%	54.2%	51.7%	49.5%	47.6%	45.9%	44.4%	43.0%
2.5	66.6%	62.2%	58.5%	55.5%	53.0%	50.7%	48.8%	47.1%	45.5%	44.1%
2.75	68.1%	63.5%	59.8%	56.7%	54.1%	51.8%	49.9%	48.1%	46.5%	45.1%
3	69.5%	64.8%	61.0%	57.9%	55.2%	52.9%	50.8%	49.0%	47.4%	46.0%

Figure E.1: Values for debris blockage p_d

$$p_d = 0.17 \cdot \ln(1 + 27.8 \cdot V_s / (B\delta^2)) \cdot \left(\frac{h_0}{\delta}\right)^{-0.45} \quad (\text{E.1})$$

$$p = p_g + p_d \cdot (1 - p_g) \quad (\text{E.2})$$

$$K_{gd} = c_D \cdot \frac{p}{1 - p} = (1 + 1.37p + 0.78p^2) \cdot \frac{p}{1 - p} \quad (\text{E.3})$$

$$\Delta H = K_{gd} \cdot \frac{U_{cul}^2}{2g} \quad (\text{E.4})$$