



NTNU – Trondheim
Norwegian University of
Science and Technology

Different Aspects of Flushing of Hydropower Intakes

Bjørnar Rettedal
Lars Eid Nielsen

Civil and Environmental Engineering

Submission date: June 2012

Supervisor: Leif Lia, IVM

Co-supervisor: Hanne Nøvik, IVM

Norwegian University of Science and Technology
Department of Hydraulic and Environmental Engineering



MASTER THESIS SPRING 2012

Students: *Lars Eid Nielsen*
Bjørnar Rettedal

Title: **DIFFERENT ASPECTS OF FLUSHING OF HYDROPOWER
INTAKES**

Tittel: **ULIKE FORHOLD VED SPYLING AV INNTAK FOR VANNKRAFTVERK**

1 INTRODUCTION

A well-operating intake is a prerequisite for the successful operation of a hydropower plant. The main challenges with developing good design principles for shallow intakes for hydropower plants involve sediment handling, debris, leaves, ice (frazil ice and ice drift), entrainment of air, and general hydraulic conditions. It is a major challenge to allow for all the different concerns in the design of an intake in a shallow river with rapid flow. Internationally, and specifically in the Himalayas, with a heavy rain season combined with a lot of sediments, sediment handling is of main concern. Desilting basins are common components at headworks for preventing sand erosion of the waterways and the turbines. In Norway, debris and ice accumulations at the intake construction is often a more important problem than sediment transport. Never the less, an intake design adapted to local conditions is essential. There are various principles for the design of intakes existing today, some more successful than others. Several intake structures undergo reconstruction after only a few years in service, due to problems with maintenance and operation due to a design poorly adapted to local conditions. Both private initiatives and hydropower companies are contributing with new solutions for testing. NTNU Vassdragslaboratoriet seeks to contribute to further development, verification and innovation within this area.

2 BACKGROUND

Flushing of desilting basins and sand traps is widely used, and different flushing concepts have been developed. Because of the unique conditions for every single hydropower project and the complexity of the sediment transport, physical and/or numerical model studies of the headworks are often recommended. Experiences from existing hydropower plants and available physical models are very valuable for planning of new intake constructions. A physical model of the headworks of the 93 MW Lower Manang Marsyangdi hydropower project, located in Manang District of Gandaki Zone in Nepal, is built at HydroLab Pvt. Ltd in Kathmandu, Nepal. The plant is scheduled to be commissioned in 2016.

In the last couple of years it has also been studied the possibility of cleaning intake screens with back flushing instead of the conventional method of manual or mechanical cleaning. Back flushing implicate for a short period to let water flow over the rack with the opposite direction to normal operation, and divert loosened debris/trash out of the intake pond. In order to assess the properties of the back flushing process, a physical model of a part of a full scale trash rack was built at the Vassdragslaboratoriet at NTNU autumn 2011, as a part of the project work of Lars Eid Nielsen and Bjørnar Rettedal. Back flushing proves to be an efficient cleaning method. Preliminary tests in the physical model are the basis for the setup of a test program to be conducted in the master thesis.

3 PROBLEM DESCRIPTION

Performance of different intake structures and headworks arrangements with flushing facilities should be assessed by field visits to run-off river hydropower plants in Norway and Nepal. It should also be conducted test series in two different physical models. One is the physical model of the Lower Manang Marsyangdi hydropower project, built at HydroLab Pvt. Ltd in Kathmandu, and the second is the physical model of a full scale intake screen at Vassdragslaboratoriet, NTNU.

The goal of the physical model study of the intake of the Lower Manang Marsyangdi is first and foremost to evaluate the performance of the preliminary design of the sediment handling arrangement. The test program for flushing of intake screen should be designed to find the physical conditions required for removing different debris types clogged at the screen.

Experiences from different intake arrangements at existing hydropower plants and gained knowledge from the physical model studies of sediment and debris flushing should be systematized and thoroughly reported.

4 GOAL

The goal of the master thesis is to gain experiences with hydropower intakes with different aspects of flushing. It is a goal to find and designate the physical conditions for successful flushing of sediments and debris. Uncertainties and errors should be evaluated. It should be concluded on whether the work has been successful and if there should be conducted further studies.

5 CONTACT PERSONS

NTNU

Leif Lia, Professor (supervisor)

Hanne Nøvik, PhD-student (co-supervisor)

HydroLab Ltd

Meg Bishwakaram

Padham Pokharel

Butwal Power Company (BPC)

Pratik Man Sing Pradhan

Discussions with colleagues and employees at NTNU, SINTEF, HydroLab, BPC, Tafjord Kraft AS and eventually other hydropower plants are recommended. All contributions should be correctly referred.

5 REPORT FORMAT, REFERNECES AND CONTRACT

The report should be written with a text editing software, and figures, tables, photos etc should be of good quality. The report should contain an executive summary, a table of content, a list of figures and tables, a list of references and information about other

relevant sources. The report should be submitted electronically in A4-format .pdf-file in DAIM, and three paper copies should be handed in to the institute.

The executive summary should not exceed 450 words, and should be suitable for electronic reporting.

The Master's thesis should be submitted within Monday 11th of June 2012.

Abstract

Different design criteria for successful flushing of hydropower headworks have been evaluated. Main focus has been on handling of floating debris for small hydropower plant in Norway, as well as sediment handling for run of the river-projects in sediment-carrying rivers.

As a new way of cleaning intake screens clogged by debris, the concept of backflushing has been investigated. The intake screen, called *trash rack*, is then cleaned by a reversed water flow over a short period of time, and the clogged material flushed out through a flushing pipe. A physical test tank was developed to evaluate parameters for a successful flushing of trash racks with different degrees of clogging. A test device was developed to evaluate adhesion, i.e. the level of friction between clogged material and the trash rack.

The required gross water velocity over a trash rack during flushing for detaching of the clogged material was found to be in the range of 0.1 to 0.2 m/s, with a typical value of 0.12 m/s. The clogged material tend to resist a maximum limit of pressure difference over the trash rack before it detaches. Results are showing a pressure difference resistance in the range of 0.05-0.10 meterWater Column (mWC). The pressure difference prove hard to obtain as parts of the trashrack is cleaned. Hence, a flushing gate should be opened at a high rate to obtain the best flushing efficiency.

A field trip to Nepal has been conducted to gain experience in sediment handling, and to work with physical models at HydroLab Pvt Ltd in Kathmandu. Observed intake solutions have been evaluated both against existing theory and results from a physical model study. It has been documented through a model test series that the flushing ability of an undersluice gate is limited to only a few meters upstream of the flushing gate.

The limit for flushing by bed transport has been tested in a model, and compared to theory for evaluating initial movement of the sediment bed. Experiments are supporting that Shield's theory of critical shear stress can be used to predict the occurrence of bed movement.

Design and operation of settling basins have been investigated, together with the appurtenant different strategies for flushing. The possibility of applying the concept of backflushing of trash racks for headworks arrangements including sediment settling basins have been evaluated. For projects where the trash rack can be located upstream of the settling basin, the combination seems feasible, as parts of the water storage in the settling basin could be used for the backflushing. However, the trash rack should be placed downstream of the settling basins for most Himalayan headworks arrangements, which makes backflushing impossible.

Sammendrag

I denne masteroppgaven har forskjellige kriterier for vellykket spyling av inntaksområdet blitt vurdert. Hovedfokuset har vært på håndtering av drivgods for norske småkraftverk, i tillegg til sedimenthåndtering for elvekraftverk i Himalaya. Tilbakespyling er en ny metode som har blitt utviklet for rensing av inntaksrister på småkraftverk. Ved å sette opp en reversert vannstrøm over inntaksristen, kalt varegrinda, for en kort periode, vil drivgodset løsne fra varegrinda og bli spylt ut gjennom en spyleluke. Det har blitt utviklet en fysisk modell for å vurdere parametere for vellykket tilbakespyling av varegrinder med ulik grad av tilstopping.

Nødvendig brutto spylehastighet over varegrinda for å løsne drivgodset fra varegrinda er funnet til å være 0.1 - 0.2 m/s, med en typisk verdi på 0.12 m/s. Det ser ut til at det tilstoppede materialet motstår en maksimal trykkforskjell over varegrinda før det løsner. Resultater viser at trykkforskjellen over grinda da er mellom 0.05 - 0.10 meter vannsøyle (mVS). Etterhvert som deler av drivgodset løsnet, viste det seg vanskelig å opprettholde trykkforskjellen over varegrinda. Det er derfor anbefalt å åpne spyleluken raskt for å oppnå best mulig spyleeffektivitet.

Iløpet av masteroppgaven har det blitt utført en feltbefaring til Nepal for å skaffe erfaring rundt sedimenthåndtering, og for å jobbe med fysiske modellforsøk ved HydroLab Pvt. Ltd. i Katmandu. Observerte inntaksløsninger har blitt evaluert med bakgrunn i teori og resultater fra et fysisk modellforsøk ved NTNU. Det har blitt dokumentert gjennom testforsøk at spyleeffektiviteten til en underløpsluke er begrenset til kun et kort område oppstrøms luka.

Grensesjiktet for aktivering av bunntransport har blitt testet i modellforsøk, og sammenlignet for teorien for når bunntransport skal starte. Forsøkene støtter at den kritiske skjærspenningen for Shields kan brukes til å anslå når bunntransport starter.

Design og drift av sedimentbassenger har blitt undersøkt, sammen med tilhørende spylestrategier. Muligheten for å kombinere tilbakespylingskonseptet med vannkraftanlegg med sedimenteringsbasseng har blitt undersøkt. For prosjekter hvor varegrinden er plassert oppstrøms sedimenteringsbassenget virker kombinasjonen gjennomførbar, siden vannvolumet fra sedimenteringsbassenget kan bli brukt som spylevann. Sedimenteringsbasseng i dagen vil normalt kreve installasjon av en varegrind nedstrøms bassenget, og tilbakespyling vil være et uegnet konsept.

Preface

We, Bjørnar Rettedal and Lars Eid Nielsen, have written and conducted the work described in this master's thesis. We feel humble to have been given the opportunity to work within such a wide and interesting field of hydraulic engineering, and we truly appreciate the academic outcome we have had doing it. Together with our co-supervisor, PhD student Hanne Nøvik, we have been allowed to travel to Nepal to gain experience in projects in Himalayan regions, and we have observed solutions to challenges not typically found home in Norway. Doing so, we have not only extended our academic horizon, but also received valuable input on a cultural and personal level.

This thesis would not have been possible to conduct if it had not been for helpful and caring people surrounding us. During our entire trip to Nepal, we were treated as long lost friends wherever we went, and received valuable help and assistance to obtain our goals. The friendly staff at HydroLab Pvt. Ltd. should be mentioned in particular, we are truly impressed by the working spirit and the academic level of their work.

To develop and conduct the test series performed in Norway, Geir Tesaker, together with Samuel Vingerhagen and the staff of SINTEF has been of great help. For sorting out the mysteries of sediment transport, Tom Jacobsen from SediCon has generously shared his experience in an impressively enthusiastic way. Egil Berge from Eidsdal Power Company should also be mentioned, willingly sharing experiences from his pioneering work on backflushing of trash racks.

But above all, this thesis would not have been possible without our supervisors Leif Lia and Hanne Nøvik. As a PhD-student, Hanne has provided us with continuous follow-up during the project period, and served as a caring travel partner and good friend. Showing true enthusiasm and commitment to our work, she has motivated and guided us in the right direction through the entire working period.

Trondheim, June 11, 2012


Bjørnar Rettedal


Lars Eid Nielsen

Contents

Abstract	i
Sammendrag	iii
Preface	v
1 Introduction	1
2 Backflushing of trash racks	3
2.1 Introduction	3
2.2 Theory	5
2.2.1 Intake hydraulics	5
2.2.2 Bypass of floating debris	6
2.2.3 Conventional cleaning of trash racks	7
2.2.4 Backflushing concept	8
2.2.5 Parameters for backflushing of trash rack	10
2.3 Procedure	13
2.3.1 Scope of model study	13
2.3.2 Model concept	13
2.3.3 Instrumentation and interpretation	18
2.3.4 Field measurements of trash rack adhesion	22
2.4 Results	23
2.4.1 Field measurements of trash rack adhesion	23
2.4.2 Release pressure difference	23
2.4.3 Release velocity	24
2.5 Discussion	27

3	Flushing of sediments	33
3.1	Introduction	33
3.2	Theory	35
3.2.1	Sediment transport	35
3.2.2	Settling basin design	39
3.2.3	Flushing of headworks structures	41
	Serpent Sediment Sluicing System (S4)	42
	SediCon Sluicer	45
	Bed control at intake	45
3.2.4	Model theory	46
3.3	Procedure	49
3.3.1	Field visits in Nepal	49
3.3.2	Experiments in flushing flume	50
3.3.3	Scaling of flushing range tests	54
3.3.4	Numerical model study	56
3.4	Evaluation of headworks in Nepal	57
3.4.1	Bed control at intake	57
3.4.2	Settling basins	59
3.5	Results from model study	61
3.5.1	Flushing range	61
3.5.2	Numerical model	62
3.5.3	Limit for bed load transport	63
3.6	Discussion	65
3.6.1	Lower Manang Marsyangdi Hydroelectric Project (LMM)	65
3.6.2	Flushing range experiments	68
3.6.3	Numerical model study	68
3.6.4	Verification of Shields to predict bedload transport	69
3.6.5	Modi Khola Hydroelectric Project	69
3.6.6	Lower Modi I Hydropower Project	69
3.6.7	Combination of settling basin and trash racks	70
3.6.8	Further work	71

4	Conclusion	73
	Bibliography	75
A	Trash rack adhesion tests at Bergedammen	77
B	Adhesion test measurements from laboratory	79
C	Results for backflushing test series	81
D	Photos of trash racks after flushing	83
E	Trap efficiency of settling basins	85
F	Friction calibration of pulling device	87
G	Star CCM+ setup	89
H	Test procedure for the flushing flume	93

List of Figures

2.1	Required submerging of intakes(Guttormsen, 1989)	6
2.2	Floating boom at Nedre Leirfossen power	7
2.3	Manual raking of trash rack	9
2.4	Wire rope trash rack cleaner at Leirfossen	9
2.5	A principle sketch of Bergedammen	10
2.6	Concept of the backflushing testing tank	13
2.7	Pulling Equipment for Evaluation of Trash Rack Adhesion («PEETRA»)	15
2.10	Velocity development, test 1	19
2.8	Pressure difference development over trashrack, test 1	20
2.9	Tank 2 water level development, test 1	20
2.11	Estimation of release velocity	21
2.12	Adhesion measurements at the Bergedammen intake	22
2.13	Pressure difference development during flushing tests	24
2.14	Pressure difference over trash rack at time of release	25
2.15	Required release velocity for clogged trash racks	25
2.16	Pressure difference development, test 6	27
2.17	Trash rack after flushing test 1	29
2.18	Velocity development comparison, empty channel	30
3.1	Shield's diagram	36
3.2	Sediment yield of the world	37
3.3	Sediment concentration distribution	38
3.4	Settling basin at Jhimruk in Nepal	39
3.5	Principle sketch of a settling basin	40
3.6	Flow tranquilizers at Jhimruk Hydropower Plant	40

3.7	The Serpent Sediment Sluicing System at Andi Khola in Nepal	43
3.8	Concept of the S4 system (Støle, 1993)	44
3.9	SediCon sluicer concept	45
3.10	Undersluice slot at Middle Marsyangdi HEP	47
3.11	Undersluice flushing gates at Lower Modi I, Nepal	47
3.12	Comparison of river model and prototype of LMM	49
3.13	Laboratory flushing flume	51
3.14	Placement of sediments in front of model gate	52
3.15	Flushing length l , as function of h_w and a	53
3.16	Fitting of the discharge curve for model gate	55
3.17	Bed control solution at Middle Marsyangdi	57
3.18	Downstream flushing gates at Modi Khola HEP	58
3.19	Flushing channel at Lower Modi HEP	59
3.20	Flushing range in model flume as a function of water discharge	61
3.21	Estimated flushing length in front of prototype gate	62
3.22	Shear stress distribution in front of flushing gate, from Star CCM+	63
3.23	Natural river conditions, Lower Manang Marsyangdi	65
3.24	River conditions upstream of the LMM project site	66
3.25	Estimated flushing length for Lower Manang Marsyangdi flushing gate	67
3.26	The settling basins at Kali Gandaki	71
C.1	Detailed results from backflushing test series	82
D.1	Images of trash racks after flushing tests	84
E.1	Effect of turbulence on trap efficiency	85
F.1	Calibration measurements of the PEETRA system	87

Chapter 1

Introduction

A well performing headworks arrangement is an important part of a hydropower system, and the design is vital to secure a reliable operation. A well functioning headwork design should be able to extract water from the river or the reservoir, and in a safe way handle trash, floating debris and sediments also during challenging conditions. The term headworks refer to all structural components required to extract the water from the river, clean it for floating objects, air and sediments, and deliver it to the waterways leading to the powerplant. The term intake is more narrow, and is in this report defined only to cover the structure where the water is abstracted from the river. Hence, the intake is one part of the headworks for a hydropower system. The headworks may also include eventual river training measures, weirs, gravel traps and settling basins.

Professor Haakon Støle has in his PhD thesis suggested five performance criteria that needs to be fulfilled for a well performing headworks design (Støle, 1993). According to Støle, the headworks should provide:

- Passage of floods, including hazard floods
- Passage of ice, trash and floating debris
- Passage of sediments
- Bed control at the intake
- Exclusion of suspended sediments and air

Different geographical locations requires different design, and each headworks design needs to be tailor made to suit different challenges. Norwegian conditions are allowing for relatively simple headworks solutions, as the amount of transported sediments is low. Typical operational problems are often connected to accumulation of floating debris on the intake screens, blockage by ice or entrainment of air into the waterways.

In general, run of the river projects may experience a concentrated distribution of the available discharge, where a major fraction of the annual production takes place during

seasonable floods. Hence, major operational problems during periods of high transport of sediments and/or floating debris could be critical for the economy of the projects.

The recent introduction of green certificates on renewable energy production in Norway has led to an increased focus on small scale hydropower plants and solutions to improve the intake performance. Solutions to obtain automatic self cleaning of the intake screen - called *trash rack* - have been of interest as many companies are planning to carry out new projects. Over the last years it has been developed a new concept for cleaning trash racks, that involves to reverse the water flow over the trash rack for a small period of time. The concept - called *backflushing* - has been investigated in this thesis, and laboratory work has been conducted to try to find usable design parameters.

In parts of the world where sediment problems is the main concern of the design of headworks, «flushing» is used in another context. To avoid uncontrolled accumulation of sediments in the different parts of the headworks, different strategies for flushing are applied in the design.

In this thesis, observations and experiences from field visits in Nepal has given a foundation to evaluate the performance of different headworks designs. Evaluations of the planned headworks design of the 93 MW Lower Manang Marsyangdi project in Nepal has been conducted by using experiences from field visits and results from a model study at Vassdragslaboratoriet at NTNU.

The purpose of this report is to evaluate different design considerations for headworks, and to designate physical parameters for successful flushing of both sediments and debris. Since flushing of sediments and debris describes different challenges for different geographical regions, the two topics are separated by two somewhat independent chapters that will be followed by a collective summary. As a link between the two main chapters in this thesis, the possibility of adapting the concept of backflushing of trash racks to headworks including settling basins has been evaluated.

Chapter 2

Backflushing of trash racks

2.1 Introduction

The number of small hydropower projects in Norway has an upward tendency, and the list of projects waiting for a licence to start construction is at an all time high. In Norway, small hydropower is defined as projects with installed capacity between 1 and 10 MW, and it represents a total of 6.1 % of the Norwegian power production (NVE, 2011). One of the causes for the high activity in the field of small hydropower is the recent introduction of green certificates, which ensures producers of renewable energy an increased income for their production if their project is up and running within 01.01.2020.

The Norwegian Water Resources and Energy Directorate (NVE) have estimated that there is a remaining small hydropower potential of 18 TWh with investments cost under 3 kr/kWh in Norway. Out of the remaining potential, 5 TWh is realistic to be built within a 10-year period (NVE, 2009).

The increased focus on small hydropower over the latest years has brought further attention to development of new ideas and concept suited for Norwegian conditions. Many projects are today facing challenges with operation during periods with high transport of trash and debris in the river. The intake screens that are installed to prevent trash and debris from entering the turbine - called trash racks - are often clogged by the transported material, resulting in reduced production. Self-cleaning solutions that do not require manual or mechanical removal of clogged material on the trash rack have obtained much attention over the last years, and have been subjects for an increased research effort. Backflushing of trash racks is among the concepts for cleaning of intake structures that is considered to be promising for challenging conditions, and it is of great interest to gain experience on the principles and criteria for a successful design.

2.2 Theory

In the following sections, theory related to intake hydraulics and the topic backflushing is presented. The Norwegian Water Resources and Energy Directorate (NVE) have made guidelines and manuals to ensure quality in built projects for Norwegian small hydropower plants, and this master thesis has used NVE's «Inntakshåndboken» (Jenssen et al., 2006) («The intake manual», only available in Norwegian) as a main source.

2.2.1 Intake hydraulics

Dimensioning of trash racks

Trash racks are designed to prevent unwanted debris like leaves, branches and ice to enter the power station. The bar spacing of the trash racks depends on the least sized opening in the power station, i.e. any particle passing the trash rack shall be able to pass through the power station and turbine without jamming or disturbing the machinery. For cleaning of the trash rack, an incline of 5-10 degrees is recommended. The intake should be designed so that cleaning always is possible, i.e. also during floods. An economic optimization of the trash rack size will often result in gross velocity through the trash rack less than 0.5 m/s. For more details concerning dimensioning of trash racks, the authors recommends to look into more specific literatures, like Inntakshåndboken (Jenssen et al., 2006).

Hydraulic losses over a trash rack

Hydraulic losses over the trash rack during operation is an important factor when designing intakes. The hydraulic losses over a trash is a function of the water velocity and the trash rack geometry, can be calculated using different formulas. Kirschmer-Mosonyis is the most commonly used method for evaluation of clean trash racks, details can be found in Jenssen et al. (2006). In many projects, especially for run-of-the-river projects, the trash rack will for longer periods not stay clean. Meusburgers formula, as described in Inntakshåndboka (Jenssen et al., 2006) also takes clogging into consideration when calculating head loss.

Submerging

Sufficient submerging is important to avoid unwanted air entrainment into the intake which in turn can lead to vibrations in the turbine. Vortexes can also divert floating debris down to the trash rack. Different criterions has been developed of necessary submerging to avoid vortexes. Lia and Jenssen (2003) gives a coarse minimum submerging of $H_{free} = 2 \cdot D_{min}$, where D_{min} is the diameter of the water pipe. Guttormsen (1989) describes another one which is $S_i = 0.6 \cdot v_i \cdot D$, depending of the size of the intake and water velocity (Figure 2.1)

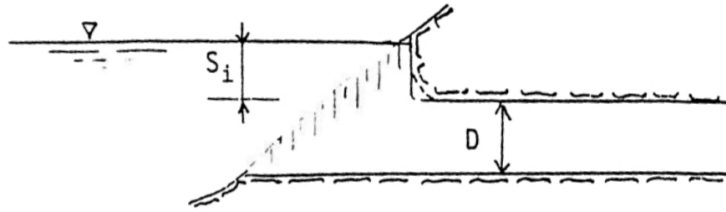


Figure 2.1: Required submerging of intakes(Guttormsen, 1989)

Ice problems

Ice problems can occur either in cold regions or at high altitudes. It is different problems related to ice, but this section will mainly focus on frazil ice and ice clogging, since this is most relevant for flushing. Frazil ice is a slurry kind of ice that has adhesive capacities, that means it easily attaches to trash racks. Frazil ice can be recognized as a state between ice and snow, dissolved in water, and forms when the water is supercooled, i.e. below freezing temperature. Supercooling requires large heat loss and typically occurs when it is open water, the air temperature is below 6 degrees minus and it is clear nights (Daly, 1991).

Ice can be diverted as a floating debris using methods described in section 2.2.2. Frazil ice, on the other hand, is more difficult to divert, and can clog trash racks directly. Air entrainment combined with low temperatures, is a direct source to frazil ice. Streams and waterfalls works as «ice machines» during cold periods. Intake locations close to streams and waterfalls is therefore not recommended, but if it otherwise is a favorable location, one must either divert it past the intake or control if it is enough space to deposit it as ice banks. Removal of frazil ice and clogged ice is, depending on the size or the power plant, is done either manually by raking or mechanical by machines. A third option is to use the principle backflushing, which is described more in detail at section 2.2.4. (Jenssen et al., 2006)

2.2.2 Bypass of floating debris

Proper headwork design requires ways to handle floating debris. Examples of floating debris can be timber, plants, plastic bags or ice. If not floating debris is diverted past the intake structure, it can lead to clogging of the trash rack if the trash rack is not sufficiently submerged. Water vortexes can also drag floating debris down to the trash rack. A free surface overflow must therefore be installed close to the intake.

On larger power plants is it often installed a debris gate, which secure free surface flow for all possible water levels. It is important to design the spillway crest so that no floating debris can get jammed which again can lead to an uncontrolled increase in water level (Lysne et al., 2003). One way to control that floating debris does not affect the intake, is

to install a floating boom. A floating boom as illustrated in Figure 2.2 can be installed to either work as a coarse trash rack, or just to divert the floating past the intake.



Figure 2.2: Floating boom at Nedre Leirfossen power

2.2.3 Conventional cleaning of trash racks

Manual cleaning

For small hydropower projects, the economy of the project does not always allow for the installation of automatic trash rack cleaners. Various solutions have been developed for manually keeping the intake clean of debris, and mostly involve some kind of raking, as illustrated in Figure 2.3.

The need of manual operation do in many cases involve a safety risk, as the operator could have to clean the intake in the dark or during extreme weather. Another downside with manually raking is that it usually requires the power plant to turn down the load during raking. Less load gives less velocity through the trash rack which make the raking easier.

Automatic cleaning

Power plants over a given size will often have installed automatic systems for cleaning the intake area. In many cases, hydraulic rack cleaners remove debris from the trash racks mechanically. It is possible to install sensors that will detect if cleaning is necessary, by logging the head loss over the trash rack. Depending of the expected amount and size of the debris that should be removed, different solutions are in use.

A normal solution for Norwegian small hydropower plants is a wire rope trash rack cleaner as shown in Figure 2.4. A metal frame with teeth fitting in between the bars of the trash rack is then lowered down by a wire, and pulled up using a winch. For wire rope trash

rack cleaners, the trash racks are often tilted slightly from vertical, to increase the normal force from the cleaner on the rack. Intakes for small hydropower projects are normally restricted to shallow intake ponds, but the wire rope trash rack also has the ability to operate on deep water depths.

Another solution is to fix rails or a framework along the intake opening, and maneuver the cleaner using hydraulic pistons, chains or wires. By attaching the cleaner to a fixed rail, it is possible to obtain an efficient cleaning also for vertical trash racks. For projects of larger size, it is normal to consider cleaning by larger cranes, telescope beams, or other hydraulic equipment to provide a flexible and reliable cleaning. Depending on the size of the intake, many trash rack cleaners can be designed to be either stationary or mobile. By installing the base of the trash rack cleaner on rails running perpendicular to the water front, the cleaner can be moved freely sideways.

According to Kåre Natland, the inventor of the trash rack cleaner concept «Sopeleisten», the typical installation cost of a trash rack cleaner would be in the range 150 000 NOK - 250 000 NOK for a width of 3 m (Converstation with Kåre Natland, 2012). For most trash rack cleaners designed for small hydropower, it will not be possible to operate the cleaner if there has been established an ice cover on the intake pond.

2.2.4 Backflushing concept

For small hydropower projects, much of the operating costs is due to head loss and maintenance costs. It is of great interest to design intake constructions which can be cleaned in a simple and safe way, even through challenging conditions. The operational costs and efforts made to reduce these, must be capitalized by increased production.

Backflushing of trash racks is a concept for self-cleaning intake constructions. By installing a flushing gate upstream of the intake, the water flow over the trash rack can be reversed for a short period of time, and the debris will detach from trash rack and be flushed out through a flushing gate. The concept depends on a water volume available for flushing, enough to achieve sufficient water velocity and pressure difference over the trash rack for releasing the debris.

The origin of the idea is from the water hammer that can occur during a rapid shutdown of the turbines in a powerplant. Egil Berge from the power company Eidsdal Kraft AS developed this idea, and patented his concept «Bergedammen». An installed flushing pipe induces a reversed water flow over the trash rack, and transports released debris out to the river downstream. «Bergedammen» has a two chamber solution which makes the power plant able to flush one of the trash racks in one of the chambers while running the turbine with water from the other chamber. See Figure 2.5 for a principle sketch for Bergedammen.



Figure 2.3: Manual raking of trash rack



Figure 2.4: Wire rope trash rack cleaner at Leirfossen

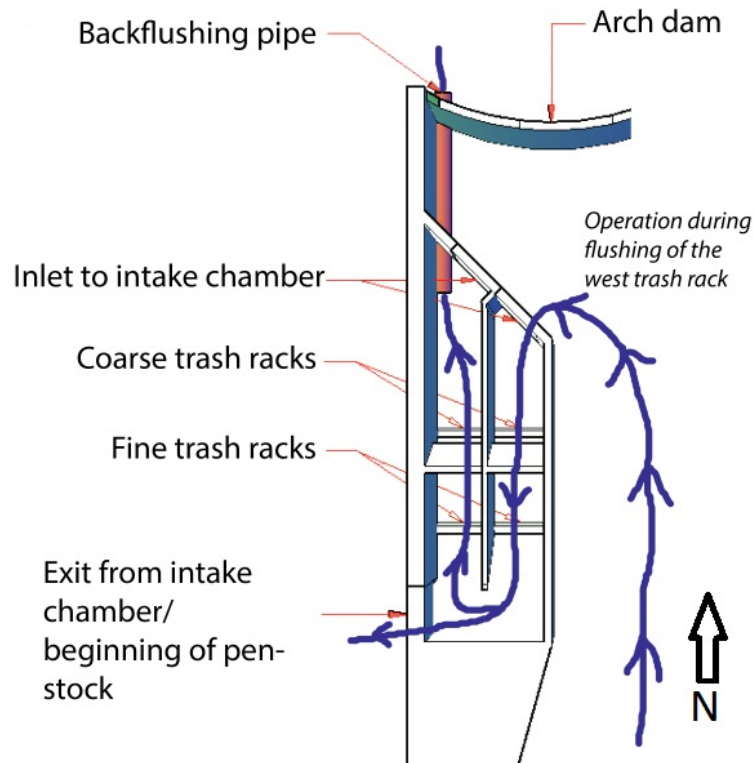


Figure 2.5: A principle sketch of Bergedammen

Similar concepts to «Bergedammen» have also been developed, like «Viddal kraftverk» which is owned by Tussa Energi AS, and «Heidal-inntaket».

The backflushing concept is designed to also work in cold regions, like in Norway during winter time. Ice and frazil ice as described in section 2.2.1, should not be a problem. According to phone calls with Egil Berge, it is possible to flush frazil ice that is attached to the trash rack (Converstation with Egil Berge, 2012).

The existing experiences of using backflushing as a concept for hydropower intakes is limited. It is of interest to investigate the limitations and abilities for the backflushing concept, to develop general design guidelines.

Research done by Nøvik et al. (2011) defines that the forces needed for release of the debris is a result of pressure difference over the trash rack, and also the shear forces due to velocity over the trash rack. Until now, research indicates that the flushing velocity must be at least the same or bigger than the velocity during clogging.

2.2.5 Parameters for backflushing of trash rack

In this report, physical parameters for a successful backflushing of a trash rack has been evaluated. This section will define what physical parameters that have been used.

Forces acting on clogged debris

For a solid body submerged the depth h in water with density ρ_w , the static water pressure acting on the body can be defined as

$$p = \rho \cdot g \cdot H \text{ [Pa]} \quad (2.1)$$

The water pressure can be rewritten as the the pressure from a water column at a given height, as

$$H = \frac{p}{\rho \cdot g} \text{ [mWC]} \quad (2.2)$$

hence, 1 mWC = 9810 Pa.

With no flow velocity, the pressure on both sides of a trash rack will equilibize, and there will be no net horizontal force acting on debris clogged to the trash rack.

When there is movement of the water over the trash rack, there will be formed a singular head loss over the rack, increasing with the square of the water velocity

$$\Delta p = f(v^2) \quad (2.3)$$

For material clogged on the trash rack, the pressure difference will result in a net force acting in the flow direction, with the magnitude

$$F_h = \Delta p \cdot A \quad (2.4)$$

where A is the projected area in the flow direction. As the net force in the flow direction will depend on the pressure difference, it should not vary with the water depth.

Adhesion of clogged debris

Adhesion is in this report defined to represent the pressure needed to make the clogged debris detach from the trash rack by mechanical pulling.

2.3 Procedure

As a part of the author's project work fall semester 2011, «Inntak for småkraftverk med tilbakespylingsmulighet» (only available in Norwegian), a backflushing test tank was constructed. The tank was used to investigate the mechanisms for clogging of trashracks, and to evaluate the water velocities needed for sufficient flushing efficiency (Rettedal and Nielsen, 2012). As concluded in the project report, it was raised question on whether the clogging obtained by the test series were realistic to conditions in nature. The debris clogged in the tests did not seem to stick to the trashrack in the same way as observed in the nature, and it was not possible to draw conclusions on physical parameters for flushing of the prototypes. In this master's thesis the model study was developed.

2.3.1 Scope of model study

In this master's thesis, the testing tank used in the projectwork by Rettedal and Nielsen (2012) has been modified and the methodology changed to obtain results that can be compared with conditions found in nature. A measuring device has been developed to investigate how the debris stick to the trashrack in the laboratory, compared to conditions in nature. Because of the climatic conditions during the report period, extensive field-measurements has not been applicable. The intake structures connected with the research project have not reported of clogging of debris on their trash racks. One field trip was conducted to Bergedammen in Eidsdal, where some clogging had occurred, and some initial observations were done.

The test series conducted in the report period will serve as a basis of comparison for field-testing in further studies, ideally during autumn-floods with high transport of debris.

2.3.2 Model concept

The purpose of the tests in the laboratory during the project period has been to measure the physical conditions around a trash rack during flushing, and to evaluate the water velocity, pressure difference and discharge over the rack needed for a sufficient flushing efficiency. The goal has been to provide some initial physical values that developers can use when considering implementation of the backflushing concept in their projects.

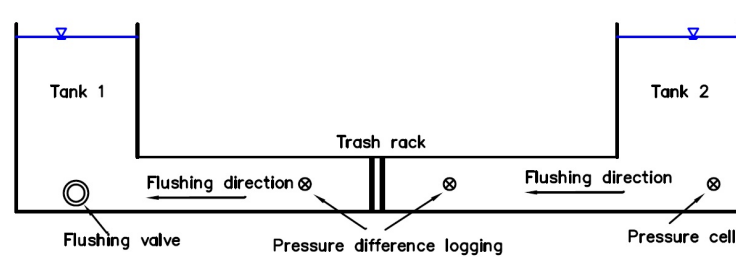


Figure 2.6: Concept of the backflushing testing tank

The backflushing testing tank is described in Figure 2.6. The system consists of two water tanks, connected by a 6 m long 0.35 m x 0.35 m rectangular channel. In the channel, it is possible to place a section of a full scale trash rack. A duplicate of a section of the trash rack at Bergedammen was used, with a bar thickness of 10 mm and space opening of 20 mm. The rack section was manually clogged with debris, consisting of leaves, grass and moss. The manual clogging of the trash rack is described in more detail later in this chapter.

After clogging, the rack would be installed into the channel with the clogged material facing towards the tank 1, and the channel-walls were sealed off. The tanks would slowly be filled up from the water supply in tank 2, to prepare for flushing of the trash rack.

To simulate flushing, a manually operated valve was installed in tank 1, By opening the valve, a water flow would occur from tank 2 to tank 1 in the channel, and provide backflushing of the installed trashrack. By gradually opening the valve at a constant rate the water velocity through the rack was gradually increased, to determine the needed water velocities needed for sufficient flushing efficiency. Pressure cells installed directly in front and after the trash rack in the channel were used to monitor the pressure development over the rack during flushing.

Defining trash rack adhesion

A method for measuring the adhesion, the resistance of the clogged material on a trash rack, has been developed. Adhesion is in this report defined to represent the pressure needed to make the clogged debris detach from the trash rack by mechanical pulling.

Figure 2.7 is showing the Pulling Equipment for Evaluation of Trash Rack Adhesion (PEETRA), that has been designed and constructed by the authors together with Geir Tesaker at NTNU. The concept of PEETRA is to measure the required force it takes to pull out a standardized area of clogged material from a trash rack. Doing standardized tests with PEETRA, adhesion values obtained by manual clogging of the trash racks in the laboratory could be compared with observed values measured in the field. Hence, PEETRA was used to calibrate the degree of clogging in the laboratory, to represent the conditions in nature.



Figure 2.7: Pulling Equipment for Evaluation of Trash Rack Adhesion («PEETRA»)

PEETRA consists of four aluminum members, separated to fit in between the bars of the trash rack in use. By leading the bars carefully in behind the clogged material on the trashrack, it is possible to pull the clogged material off the rack in a controlled way, measuring the needed force with a spring scale. The spring scale operates with an accuracy down to one Newton. A peak function was used, so that the highest registered value was stored in the display, and noted down by the operator after each pull.

The trash rack adhesion was defined as the measured peak pulling force divided by the effective area of the trash rack covered by the four pulling teeth. Testing for trash racks with a space opening of 0.02 m, and a height of the clogged debris of 0.17 m, the effective area was defined as

$$A_e = 4 \cdot 0.02 \cdot 0.17 = 0.0136m^2 \quad (2.5)$$

Hence, for each measurement of pulling force, the trash rack adhesion would be defined as:

$$\sigma_r = \frac{F}{A_e} = \frac{F}{0.0136m^2} [Pa] \quad (2.6)$$

$$\sigma_r = \frac{F}{0.0136} [Pa] \cdot \frac{1}{9810} [mWC/Pa] = \frac{F}{133.4} [mWC] \quad (2.7)$$

The trash rack adhesion was calculated as pressure using the unit *meter water column* (mWC), to be comparable with the results of pressure difference over the trash rack during flushing tests. The trash rack adhesion should not be confused with the head loss over the trash rack during normal operation of a power plant, it is a measure of the required pressure for removing the debris using the standardized PEETRA concept.

To provide stability and comparable results, a rectangular steel profile can be attached to the trash rack using bolts with anchor ends. Two parallel steel pipe sections welded on to the profile serves as rails for the four aluminum pulling teeth, which are mounted on a fixed platform sliding on the rails. By mounting the pulling device on the fixed frame, the measurements can be conducted stabilized and standardized, even at challenging locations like down in the intake chamber of a hydropower plant.

Calibration of adhesion values in the laboratory

In the laboratory, test series were conducted to predict the adhesion on the trash rack before it was installed into the flushing channel. Debris was manually clogged to a section of the rack, and the appurtenant adhesion was measured with the PEETRA. The manual clogging and adhesion measurements would be repeated at a minimum of four times, trying to use the same amount of force for each clogging session.

After the test series, a 70 % prediction interval was calculated to predict the adhesion of the final clogging. To obtain reliable results, it was important the last clogging of the trash rack would be clogged in the exact same way as for the test series, and the operator would have to rely on his «calibrated fingers». After the trash rack was clogged for the last time, it was mounted into the channel, and the tank would be prepared for flushing.

The results from the test series are summarized in Table 2.1. Complete data series from all the test series is enclosed in Appendix B.

For each test series, the measurements in the sample were treated as continuous, random variables. The estimators for the mean adhesion $\bar{\sigma}_r$ and and the standard deviation s for the sample was calculated using:

$$\bar{\sigma}_r = \frac{\sum \sigma_{r,i}}{n} \quad (2.8)$$

$$s^2 = \frac{\sum (\bar{\sigma}_r - \sigma_r)^2}{n} \quad (2.9)$$

The prediction interval was calculated assuming a gaussian (normal) distribution, to estimate the adhesion range of the next measurement. As the samples only include estimators for the mean and the standard deviation, the prediction interval was defined as:

$$P(\bar{\sigma}_r - T_a s_n \sqrt{1 + \frac{1}{n}} \leq \sigma_{r_{n+1}} \leq \bar{\sigma}_r + T_a s_n \sqrt{1 + \frac{1}{n}}) = 0.70 \quad (2.10)$$

where T_a is the $\frac{(1-0.7)}{2}$ th percentile of the Student's t-distribution with n-1 degrees of freedom.

The upper and lower limits of the prediction interval would then be calculated as:

$$PI_{70} = [\bar{\sigma}_r + T_a s \sqrt{(1 + \frac{1}{n})}, \bar{\sigma}_r - T_a s \sqrt{(1 + \frac{1}{n})}] \quad (2.11)$$

Table 2.1: Adhesion measurements with 70 % pred.int.

Test number	Number of pulls	$\bar{\sigma}_a$ [mWC]	PI_{70} [mWC]
1	4	0.10	± 0.02
2	4	0.23	± 0.03
3	6	0.17	± 0.08
4	4	0.20	± 0.04
5	4	0.13	± 0.03
6	4	0.25	± 0.05
7	4	0.11	± 0.01
8	5	0.18	± 0.05
9	5	0.21	± 0.03

The values used for estimating the adhesion of the debris on the trash rack was adjusted for the friction between the sliding platform and the rails. The pulling device is designed to minimize the transferred momentum between the platform and the sliding rails, and it was assumed that the background friction would not increase with an increasing axial load. A test series was conducted in the laboratory to measure this background friction, attaching the pulling teeth with a string through a pulley, and to different counterweights. By comparing the actual load of the counterweights with the measured pulling force, a relationship was derived showing that the measured result laid approximately 5 N above the actual counterweight regardless of the load. As the conditions in the tank during clogging and pulling of the trash rack made it difficult to keep the rails and the platform perfectly clean and greased at all times, it was decided to define the net pulling force, F_{net} as:

$$F_{net} = F_{gross} - F_{friction}$$

and

$$\sigma_r = \frac{F_{net}}{A_e}$$

where F_{gross} is the value logged by the spring scale for each test, and $F_{friction}$ is the pulling force required to slide the mounted platform out of the trash rack without any debris attached on the rack. After each measurement, the background friction would be noted down by doing the same pull without debris on the trash rack.

Backflushing efficiency

For the flushing tests in the laboratory, the trap efficiency was defined as

$$\eta = \frac{A_{clean}}{A_{tot}}$$

where A_{clean} is the area of the trash rack cleaned for clogged material, and A_{tot} is the total area between the bars of the trash rack.

2.3.3 Instrumentation and interpretation

In the tank, three type S-10 pressure transmitters from WIKA were installed to monitor the pressure and velocity development during flushing. The pressure transmitters have a measuring range from 0 to 0.4 bar, and deliver a 2-wire output signal in the range 4-20 mA. The pressure data was recorded at a frequency of 100 Hz, using a the U2355A USB Data Acquisition Module from Agilent.

Pressure development

The two pressure cells monitoring the pressure difference over the trash rack were used to investigate when the debris would loosen from the trash rack during flushing. The pressure difference over the trash rack, was defined as

$$\Delta p = p_{upstream} - p_{downstream} \quad (2.12)$$

where $p_{upstream}$ and $p_{downstream}$ are the readings from the pressure cells installed in the channel immediately on the up- and downstream side of the trashrack. A sampling frequency of 100 Hz was used. A 15 point moving average trendline of the Δp data series was calculated, to avoid incorrect peak values due to turbulence and precision errors. The pressure development during test 1 is shown in Figure 2.8.

Velocity development

To monitor the velocity in the flushing channel, velocity series was derived from a pressure cell located in tank 2. By applying the continuity equation, it can be shown that the rate of decline of the water level in tank 2 must be directly connected with the velocity in the flushing channel. It was assumed that the pressure in tank 2 would directly represent the

water level in the tank, neglecting pressure differences due to acceleration of the water body.

The water surface development in tank 2 was logged by the pressure cell in tank 2 during each flushing. In order to evaluate the rate of decline in the tank, a polynomial curve of 15th degree was fitted to the data for each test with the least square method, using the software Matlab from Mathworks Inc. Using a polynomial of a high degree, the fitted curve would catch minor changes in the pressure development. Polynomials of lower degree turned out to miss out on details in the curvature.

Figure 2.9 shows the water surface development in the right tank during flushing test no.1, and the polynomial function fitted to the dataset. By using Matlab, the curve was differentiated and transformed to velocity by use of the formula

$$V(t) = -\frac{d}{dt}P(t) \cdot \frac{A_{tank}}{A_{channel}} \quad (2.13)$$

The derived velocity development in the channel for test no. 1 is shown in Figure 2.10 .

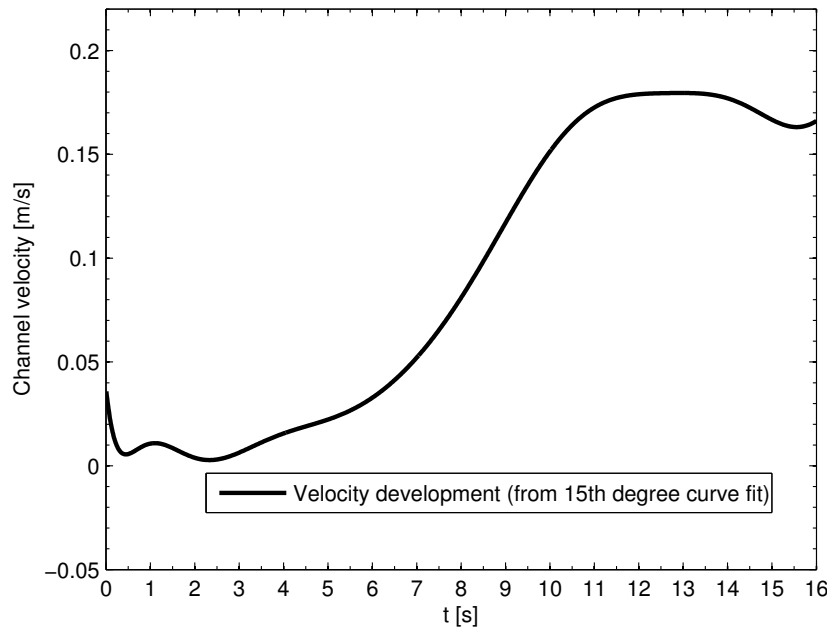


Figure 2.10: Velocity development, test 1

Evaluation of accuracy and uncertainties of the estimation of the velocity is described in more detail in section 2.5. It was considered placing an Acoustic Doppler Velocimeter (ADV) in the channel to monitor the velocity, but due to the amount of debris in the channel, this was not found applicable.

To achieve similar velocity developments for all the tests, the opening rate of the flushing valve was standardized. In lack of a mechanical opening system, the valve had to be

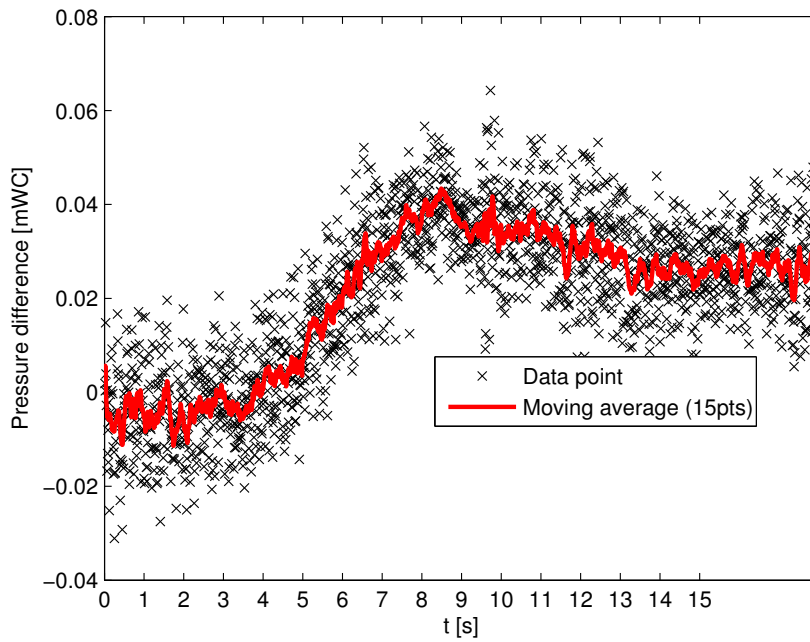


Figure 2.8: Pressure difference development over trashrack, test 1

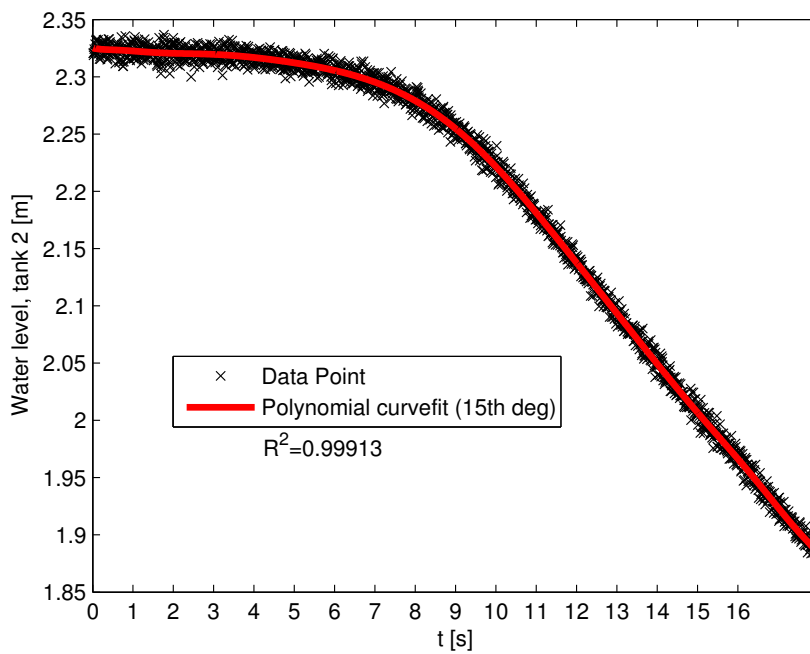


Figure 2.9: Tank 2 water level development, test 1

opened manually. By mounting a pointer to the valve bar and marking up ticks at a fixed interval on a background steel plate, the opening rate of the valve could be controlled by use of a metronome. A metronome keeps a given rhythm, and marks every beat at a fixed rate. During the opening of the valve, the operator would wear a headset with a metronome set to 60 beats pr. minute, and move the valve bar at a smooth rate, covering one mark on the scale per beat. In the figures of pressure difference and channel velocity during flushing, $t = 0$ represents the beginning of the opening of the valve. The valve was opened at a constant rate until maximum opening was reached at $t = 14$. At maximum opening of the valve, the velocity in the channel would start to decrease as the water level in the tank would sink.

Defining the release of the clogged debris

For all the tests, the peak of the moving average trendline of the pressure difference was chosen to define the event of the debris letting go of the trashrack. By evaluating the velocity in the channel at the same time as the peak in pressure difference occurs, a value of the *release velocity* was established. In Figure 2.11, both the pressure difference and channel velocity during flushing no.1 are plotted. The thin dotted line marks the reading of the maximum pressure difference, and the corresponding channel velocity.

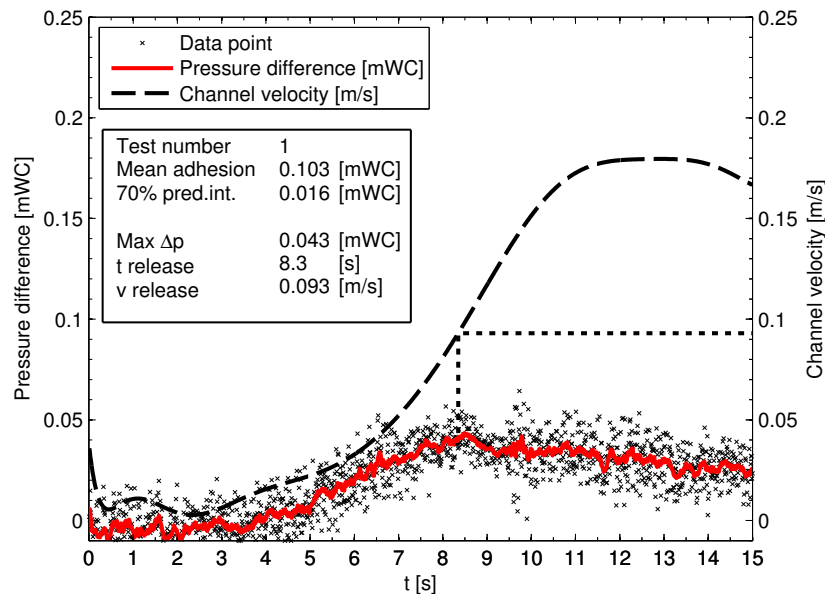


Figure 2.11: Estimation of release velocity .

Manual evaluation

All flushing tests were recorded on camera, and it is possible to manually evaluate at what time the debris detach from the trash rack. In the enclosed CD, movies documenting each

flushing test can be found. For the filming, the camera was turned on 40 s before the opening of the flushing valve, hence 40 s should be added to the t-value found in the pressure diagrams for comparison.

After each flushing test, some debris would remain stuck on the trashrack. Photos were taken of the resulting trashracks to document the flushing efficiency. Photos for each test can be found in Appendix D.

2.3.4 Field measurements of trash rack adhesion

To have a basis of comparison for backflushing experiments in the test tank, it was necessary to evaluate the trash rack adhesion for an actual trash rack at a power plant. It was scheduled to visit several power plants, but as spring time is a season with little debris, Bergedammen in Eisdal was the only power plant with clogging that could be visited. Figure 2.12 is showing the PEETRA system set up in the field.



Figure 2.12: Adhesion measurements at the Bergedammen intake

To mount PEETRA at the trash rack, an area would have to be cleared for clogged material before fixing the anchor bolts of the metal frame. As the measurements only test the adhesion of four parallel bar openings, it was discussed whether the debris in the four bars should be cut loose from the surrounding debris. It was observed that the material would stick to the debris surrounding the bar openings. It was attempted cutting free the material covering the four bar openings of interest, but it was difficult to avoid disturbing the material for the test.

2.4 Results

The goal of the test experiments was to find physical parameters for successful flushing of trash racks, to be used for the design of hydropower intakes using the backflushing concept. Measurements of the pressure difference and water velocity over the trashrack at the event of release of the clogged material was done.

2.4.1 Field measurements of trash rack adhesion

Using PEETRA as described in section 2.3.2, measurements of the adhesion of clogged debris on the trash rack of the Bergedammen intake in Eidsdal were done. The adhesion values obtained on Bergedammen are meant as a reference to the adhesion values used in the laboratory. The results from all the adhesion tests can be found in Appendix A, in Table A.1.

The mean trash rack adhesion measured at Bergedammen was found to be: $\bar{\sigma}_r = 0.10$ mWC.

As the conditions in the report period did not allow for measurements of mutiple projects during floods carrying large amounts of debris, the results are only evaluated as initial observations.

2.4.2 Release pressure difference

The development of pressure difference over the trashracks for all the tests in the laboratory is presented in Figure 2.13. The estimated adhesion of the clogged debris is represented in the figure by the color of the lines, as defined in the colorbar on the right side. It can be seen that the trash racks subjected for higher clogging adhesion requires a higher pressure difference before the debris detaches.

Pressure difference development during flushing tests

The peak of the curve describing the development of pressure difference over the trash rack during flushing was defined to represent the event of the debris detaching from the trash rack. The value of the pressure difference at the time of release was defined as the *release pressure difference*. Figure 2.14 shows the registered release pressure differences plotted against the estimated adhesion of the debris on the trash rack.

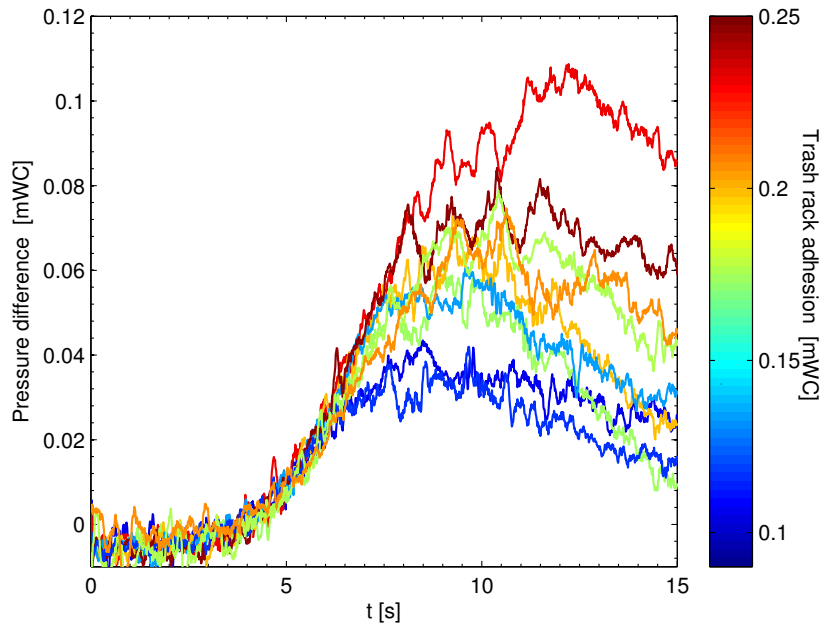


Figure 2.13: Pressure difference development during flushing tests

2.4.3 Release velocity

After defining the event of release of the clogged debris, the corresponding channel water velocity could be evaluated. The estimated velocity in the chamber at the time of the maximum pressure difference over the trash rack was defined as the *release velocity*.

Figure 2.15 shows release velocity for each test, plotted against the estimated adhesion of the clogged trash racks. The data is gathered in the range of 0.1 - 0.14m/s, with some tests giving remarkable higher results. The adhesion of the Bergedammen intake in Eidsdal was measured to be in the lower range of the values tested for in the laboratory, with a mean value of $\sigma_r = 0.10$ mWC. From Figure 2.15, the required release velocity for the Bergedammen material can be expected to be approximately 0.1 m/s . The backflushing concept of the prototype in Bergedammen is found to use a flushing velocity of 0.15 and 0.26 m/s for the two chambers during flushing, with sufficient flushing efficiency. (Nøvik et al., 2011)

The results for all the 9 test series are presented in Table 2.2. The detailed pressure- and velocity developments over the trashrack during the different flushing tests can be found in Appendix C.

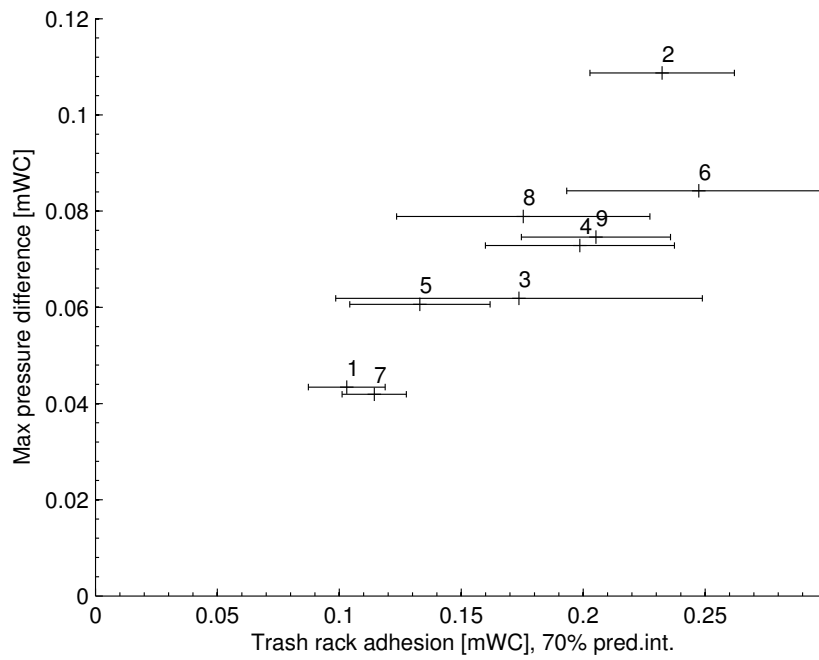


Figure 2.14: Pressure difference over trash rack at time of release

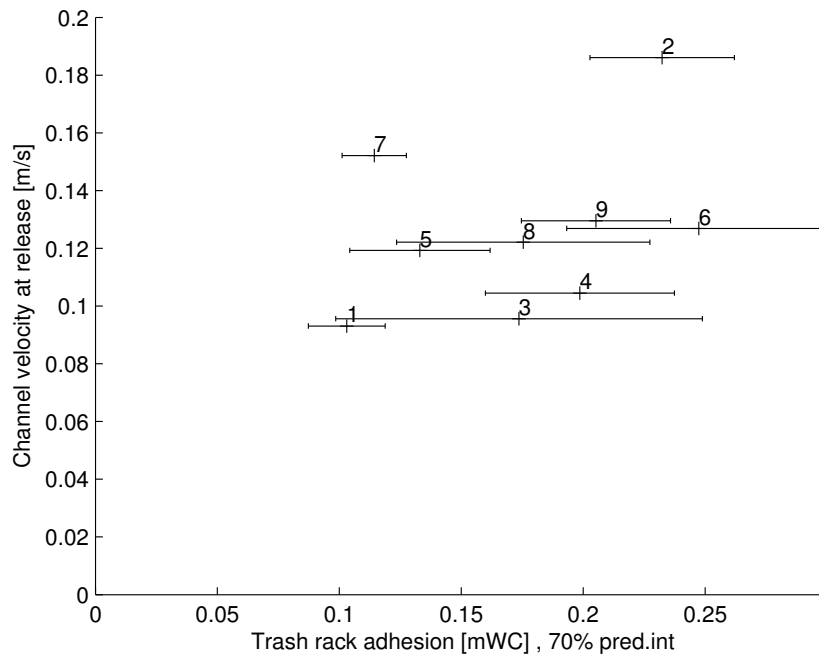


Figure 2.15: Required release velocity for clogged trash racks

Table 2.2: Backflushing test results

Test number	Number of pulls	Adhesion [mWC]	Std. error [mWC]	Δp [mWC]	$t_{release}$ [s]	$v_{release}$ [m/s]	η
1	4	0.10	0.02	0.04	8.35	0.09	35 %
2	4	0.23	0.03	0.11	12.08	0.19	25 %
3	6	0.17	0.08	0.06	8.84	0.10	48 %
4	4	0.20	0.04	0.07	9.11	0.10	38 %
5	4	0.13	0.03	0.06	9.50	0.12	-
6	4	0.25	0.05	0.08	10.25	0.13	28 %
7	4	0.11	0.01	0.04	9.53	0.15	46 %
8	5	0.18	0.05	0.08	10.28	0.12	38 %
9	4	0.21	0.03	0.07	10.49	0.13	37 %

2.5 Discussion

Results from the physical model study indicated a debris release velocity in the range of 0.1 m/s to 0.14 m/s, and a that the debris detaches with a pressure difference over the trash rack of 0.04 mWC - 0.12 mWC. This section aims to evaluate the results and explain what assumptions the results are based on, to see if the results are trustworthy and how the results from the laboratory correlates with prototypes.

Pressure difference development during flushing

As can be seen from Figure 2.14, the peak value of the pressure difference over the trash rack during flushing is clearly depending of the level of the adhesion of the clogged debris. The same can be observed from Figure 2.13.

As can be seen from the pressure development curves collected in Appendix C, many of the tests tend to have multiple consecutive peaks in the pressure difference development, approximately in the same value range. Figure 2.16 displays test number 6, which shows evident peaks after 8, 9, 10.5 and 11.5 seconds. The pressure difference for all the peaks are in the range of 0.07 mWC. The reader is urged to compare the figure with the video of flushing test number 6, as can be found enclosed with the report. The peaks in Figure 2.16 correspond well with the distinct individual detachments of material that can be seen in the video.

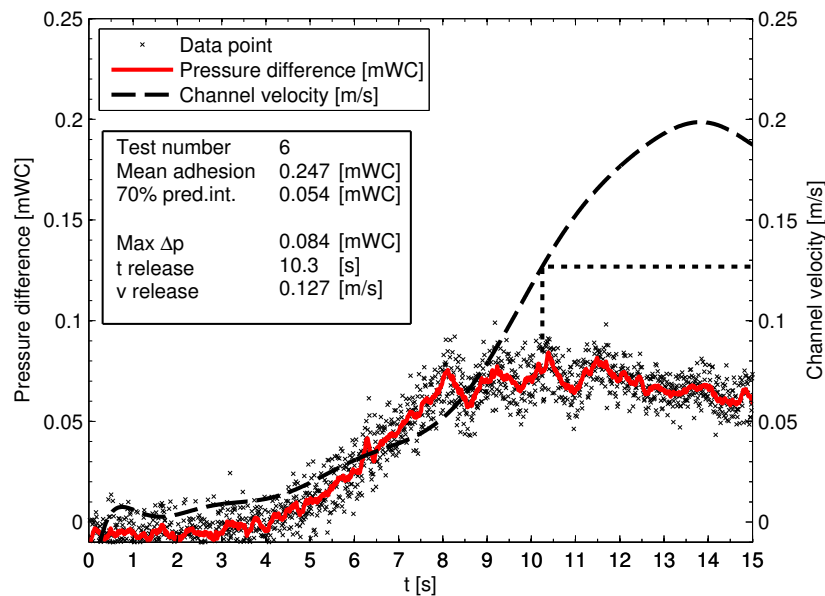


Figure 2.16: Pressure difference development, test 6

Based on the observations of multiple pressure difference peaks, it can be assumed that there is a limit value of pressure difference that will provide detachment of the clogged

material. As parts of the debris is removed, more area is available for the water flow, and the pressure difference is dropping. As the velocity in the channel increases, the pressure difference builds up to the same critical level before new areas are cleaned.

The peak value of the pressure difference seems to be the controlling parameter for the efficiency of the flushing. As a larger area of the trash rack is opened, it continuously requires a higher water velocity to set up the same pressure difference. Hence, initiating the flushing by a rapid pressure drop in front of the trash rack should lead to a higher flushing efficiency.

Evaluating the results afterwards of the tests, it should have been run test series with a shorter opening time of the flushing gate for comparison. One argument for the controlled, slow opening was to avoid the pressure oscillations that occur during rapid acceleration of the water body. Pressure oscillations turned out to be a problem during the author's work in the model during 2011 (Rettedal and Nielsen, 2012).

If it can be assumed that there is a constant pressure difference limiting the detachment of clogged material, the effect of internal distribution of adhesion over the trash rack should be considered. If there are great variances of the adhesion over the rack, it will be difficult to obtain simultaneous detachment of debris. The areas with loosely clogged material will be flushed early, making it more difficult to flush out the rest. Hence, evenly distributed flow conditions over the rack during operation may be of great importance to the ability of flushing.

Backflushing efficiency

The backflushing efficiency of the tests in the laboratory varies from 25 % to 46 %. An example of a trash rack after flushing is shown in Figure 2.17. Pictures from every flushing is enclosed in Appendix D. Is this efficient backflushing? As discussed in the previous paragraph, it was concluded that an evenly flow condition over the trash rack, i.e. clogging, is important for backflushing. In the test series conducted, the trash rack was clogged by using «calibrated fingers». The good correlation between maximum pressure difference and adhesion in Figure 2.14 proves that the method of «calibrated fingers» works well. On the other side, it does not necessarily indicate an equal internal distribution of adhesion. Based on uneven adhesion distribution and slow flushing gate opening, it is believed that this is a contributing factor to the low flushing efficiency observed in the laboratory.



Figure 2.17: Trash rack after flushing test 1

What was observed at Bergedammen, was that the debris had adhesion to the surrounding debris. The surrounding adhesion effect is also observed by Egil Berge during flushing, i.e. that debris detaches collectively. A similar effect is not reproduced in the experiments of this master's thesis, which means that in the test tank is every clogged space opening independent, and a «domino effect» is not present.

Uncertainties

Dealing with such a chaotic and randomized phenomenon as floating debris clogged to a trash rack, there will always be a number of uncontrollable parameters appearing. Hence, a high degree of uncertainty should be expected for the experiment series.

Clogging of the trash rack

To be able to run flushing tests on trash racks with differing degree of clogging, the manual clogging concept and the appurtenant adhesion measurements with PEETRA was used. It was unrealistic to believe that one should be able to fully represent all the physical parameters of a naturally clogged rack, so the defined value of adhesion was used.

The 70 % prediction intervals indicating the range of the expected adhesion for the tests in Figure 2.14 and Figure 2.15, are revealing that it was difficult to obtain the same adhesion value for every round of manual test clogging. However, the clear dependency between maximum pressure difference and expected adhesion described in Figure 2.14 is indicating that the concept is within an acceptable range of uncertainty.

To obtain standardized flushing conditions, the flushing valve was always attempted opened in the exact same way for all the tests. The tank would be filled up to a fixed level before the tests, so the flushing valve would release the same discharge every time. Due to leakages in the tank, it was attempted to use the same amount of time from the end of the filling until the flushing started.

Opening of flushing valve

The operation of the valve was standardized by the use of a metronome, a device that keeps a given rhythm. Although the two different operators used the same metronome and the same scale for controlling the valve opening rate, differences in the way of opening the valve was expected.

To document the uncertainty of the opening of the valve, 6 test openings were conducted in the channel without any trash rack installed. Figure 2.18 is showing the variations in the velocity distributions for the 6 tests, and the results are considered not to influence the presented results in a major way.

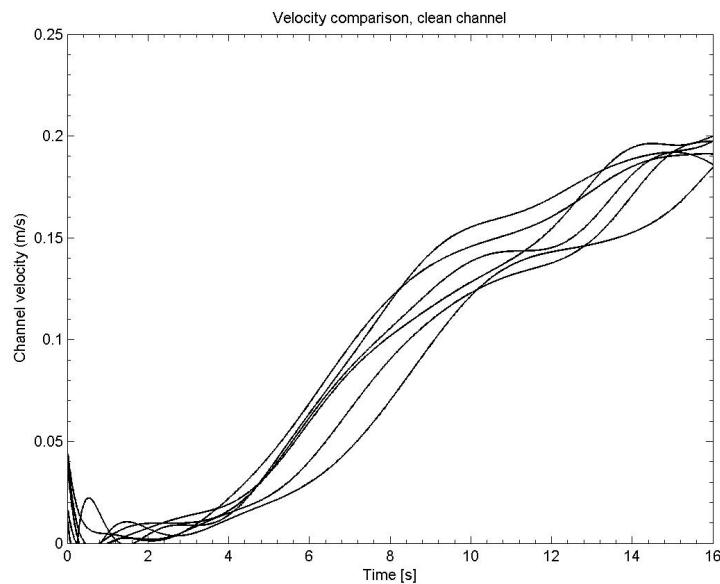


Figure 2.18: Velocity development comparison, empty channel

Release velocity

The parameter *release velocity* was defined as the velocity in the channel at the time of maximum pressure difference over the trash rack, and is presented in Figure 2.15.

As the velocity in the channel during flushing increases during the test, the value of release velocity is depending strongly on what time the maximum pressure difference is recorded. As described earlier, many of the tests revealed multiple peaks in the pressure difference development during flushing. Where multiple peaks were within the same range, small differences in the peak value could strongly affect the resulting release velocity, making the release velocity sensitive to the registration of the highest peak.

The sensitivity of the release velocity can clearly be observed from test number 4,6 and 9, found in Appendix C.

Velocity measurements

The velocity development in the channel over the trash rack was estimated using measurements from a pressure cell located in tank 2 in Figure 2.6. As described in section 2.3.3, velocity functions were differentiated from the measured water-level developments, by using the principle of continuity between the channel and the water surface in tank 2.

More accurate and reliable velocity measurements would be possible by installing an ADV. The amount of debris in the channel was one argument for the use of the pressure cell, but seen afterwards it seems like it would have been fully possible to use a ADV. However, the good fit of the 15th degree polynomial curve representing the pressure development in tank 2 during flushing, as shown in Figure 2.9, is indicating that the derived velocity series should be within an acceptable range of uncertainty.

Further work

For further research on the topic of backflushing of trash racks, a survey of the appearing adhesion at trash racks in the field is recommended. In this report, trash racks with different degrees of clogging has been analyzed, categorized by the estimated adhesion with the PEETRA system. Only brief measurements in the field have been conducted, with one trip to Bergedammen in Eidsdal. For a further study, the following parameters should be considered evaluated at multiple intake trash racks in the field:

- Adhesion level
- Internal distribution of adhesion
- Type of debris

It is still not known what causes the debris to clog differently. Effects that can be investigated both in laboratory and/or at field visits:

- Duration of clogging and/or frequency of flushing
- Biological effects/types of debris

Further work in the laboratory that is recommended is:

- Rapid gate opening
- Larger model to dampen pressure oscillations with rapid gate opening
- Installation of an ADV

Chapter 3

Flushing of sediments

3.1 Introduction

Worldwide, heavy sediment transport in rivers often represent a major challenge when designing hydropower systems. Sediment induced wear on turbines and other mechanical equipment can drastically reduce the lifetime of a project's components, and result in comprehensive and expensive overhauling programs.

A hydropower project in a sediment-carrying river will have to cope with a series of additional challenges compared with a project where the sediment yield is low. The general performance criteria for hydropower headworks mentioned in chapter 1 is proposed to ensure ability of reliable operation also during challenging conditions like heavy sediment transport.

An important part of the headworks design in sediment-carrying rivers is to ensure that no uncontrolled accumulation of sediments will take place in front of the intake or in the settling basin. To maintain control of the sediment accumulation, different concepts of flushing have been developed.

This chapter has assessed strategies for flushing of settling basins, gravel traps and the area in front of the intake. Flushing of reservoirs is defined to lie without the scope of this project.

3.2 Theory

This section aims to collect theory regarding the extensive topic of headwork design in sediment-carrying rivers, and the relation to flushing of sediments.

3.2.1 Sediment transport

To be able to predict the behavior of sediment-carrying rivers, it is of major importance to understand the principles of sediment transport and particle movement. Sediment movement is a complex topic, and has been the subject of intense research. However, the complexity and uncertainties connected with sediment transport call for ways of simplifying problems that occur, in order to be able to apply sound judgement. In this section, simple approaches to estimate the behaviour of sediments are presented. For more detailed approaches, the reader is recommended to seek special literature like Vanoni (1975), or (Goldman et al., 1986).

Initial movement

A single object lying exposed in a moving fluid will be affected of drag and lift forces. Simplified, the drag force will be a function of the object's shape and projected area, and the fluid's density and velocity.

$$F_d = C_d \cdot A \cdot \rho \frac{v^2}{2} \quad (3.1)$$

A lift force will also occur, as a result of differences in pressure due to varying velocities around the object.

$$F_L = C_L \cdot A \cdot \rho \frac{v^2}{2} \quad (3.2)$$

Constants for C_d and C_L can be found in NVE (2010) or more detailed in Hoerner (1965). Calculating drag, lift and gravitational forces, the stability of a single object can be evaluated for tipping, lifting and sliding.

A river bed will normally consist a variety of grains sizes, and the particles will all stabilize each other. Hence, when evaluating whether scour or transportation will occur in deposited material, it is natural to look at the shear stress acting on the bottom, and the shear resistance of the bed material in general. When the shear stress on the bottom exceeds the shear resistance in the bed material, movement will occur.

There are various ways of evaluating shear stress and resistance of the bed material. It can be shown that the shear stress at the bottom is a function of the slope of the energy line and the hydraulic radius.

For a stationary flow with no acceleration, the shear stress acting on a element of the water body must equalize the gravitational forces working in the flow direction. Hence, the following relationship can be shown:

$$\tau_o \cdot P \cdot dx = g \cdot A \cdot \rho \cdot dx \cdot I \quad (3.3)$$

$$\tau = g \cdot \rho \cdot R \cdot I \quad (3.4)$$

where R is the hydraulic radius and I is the slope. A popular approach for evaluating the shear resistance in the bed material is presented by Shield, as described in Vassdragshåndboken. Figure shows Shield's diagram (Shields, 1936). In the diagram, the critical Shields number, C_s , is evaluated against the Reynold's number in the boundary area. For most natural river beds, a C_s value of 0.05 could be assumed.

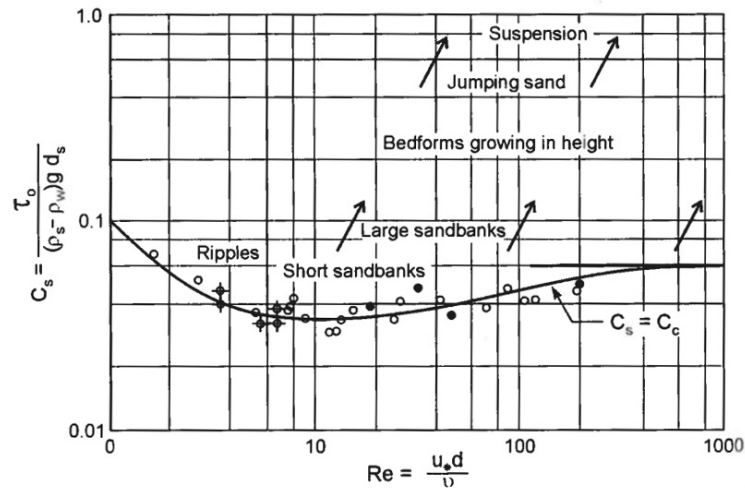


Figure 3.1: Shield's diagram

The shear velocity v_* is defined as:

$$\tau_0 = v_*^2 \cdot \rho_v \quad (3.5)$$

Combining the evaluation of shear stress at the bottom and the shear capacity by Shield's number, the following expression for stable grain size can be found:

$$d_c = \frac{\rho_s}{\rho_s - \rho_v} \cdot \frac{R \cdot I_e}{C_s} \quad (3.6)$$

A more simplified approach to evaluate the stability of a river bed is to look at the mean velocity in the flow. As the flow velocity in the boundary layer directly above the bed is difficult and complicated to measure, the mean velocity method is a popular and quite

simple approach. However, this is not an accurate way of assuming the conditions at the river bed, so this method should be used with care.

Hjulstrøm's diagram, presented in *Vassdragshåndboken* (NVE, 2010), describes three peculiar scenarios, sedimentation, transportation, and erosion. The critical flow velocities separating the three scenarios are marked in the diagram for different sediment diameters. At the sedimentation state, the velocity is low enough for sediments in the water body to settle, and not be transported further in the river. At the transportation state, the velocity will transport the particles in motion, but will not erode on particles of the same size that are already settled. At the erosion state, the already settled particles will be eroded and transported with the water.

Material transport

Sediments occur as a result of different weathering processes on the earth, and sediment transport is a key geomorphological mechanism. Erosion, transportation and deposition of sediments have created most of the natural terrain features in our nature. Hence, as well as a stream of water, a river should also be considered as a stream of sediments.

An alluvial river will act to conserve a balance between sediment load, sediment size, water discharge and river slope. As changes are applied to a river in terms of changes to the water discharge or sediment load, nature will over time balance the equilibrium by gradually adjusting the river slope and/or the sediment size at the bed.

The upstream catchment geological conditions will in many cases determine the sediment load in rivers. Sediment yield is a term used to describe the amount of sediments that can be expected to be generated in a catchment, and as described in Figure 3.2 (Lvovich et al., 1991), it can vary greatly over different regions in the world.

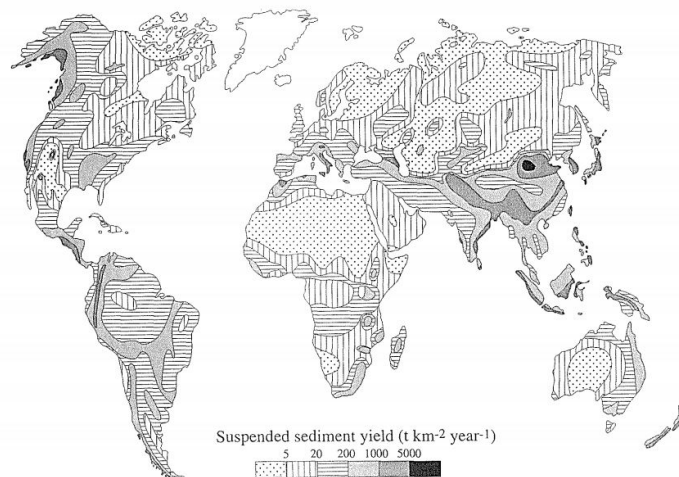


Figure 3.2: Sediment yield of the world

The sediment transport can normally be divided into two fractions, bed load and suspended load. The bed load consists of sediment grains sliding, rolling or jumping along the lower 5 % of the river depth. The suspended load is the portion of sediments is transported freely in the water body. At low flows, the bed load will be the major contributor to the sediment load, but as the flow gets higher and more turbulent, the suspended load be increasingly dominant.

The sediment concentration over the depth will also vary, and the distribution can be of great interest for designing intakes for hydropower plants. Rouse (1961) have presented the diagram shown in Figure , and displays the concentration distribution depending on the variable z . Here:

$$z = \frac{w}{B \cdot k \cdot u_*} \tag{3.7}$$

where

B is a shape factor, 1.0 for fine particles

k is a fluid factor, 0.4 for clear fluids

u_* is the shear velocity, as describe earlier

w is the fall velocity.

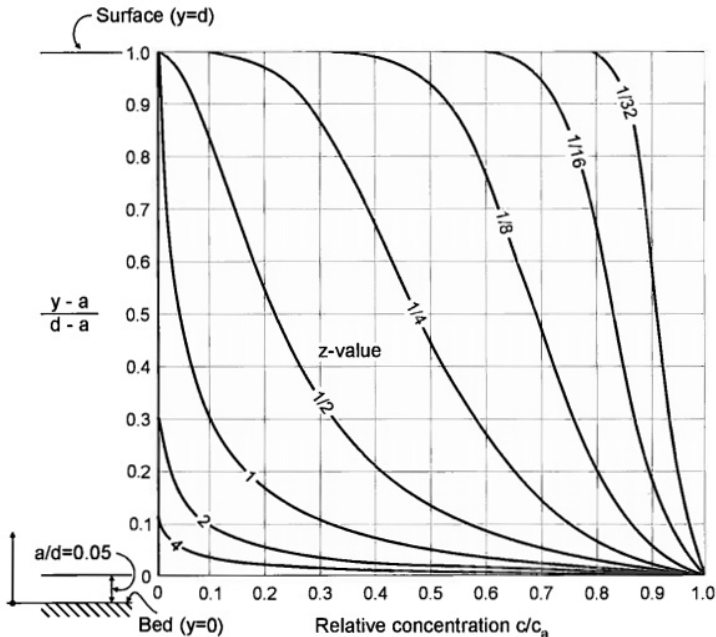


Figure 3.3: Sediment concentration distribution

Various formulas for the sediment transport capacity have been developed, among others Meyer-Peter Müller formula and Engelund and Hansens which are described in «Vassdragshåndboka» (NVE, 2010).

3.2.2 Settling basin design

Turbines in sediment-carrying rivers are exposed to turbine erosion. The total sediment load going through a power plant can be reduced by including a settling basin in the headwork design, to allow suspended solids to settle out of the water. The exclusion of sediments from the water is one of the most important considerations for a project in sediment carrying rivers, and a well-functioning settling basin is a prerequisite for a successful operation.



Figure 3.4: Settling basin at Jhimruk in Nepal

Design principles

A principle sketch of a settling basin is shown in Figure 3.5. The simple idea of the settling basin is to give the water sufficient detention time for the sediments to settle out before entering the waterways. It is not realistic to trap all suspended sediments in the water, and the size of the basin should be designed to balance the construction costs against the estimated costs of maintenance due to sediment induced wear on the turbines and other mechanical components over the project's lifetime. A normal design criteria for many projects in Nepal would be to trap all sediments with a diameter greater than 0.25 mm. Trap efficiency prediction is described in more detail later in this section.

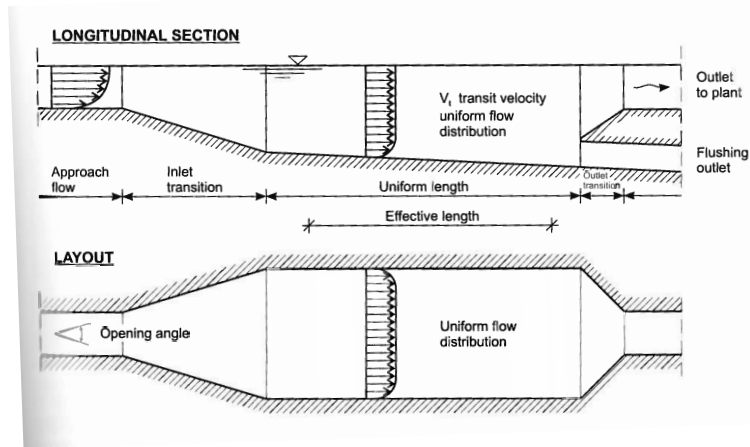


Figure 3.5: Principle sketch of a settling basin

To prevent turbulence from reducing the performance of a settling basin, it is important to secure smooth flow conditions as the water enters the basin. To be the most efficient, a settling basin should distribute the flow evenly over its width and depth. Space shortage and local topographic conditions can demand curved and unsymmetrical inlet transitions, which could develop secondary currents far down in the settling basins and also give a skew distribution of the water transported by the parallel chambers.

Flow tranquilizers can be used to reduce the level of turbulence, but they will also depending on the water velocity provide an extra head loss of 0.15 - 0.25 m to the system. The extra cost in head loss should be capitalized over the project's lifespan by reduced maintenance costs. Jhimruk Hydropower Plant in Nepal has tranquilizers installed as illustrated in Figure 3.6.



Figure 3.6: Flow tranquilizers at Jhimruk Hydropower Plant

Lysne et al. (2003) also recommends other methods to equalize the flow distribution into settling basins. One solution is to lead the flow through a pressurized section, and then smoothly transit it back to full width and depth.

To secure a stable outflow of the basin and prevent changes in the operation of the power plant from affecting the settling, Lysne et al. (2003) also recommends installing a slotted outlet of the basin. The slotted outlet will also result in extra production loss for the project due to head loss.

In general, it is hard to predict the performance of a settling basin, and a detailed model study is normally recommended. Analytical methods for estimating the trap efficiency may give indications of the capability of settling, but they cannot reveal poor hydraulic conditions, skew water distributions among the chambers or secondary currents in the basin. As these phenomenon will strongly affect the performance of a basin, a model study should be included in the design process. Analytical approaches to estimate the trap efficiency of settling basins are briefly discussed in Appendix E.

3.2.3 Flushing of headworks structures

To maintain a high trap efficiency, sediments needs to be flushed out from the settling basin to avoid an uncontrolled accumulation. Lysne et al. (2003) describes two main categories of flushing of settling basins, separated on their ability to deliver water to the power plant during flushing. Four different concepts of flushing can be defined within the two categories, as described in Table 3.1.

Table 3.1: Different types of flushing of settling basins (Lysne et al., 2003)

Settling basins flushing arrangements			
Close down during flushing		In operation during flushing	
1. Conventional gravity flow flushing	2. Excavators and manual unloading	3. Continuous flushing	4. Intermittent flushing

The two conventional flushing concepts are placed in the first category, where the settling basins are unable to deliver water to the power plant during flushing. The design of settling basins in the first category would have to include multiple parallel chambers, to allow for production while flushing or cleaning one or more chambers. In areas with shortage of electricity, power generation may be prioritized before the needs of flushing, and the flushing efficiency of the basins may be strongly reduced. Poorly operated settling basins may lead to heavy wear on turbines and other mechanical equipments due to high sediment concentration in the water passing the turbine.

The category of flushing concepts that can be used while delivering water to the power plant is divided between systems using continuous and intermittent flushing. For the

continuous flushing concept, some water is always extracted from the settling basins, so that deposition never will occur. Normally, around 20-30 % of the water entering the settling basins is used for continuously flushing. Several systems are developed for continuously flushing, i.e. the hopper-type and the Bieri system as described by Støle (1993). Both types are complex to construct, and requires a number of well-designed parts. The hopper-type requires many valves and pipes, and may experience problems if gravel enters the intake area. The Bieri has two plates at the bottom operated by a servo motor which can be slided to make several clearings. As these plates are in an sediment exposed environment, these plates are expected to suffer from sediment wear. Systems relying on continuous flushing are vulnerable to gate malfunctions or other operational problems, as it can be difficult to re-establish flushing if an amount of sediments have been allowed to accumulate in the basin.

Intermittent flushing systems are allowing for continuously production of the power plant, but is not depending on a continuous extraction of water to do so. Between flushing sessions, sediments are allowed to accumulate in a dead storage, and no excess water is used. During flushing, the different intermittent systems are using either stable or movable suction points to remove sediments from the basin. Dr. Haakon Støle has written his PhD on «Withdrawal of Water from Himalayan Rivers». In his PhD, Støle has proposed recommended design guidelines for settling basins which will be briefly summarized here.

- Sediments should be removed while the power plant is in operation. Conventional gravity flow flushing can supplement at bigger projects for safety reasons.
- The design and its operation shall be straightforward and not require any physical intervention since it is impossible to see anything through silty water.
- The system should be designed so that non-technical staff easily can operate it after receiving a minimum of training.
- It should be possible to operate the system without use of machinery which requires electrical power input.
- It should be possible to start a new flushing after maloperation, and not be necessary to do extensive repair works.
- Aim to use as little flushing water as possible. To have a system using less flushing water and high flushing capacity, a system using intermittent flushing is needed.

As a part of his PhD, Støle invented and developed a system for continuous, intermittent flushing, called *Serpent Sediment Sluicing System (S4)* (Støle, 1993).

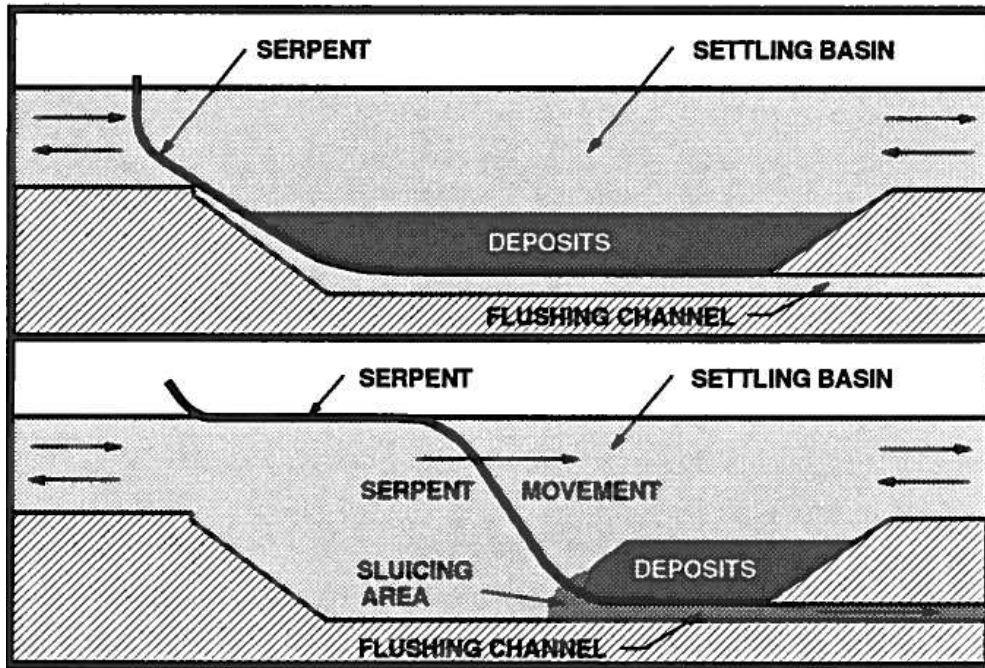
Serpent Sediment Sluicing System (S4)

The S4 system is designed to use small amounts of flushing water, and still maintain a high flushing efficiency. A sketch is shown in Figure 3.8. It is designed with a longitudinal

slot at the bottom of the settling basin, and a rubber serpent that can seal the slot if placed top of it. Below the slot is a flushing channel which is connected to a flushing pipe at the downstream end. The serpent can be filled or de-watered with an operation valve, and hence move the front of the serpent covering the slot. The head difference between the operating water level and the outlet of the flushing pipe is providing a suction point around the front of the serpent, that can be moved to cover the whole basin. When the serpent is de-watered, it will gradually get more buoyant and gradually lift and expose more and more of the slot from the upstream side. The suction area will gradually move downstream as the serpents rises, and the entire basin will get flushed for sediments. Flushing can also take place by filling the serpent with water and flush in closing mode. The method works in other words like a zip-fastener. Figure 3.7 shows the system at Andi Khola in Nepal during a rehabilitation of the headworks.



Figure 3.7: The Serpent Sediment Sluicing System at Andi Khola in Nepal



OPENING MODE

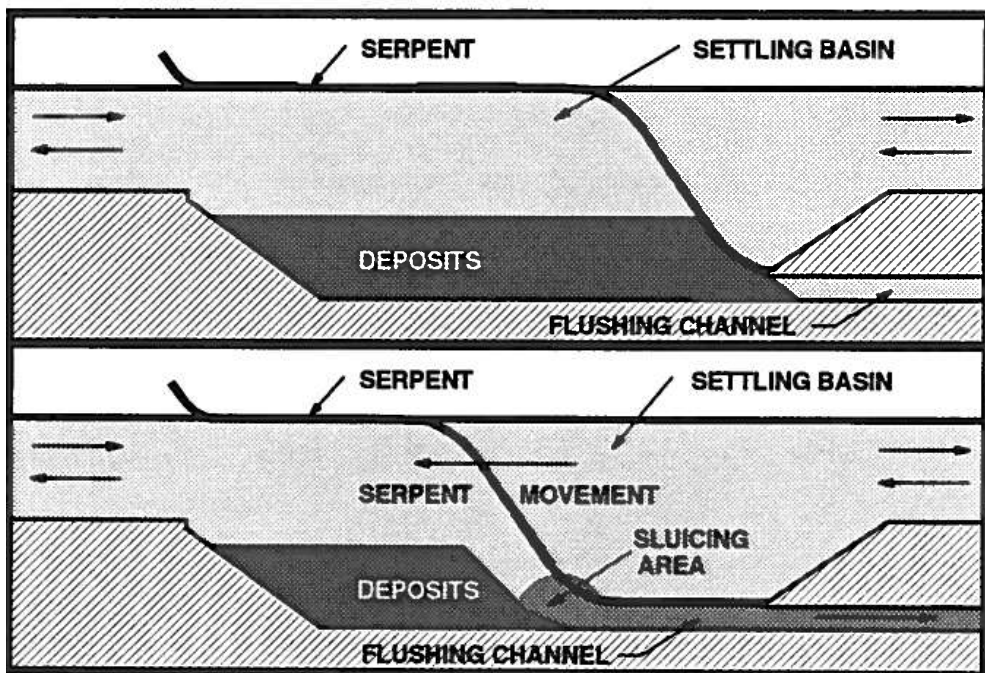


Figure 3.8: Concept of the S4 system (Støle, 1993)

SediCon Sluicer

The SediCon sluicer is another method of sediment removal in settling basins, and was invented by Tom Jacobsen during his PhD-work (Jacobsen, 1997). This system has a slotted pipe in the bottom of the settling basin where sediments can enter and be flushed out using suction and gravity. Pressure head for flushing is generated by the pressure difference from the water level in the settling basin and the outlet of the flushing pipe. Flushing with SediCon sluicer can be both continuous and intermittent. To start flushing, one person has to open a valve and then flushing starts due to gravity and pressure difference. No electric power is needed. Figure 3.9 illustrates the slotted pipe and how sediments are removed by suction.

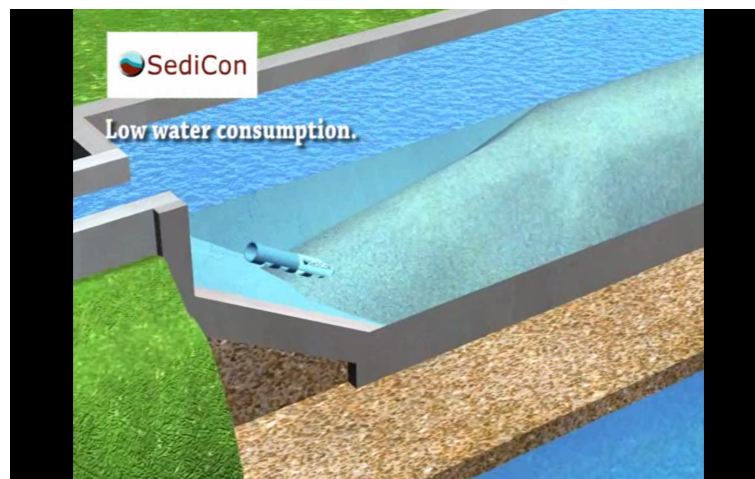


Figure 3.9: SediCon sluicer concept

Bed control at intake

In rivers with large sediment concentration, an intake needs to be designed bearing in mind that sediments can deposit and build up around the intake. The bed of the sediments should under no circumstances be allowed to reach to the level of the intake opening. A possibility to flush the accumulated sediments should be included in a proper headwork design. One solution is to place suction points connected with a flushing channel just below the intake as shown in Figure 3.10. The suction points will continuously keep the area in front of the intake clean.

Another option is to have undersluice flushing gates downstream of the intake as illustrated in Figure 3.11. It is a common experience that flushing gates have a short flushing range, but it is hard to obtain literature documenting bed control at intake using undersluice flushing gates. Designers have to lean upon the theory mentioned in section 3.2.1 and evaluate the shear distribution in front of the planned intake. For material transport, the appearing shear stress should be a bigger than the critical value for initial motion. Side

intakes, side contractions due to pillars between two gates and water through underflow gates makes shear stress calculations difficult. The complexity of the hydraulic conditions in front of an intake require physical and/or numerical model studies to be evaluated in full detail.

3.2.4 Model theory

To compare results from a model to a prototype, the kinematic and dynamic responses have to be the same. The magnitude of the forces acting on the model should be relatively the same as in prototype. Gravity can't be scaled, i.e. that the forces must be scaled in relation to gravity. The scaling factors may be developed from Newton's 2. law of motion:

$$F = m \cdot a \quad (3.8)$$

By defining the index p for prototype, m for model and r for the ratio between them, the length ratio and force ratio is respectively:

$$L_r = \frac{L_m}{L_p} \quad (3.9)$$

$$K_r = \frac{K_m}{K_p} \quad (3.10)$$

Similarities that needs to be fulfilled by the Froude number:

1. Inertia

$$F = m \cdot a = \rho \cdot L^3 \cdot \frac{L}{T^2} = \rho \cdot L^4 \cdot T^{-2} \quad (3.11)$$

which by introducing $V = \frac{L}{T}$ may be expressed as:

$$F = \rho L^2 V^2 \quad (3.12)$$

2. Gravity

$$F = m \cdot g = \rho \cdot L^3 \cdot g \quad (3.13)$$

where g is the acceleration of gravity.

The ratio between inertia and gravity may then be expressed as:

$$\frac{Inertia}{Gravity} = \frac{\rho \cdot L^2 \cdot V^2}{\rho \cdot L^3 \cdot g} = \frac{V^2}{g \cdot L} \quad (3.14)$$



Figure 3.10: Undersluice slot at Middle Marsyangdi HEP



Figure 3.11: Undersluice flushing gates at Lower Modi I, Nepal

This ratio in 3.14 must be same for both prototype and gravity:

$$\frac{\left(\frac{V^2}{g \cdot L}\right)_m}{\left(\frac{V^2}{g \cdot L}\right)_p} = \left(\frac{V^2}{g \cdot L}\right)_r = 1 \quad (3.15)$$

This is Froude's model law, usually written as:

$$\left(\frac{V}{\sqrt{g \cdot L}}\right)_r = 1 \quad (3.16)$$

The laboratory experiment conducted in this chapter required scaling of water discharge from model to prototype. By applying Froude's model law, the scaling was done as following:

$$\frac{v_m^2}{g \cdot L_m} = \frac{v_p^2}{g \cdot L_p} \iff \frac{v_p}{v_m} = \frac{\sqrt{L_p}}{\sqrt{L_m}}$$

$$\frac{Q_p}{Q_m} = \frac{v_p \cdot A_p}{v_m \cdot A_m} = \frac{\sqrt{L_p} \cdot L_p^2}{\sqrt{L_m} \cdot L_m^2}$$

$$Q_p = Q_m \cdot L_r^{1/2} L_r^2 = Q_m \cdot L_r^{5/2}$$

(Lysne, 1982)

3.3 Procedure

Different approaches have been conducted to cover the wide topic of headwork design. Field visits were conducted to expand the horizon of hydropower knowledge and sediment experience, and a model study was assessed to verify flushing in front of an undersluice gate.

3.3.1 Field visits in Nepal

In the report period, a 6 week trip to Nepal was conducted. The goal of the trip was to gain experience with sediment handling in Himalayan conditions, and to run model tests in the laboratory at HydroLab Pvt.Lmt. in Kathmandu. In total, 4 weeks was spent in the laboratory, contributing on the construction of a physical model of the Lower Manang Marsyangdi project. Unfortunately, the model was not finished on time, and no test series were run in the model during the stay. In the period of the visit, the lab staff were modeling the Marsyangdi river in it's natural condition, by carefully placing rocks and pebbles to represent boulders in the prototype.



Figure 3.12: Comparison of river model and prototype of LMM

To gain experience on headworks structures in Himalayan conditions, a long field trip was conducted to multiple project sites in Nepal. By renting a Jeep with a private driver, multiple project sites were assessed. The sites visited were:

Table 3.2: Power plants visited in Nepal

Project name	Company	Installed capacity [MW]	Stated annual production [GWh]
Jhimruk	Buwal Power Company Pvt.Lmt. (BPC)	12.3	76
Tinau	BPC	1	-
AndiKhola	BPC	5	41
Lower Manang Marsyangdi	BPC	210	735
Modi	Nepal Electricity Authority (NEA)	14.8	92.3
Lower Modi	United Modi Hydropower Pvt.Lmt.	10	60
Kali Gandaki	NEA	144	842
Middle Marsyangdi	NEA	70	470
Khodi	NEA	-	-
Sethi	NEA	1.5	9.8

The visits were conducted in dry season, and observation of operation during the monsoon period could unfortunately not be done. Using own observations and information and drawings provided by the power companies, evaluation of the different headworks structures were done. The amount of information available for the different sites varied, and hence also the level of detail on the analysis.

The performance of the intake structures are discussed in section 3.4, and evaluated up against the theory described in chapter 3.2.

3.3.2 Experiments in flushing flume

For sediment-carrying run-of-river projects it is a major challenge to keep control of the level of sediment deposition in front of the intake. In Nepal, it is normal practice to design side intakes, where the intake openings are perpendicular to a flushing channel, leading to a undersluice flushing gate.

It is a common experience that the suction obtained by the use of a undersluice gate is limited to an area relatively close to the gate itself, and that it has a limited ability to

flush out sediments at a high water level. A study of the effect of an undersluicgate was planned to be conducted at the physical model study of Lower Manang Marsyangdi, but as previously told, this model was not built finished in time. It was not possible to obtain relevant literature on the expected distance upstream a flushing gate that will be cleaned for sediments. It was then decided to conduct a physical model study at NTNU to document this *flushing range*.

Flushing range

The experiments were conducted in a 0.15 m wide, 3.25 m long and 0.48 m high, tiltable flume, shown in Figure 3.13. In the downstream end of the flume is a vertical lift gate, with a measuring rod to monitor the gate opening. Water was delivered from a pump with a governor, so that different water discharges could be set. Every experiment was documented by filming.

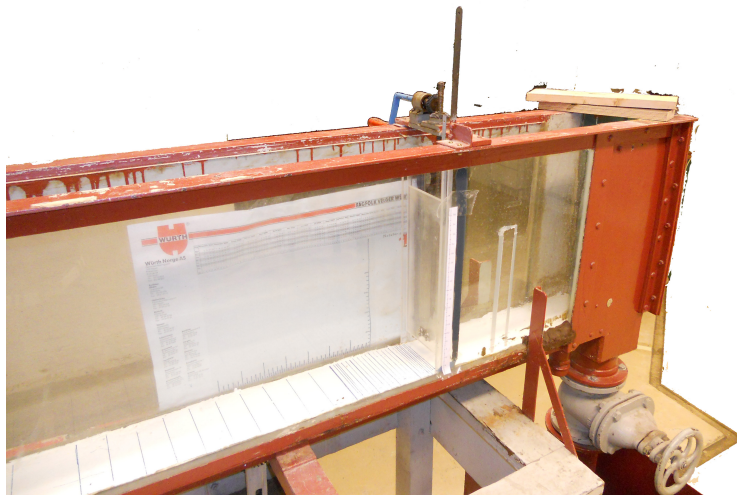


Figure 3.13: Laboratory flushing flume

To measure the flushing length, the test series were set up for a range of different discharges, and for different combinations of water level and gate opening. The test procedure is enclosed in Appendix H.

The test series were conducted with two different sediment distributions. One test series was run with sediment sizes ranging from 0.5 mm to 1 mm, and one test series with sediment diameters from 1 mm to 2 mm. It was assumed a linear grain size distribution between the upper and lower limit for each fraction, giving respectively $d_{60} = 0.8$ mm and $d_{60} = 1.6$ mm. After discussions with Tom Jacobsen in Sedicon, it was agreed upon to keep the slope of the flume at a constant value of 1/100. The slope was set to 1/100 to come close to the slope of the energy line in the flume.

During preliminary tests it was observed that the flushing length increased just after the very first seconds after the adjustment of the gate, and without any further development as the water level in the flume slowly decreased to a new stable level. It was then decided to interpret the new flushing length to correspond to the original water level before the gate was adjusted.

Figure 3.14 shows the setup during a test in the flume, with the sediment base in front of the intake at a distance of 0.06 m in front of the vertical lift gate.



Figure 3.14: Placement of sediments in front of model gate

The test series resulted in a set of flushing length measurements, for different combinations of water level h_w and gate opening a . The flushing length was represented as a function of the water level and the gate opening, by use of Matlab's polyfit function.

$$l_f = f(h_w, a) \quad (3.17)$$

A polynomial surface of degree 2x2 was adjusted to the dataset using a least square approach. The measured data points and the fitted surface contour lines for the test series for 2 mm sediments are Figure 3.15. The surface fitting gave a R^2 value of 0.945, and a Root Square Mean Error (RMSE) of 45 mm for the data series with 2 mm sediments.

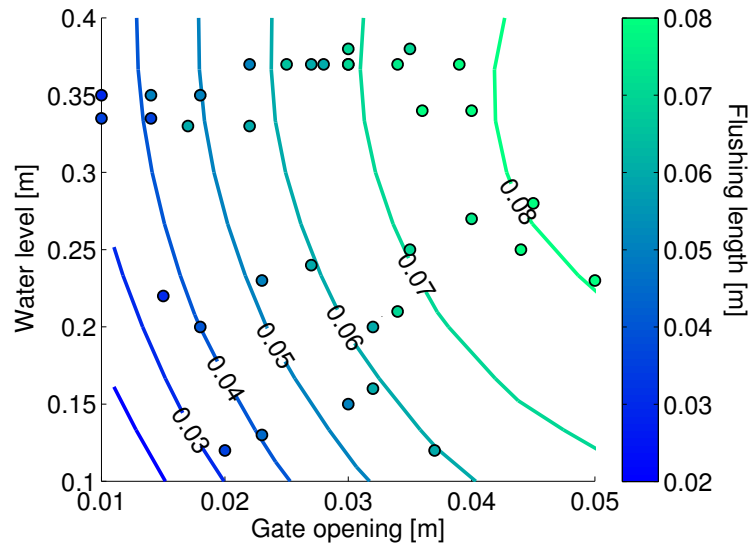


Figure 3.15: Flushing length l , as function of h_w and a

Bed load transport in the flushing model compared with Shields

As seen in the section 3.2, designers often have to make an estimate of the shear stress in the channel to evaluate if bed transport is formed. The complexity of the hydraulic conditions of an intake area makes it difficult to estimate shear stress exactly, and hence is it difficult to predict when bed load transport initiates. Therefore, in addition to look at flushing in front of the flushing gate, notes were taken when the initial movement of bed load transport for each water discharge was observed. The idea was to compare the observed water level which gave bed transport in the model, with the water level that for normal flow conditions gives bed load transport with Shields. Natural flow conditions is far from correct to assume, but if this rough estimate corresponds well with the observations from the model, it indicates that simple estimates with Shields can give a good basis for a physical model study of a headwork design.

Observed bed load transport have only been done for sediment size 1 - 2 mm, since the sediments from 0.5 - 1 mm was found to be difficult to make good observations of.

The theoretical particle size was calculated by applying Mannings formula for normal flow conditions. By adjusting the inclination in Mannings formula until normal flow conditions gave the same water level as the water level for observed bed load transport, the appurtenant critical particle size could be found. Mannings number of 60 for very coarse sand was chosen.

3.3.3 Scaling of flushing range tests

The planned design of the 93 MW Lower Manang Marsyangdi Hydroelectric Project include two 8.5 m high undersluice gates to keep control of the bed level in front of the intake openings. Due to geographic and topographic limitations for the headworks area, the intake is planned with shallow and wide intake openings, with the uppermost part of the opening as far as 45 m upstream of the flushing gate. Based on experience from existing projects, it was suspected that it would be a problem to keep this area clean without lowering the water level, so a simplified scaling of the project results was conducted.

The model tests were scaled to represent the situation for the undersluice flushing gates of the Lower Manang Marsyangdi project. During operation, the power company will seek to obtain an operating water level at 2093.9 masl, 8.5 meter above the bottom level of the flushing gate at 2085.4 masl. A water level in the model of 0.25 m in the flume was chosen to represent the operating water level at the prototype, giving a model scale of 1:34.

In the test series, it was not easy to get multiple measurements for exactly $h = 0.25$ m. Hence, the fitted surface function presented in Figure 3.15 was used, and the flushing length at $h_w = 0.25$ m was calculated as a function of gate opening a .

$$l_{0.25} = f(a) \quad \text{for} \quad h = 0.25m$$

To evaluate the discharge for different gate openings at $h_w = 0.25m$, the discharge was assumed to follow the equation based on NVE (2005):

$$Q_m = k \cdot b \cdot a \sqrt{2 \cdot g \cdot \left(h_w - \frac{a}{2}\right)} \quad (3.18)$$

where k is a correction constant, and b is the width of the gate.

For a constant width, the discharge could be written as

$$Q_m = K \cdot a \cdot \sqrt{h_w - \frac{a}{2}} \quad \text{where } K = k \cdot b \sqrt{2 \cdot g} \quad (3.19)$$

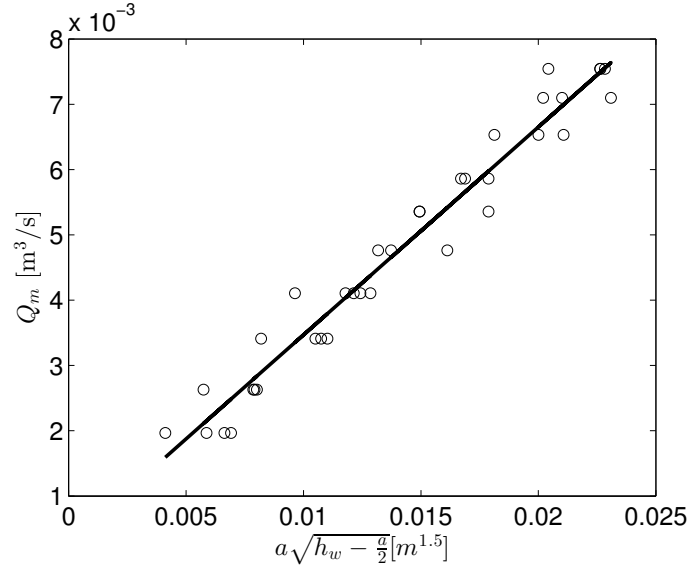


Figure 3.16: Fitting of the discharge curve for model gate

Figure 3.16 shows the values of Q was plotted against the parameter $(a_g\sqrt{h_w - \frac{a}{2}})$ for the series with 2 mm sediment size. The constant K is represented as the gradient of the curve, and calculated by linear regression in Matlab. By determining K , an explicit fitted function of Q_m was developed, based on a and h_w .

By setting $h_w = 0.25m$, Q , would be a direct linear function of a alone.

$$Q_m = K \cdot h_w^{-1} \cdot a \text{ for } h_w = 0.25m$$

$$Q_m = 0.3186 \cdot a \text{ for } h_w = 0.25m$$

Having fitted functions for both the discharge in the model and the flushing length as a function of the gate opening a , they could be plotted against each other.

The resulting series were scaled to the prototype by

$$L_p = L_m \cdot L_r \quad (3.20)$$

$$Q_p = Q_m \cdot L_r^{\frac{5}{2}} \quad (3.21)$$

$$d_p = d_m \cdot L_r \quad (3.22)$$

where L_r is the length scale, Q_p is the discharge in the prototype, d_p is sediment diameter d_{90} in the prototype.

The results were adjusted to a unit discharge by adjusting for the scaled model width.

$$q_u = Q_p / (bL_r) \quad (3.23)$$

The results of the scaling for the Lower Manang Marsyangdi flushing gate is presented in section 3.6.1.

3.3.4 Numerical model study

To check the observations done in the flushing flume, a simple numerical model study was conducted in co-operation with Hanne Nøvik using the commercial CFD programme Star CCM+. The setup of the numerical model can be found in Appendix G. The idea was to get the bed shear stress values from the CFD model, and compare these with the critical shear stress from Shields. If the shear stresses equals a critical particle diameter that were observed flushed in the flushing flume, it can verify that bed shear stress calculations can be used to predict flushing in front of a flushing gate.

Since no measurement of the roughness of the bed roughness was measured for the flushing flume, the roughness value for cast iron, 0.25 mm, was chosen. Two different model tests were conducted. Each test had different water discharge, water level and gate opening. The values were chosen to match values observed from the experiments in the physical model, this to make it easier to compare results.

3.4 Evaluation of headworks in Nepal

In this section, the solutions observed during the field visit to Nepal are evaluated up against the theory presented in section 3.2. The performance of the planned project Lower Manang Marsyangdi is discussed in section 3.6.

3.4.1 Bed control at intake

Avoiding uncontrolled accumulation of sediments in front of the intake structures is crucial for hydropower projects in sediment-carrying rivers. If the sediment bed is allowed to rise up to the level of the intake opening, the bed load transported with the river could be drawn through the intake and provide a problem to the gravel trap and settling basin. Figure 3.17 is showing the intake structure at the 70MW Middle Marsyangdi project in Nepal, where it has been installed undersluice openings directly under the intake screens. The undersluices are installed to avoid any uncontrolled accumulation of sediments in front of the intake.



Figure 3.17: Bed control solution at Middle Marsyangdi

Modi Hydroelectric Project (HEP)

Modi Hydroelectric Project (HEP) is a good example of a typical run-of-the river project with side intakes. Downstream of the intakes is undersluice flushing gates which shall keep the intake area free of sediment. Figure 3.18 shows a large sediment deposition. It is clear to see the small area the flushing gates manage to keep free of sediments. The flushing range can be seen upstream of the left flushing gate. In the same picture is a sediment front dipping from the deposition towards the intake area. Sediments are in

other words transported further into the gravel trap, and the problem is moved to the gravel trap.



Figure 3.18: Downstream flushing gates at Modi Khola HEP

Lower Modi I Hydropower Project

A similar problem to the one mentioned above can occur at a Lower Modi, which is a hydropower plant under construction in Nepal. Lower Modi uses undersluice gates downstream of the intake, just as Modi HEP. It is a common experience that undersluice gates only will keep small parts upstream of the gate clean of sediments.

Figure 3.19 shows the flushing channel for the gravel trap. If large amounts of sediments enter the gravel trap during high discharges, it may be impossible to flush it out due to the small suction point this flushing channel will provide.



Figure 3.19: Flushing channel at Lower Modi HEP

3.4.2 Settling basins

The critical electricity situation in Nepal makes it important to design power plants which can produce electricity also during monsoon with high sediment yields. Shut down in production is critical in a country with large deficits of electricity. In the following text, observations and evaluations of power plants visited Nepal are done. It was very difficult to collect data from the field visits, and the analysis are therefore mostly done on the basis of observations.

Modi Hydroelectric Project

At Modi Hydroelectric Project, it is installed conventional gravity flow flushing of the settling basin. During monsoon, they have to shut down the entire power plant a few hours to flush the settling basin. The project owners have explained that their flushing gates are too small, which slowed down the flushing process. For this particular power plant an intermittent system would be recommended, especially since they only have one desilting chamber. An intermittent system would have made the power plant run continuously and most likely without shut down.

Jhimruk Hydropower Plant

Exclusion of suspended sediments in Jhimruk is done in two parallel settling basins, equipped with a S4 intermittent flushing system. Both chambers have a total uniform

length of 36 m and a cross-section of approximately 18.4 m². With the total design discharge of 7m³/s, this yields a rough transit velocity of approximately 0.2 m/s and a detention time of 3 minutes. The design trapping particle diameter is 0.2 mm. From the intake, three approach canals lead the water through a 22.5 degree direction change, installed with three sets of flow tranquilizers to reduce the turbulence in the basin.

At Jhimruk, the rate of sediment induced sediment wear on the turbines is way higher than expected. The turbines undergo a frequent overhauling program, and have to be maintained yearly (Eltvik et al., 2012). This is partly a result of an insufficient turbine design, but could also be avoided with a proper design of the settling basins.

3.5 Results from model study

A model study was conducted first and foremost to evaluate the ability of an undersluice flushing gate to keep the intake area free of sediments, i.e. evaluating the *flushing range*. The model study also served the purpose of comparing observations of initial bed load transport by theory of critical shear stress from Shield's when applying Manning's formula for normal flow.

3.5.1 Flushing range

The experiments were conducted in a small flume with two different sediment sizes. For both experiments, it was observed that flushing range increased for increased water discharge, but on the other hand, as shown in Figure 3.20, the increase in flushing length is small compared to the increase in discharge. Figure 3.21 also shows small fluctuating flushing lengths for a given discharge, which demonstrates that the flushing length does not increase considerably for different combinations of gate opening and water level. Observed flushing lengths in the flushing flume did not exceed 0.1 m. It is clear that the flushing length in front of the intake gate does not increase strongly as the discharge increases, but remains limited to the local area in front of the gate. The results were scaled to represent a gate prototype height of $H = 8.5\text{m}$, by evaluating data for a water level in the flume of 0.35 m. The scale of the test was 1:34, and the sediments would represent particle sizes of 0.03 m and 0.05 m. From Figure 3.21 it can be seen that the scaled flushing length for a 8.5m high undersluice gate is limited to approximately 3 meter for a unit discharge of $10\text{ m}^3/\text{s}$ per meter.

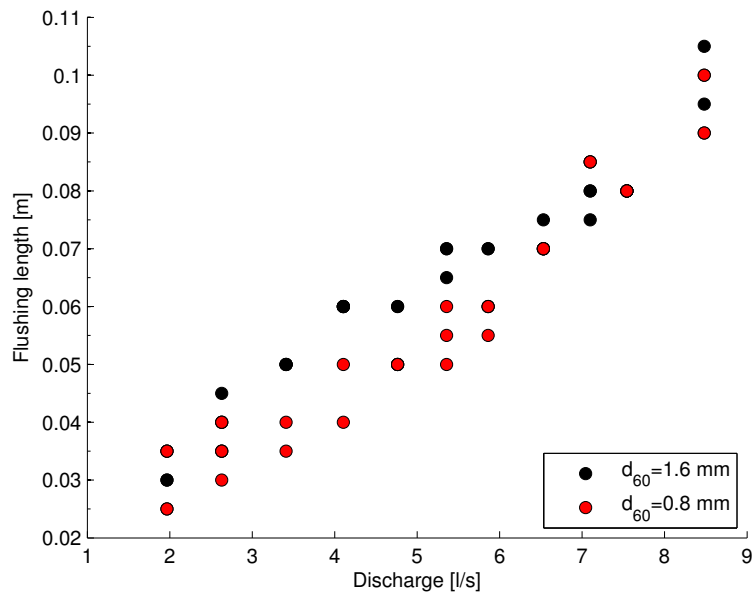


Figure 3.20: Flushing range in model flume as a function of water discharge

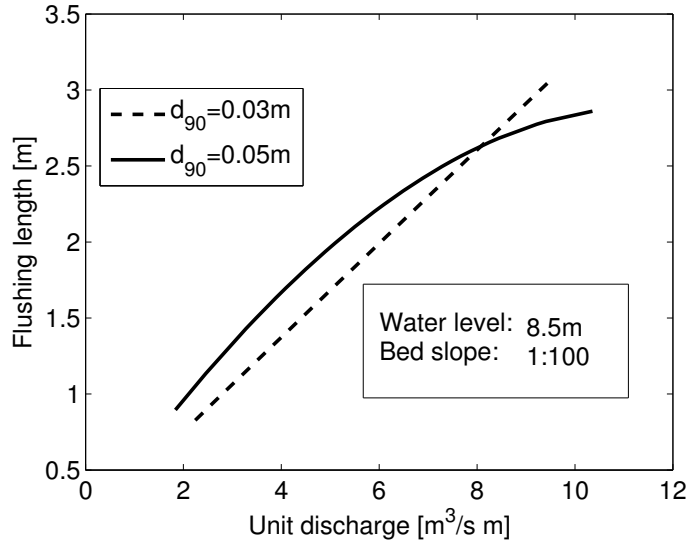


Figure 3.21: Estimated flushing length in front of prototype gate

3.5.2 Numerical model

A numerical model was set up to verify the distribution of shear stress in front of the gate. Shields formula for critical shear stress of the sediment particles was used for comparison with the measured flushing length in the physical model flume. Two numerical models were conducted with input parameters from observed test series in the physical model. The input parameters are listed in Table 3.3.

Table 3.3: Input parameters for the two numerical models

Test number	Q [l³/s]	Water level, h [mm]	Gate opening, [mm]
1	7.54	390	36
2	5.86	370	25

The critical shear stress for bed load transport was calculated from Shields. Assuming $C_s = 0.05$ for grain sizes of about 1 mm (NVE, 2010), the critical shear stress was calculated:

$$\tau_c = 0.05 \cdot (\rho_s - \rho_w) \cdot g \cdot d_s \quad (3.24)$$

which gives 1.6 Pa for $d_s = 2$ mm, 0.8 Pa for $d_s = 1$ mm and 0.4 Pa for $d_s = 0.5$ mm.

How the theoretical shear stresses compare with the numerical model study is shown in Figure 3.22. In Figure 3.22 the shear stresses from Star CCM+ are plotted against the

distance from the flushing gate, where the blue and red line represents different test series. In addition, the critical shear stresses calculated above are plotted. How the shear stress distribution from Star CCM+ compare with observations from the model, is presented in Table 3.4 and Table 3.5.

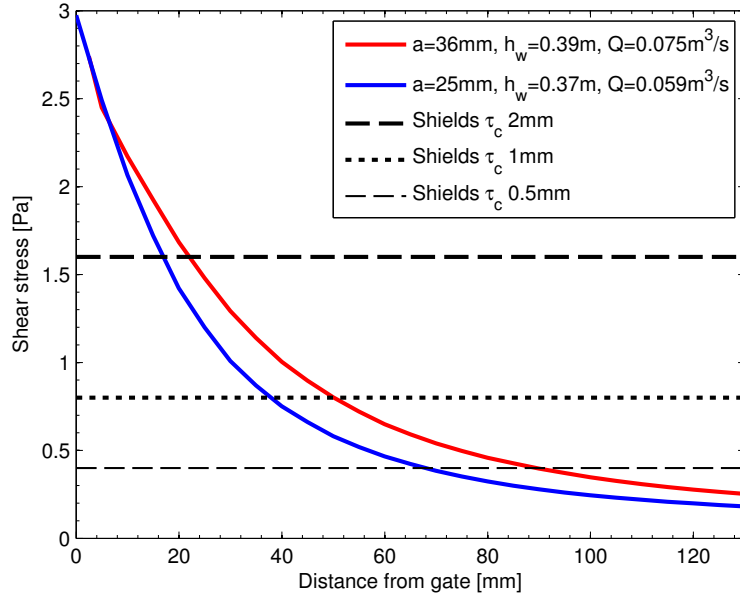


Figure 3.22: Shear stress distribution in front of flushing gate, from Star CCM+

Table 3.4: Numerical model test number 1

Sediment size	Critical shear stress from Shields	Expected flushing length upstream of gate	Observed flushing length
0.5 - 1 mm	0.4 - 0.8 Pa	50 mm - 90 mm	80 mm

Table 3.5: Numerical model test number 2

Sediment size	Critical shear stress from Shields	Expected flushing length upstream of gate	Observed flushing length
1 - 2 mm	0.8 - 1.6 Pa	20 mm - 40 mm	65 mm

3.5.3 Limit for bed load transport

As described in section 3.3.2 it was desirable to see how the observed bed load transport compares with Shields when normal flow is assumed. For every water discharge, the water

level where the initial bed load movement started was written down. For each water level a critical particle size was calculated as described in section 3.3.2. The results are presented in Table 3.6. For these discharges and water depths, the experiment gave movement of particles with $d_{60} = 1.6$ mm. Table 3.6 shows that bed movement should have occurred for a lower water level than what observations gives, i.e. the shear stresses calculated from Shields are lower than what observations from the flushing flume gives.

Table 3.6: Comparison of observed initial particle motion with theoretical critical particle size

Discharge [l/s]	Water depth [m]	Initial motion d_{60} [mm]	Critical d_{60} from Shields [mm]
1.97	0.055	1.6	0.50
2.63	0.055	1.6	0.91
3.41	0.075	1.6	0.72
4.11	0.075	1.6	1.04
4.76	0.085	1.6	1.03
5.36	0.105	1.6	0.82
5.86	0.135	1.6	0.54
6.53	0.155	1.6	0.49
7.10	0.155	1.6	0.58
7.54	0.165	1.6	0.56
8.48	0.235	1.6	0.31

3.6 Discussion

In this chapter, the observations and experiences obtained in Nepal and through model study at NTNU is used to evaluate the performance of the planned 93 MW Lower Manang Marsyangdi project. Further, the evaluation of the existing projects in Nepal is discussed, up against the theory described in section 3.2.

3.6.1 Lower Manang Marsyangdi Hydroelectric Project (LMM)

In this section the preliminary design of the Lower Manang Marsyangdi project has been evaluated up against experiences theory and existing projects, based on drawings provided by the designers. Due to restrictions from the owner of the project, the drawings could not be enclosed in this thesis.

Lower Manang Marsyangdi is a hydropower project planned in the Gandaki Zone of Nepal, developed by Butwal Power Company Pvt. Ltd. (BPC). The project is planned with an installed capacity of 93 MW, an annual production of 542 GWh, a net head of 309 m and a design flow of 37 m³/s (HydroLab, 2011). The project headworks is situated with a crest level of 2094 masl.

The Marsyangdi river is very steep, and capable of moving large boulders during periods with high discharge. Figure 3.23 is showing the river bed 500 m above the planned intake location, and the size of the transported boulders can be seen.



Figure 3.23: Natural river conditions, Lower Manang Marsyangdi

Upstream of the project site, the Marsyangdi river is running through an area dominated by deposited materials, cutting through the soil in deep ravines. Figure 3.24 is showing

a picture taken close to the town of Pisang, 10 km upstream of the project site. It is obvious that the river will transport a considerable amount of sediments during high discharges. The observed steep valley sides covering the river will also represent a major risk of landslides. Landslides blocking the river could lead to temporary detention of water, and a following extreme flood when the blockage is overtopped.



Figure 3.24: River conditions upstream of the LMM project site

The preliminary design of the intake of the LMM project is based on a *side intake* concept, with two intake openings located upstream of two flushing gates. In addition to the two spillway gates, the design includes a 35 m wide concrete gravity free surface weir, to provide the needed spillway capacity. The estimated 100 year flood is calculated to 1600 m³/s.

Bed control at intake

Lower Manang Marsyangdi is designed to have undersluice flushing gates downstream of the side intake to transport the bedload through the headworks. The distance from the undersluice gates and up to the uppermost part of the intake opening is 45 m. It will be of great importance to be able to avoid an uncontrolled accumulation of sediments in front of this area, and the ability of the spillway gates can be questioned.

The conceptual model study described in section 3.3.2 is supporting the idea that the flushing ability of an undersluice gate is limited to a short distance upstream of the gate.

For a water level in front of the gate of 8.5 m, the tests are indicating that it would require a considerable flushing discharge to achieve a flushing length over 3m. In Figure 3.25, the estimated flushing length is presented as a function of the total discharge through an effective gate width of 8m.

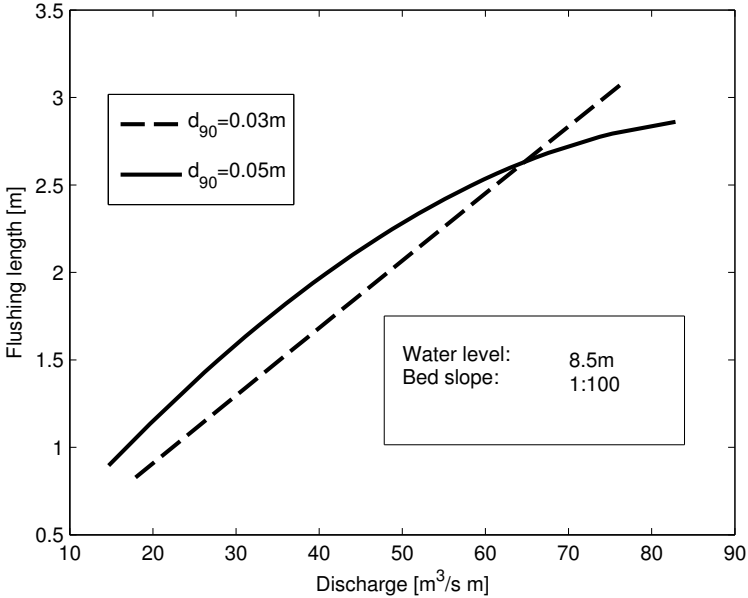


Figure 3.25: Estimated flushing length for Lower Manang Marsyangdi flushing gate

For a river discharge close to the design value of 37 m³/s, it should be expected a considerable transport of sediments in the river. As long as the the incoming discharge does not exceed the design discharge of the power plant, the water consumption used for flushing will be seeked held to a minimum. It is likely that an accumulation of sediments will occur in front of the upper intake opening also during discharges that does not exceed the power plant capacity.

Placement of undersluice slots in front of the intake openings would be a way of preventing an accumulation of sediments in front of the intake.

Exclusion of suspended sediments

The proposed headwork design of the LMM project includes a steep gravel trap directly after the intake, before the water directed to the settling basin. It has been suggested four parallel chambers, so that it can be possible to operate the power plant on three chambers, while flushing one chamber. The water consumption for flushing of gravel trap and settling basins is estimated to be 15% of the operating discharge of the power plant (HydroLab, 2011).

The use of a intermittent flushing system could provide a lower water consumption, as the estimated consumption for the S4 system is estimated to be 10% during flushing only (Støle, 1993).

3.6.2 Flushing range experiments

It has been verified that flushing with high water level and only driven by suction, only has an effect in a small area just upstream of the flushing gate. Variations in gate opening and water level does not seem to effect the flushing range. The flushing length observed has been scaled by using Froude's model law directly to present the flushing length in real life proportions. Scaling has only been made so that it reflects a water level of 8.5 meter, slope of 1/100 and sediments sizes from 0.03 m to 0.05 m. Scaled to prototype size, it can be seen that flushing is limited to a length of approximately 2-3 meters.

A source of error is that the side wall effect has not been taking into account. For a channel only 0.15 meter wide and 0.25 meter water level, the side walls amount to $\frac{0.25m \cdot 2}{0.25m \cdot 2 + 0.15m} = 77\%$ of the wetted perimeter. For comparison, during HRW at Lower Manang Marsyangdi is the side walls $\frac{8.5m \cdot 2}{8.5m \cdot 2 + (4.5m \cdot 2 + 1m)} = 63\%$ of the wetted perimeter.

It can be seen from Figure 3.21 that the test experiments and calculations gives longer flushing length for particles scaled to 0.05 m than for 0.03 m. This results seems unreasonable. The reasons for these results can be several, but one explanation can be that the experiments were conducted with uniform size distribution ($d_{60}/d_{10} = 1.6/1.0 = 1.5 < 2$) (Lysne et al., 2003), and the roughness of the bed is depending on the particles that is being flushed. Since 1-2 mm particles gives higher roughness than for 0.5 - 1 mm particles, the shear stresses are bigger for 1 - 2 mm particles. Hence, 0.05 m sediments provides the longest flushing length for this particular experiment.

Uncertainties

Flushing starts mostly with high water level due to high potential and higher velocity through the flushing gate. It is therefore desirable to observe flushing with many high water levels. After the first gate openings the water level sinks up to 0.10 m, and after a gate opening, it takes some time for the water level to stabilize. A stable water level was desired, but on the other hand, to have many water levels, the gate was opened before the water level was completely stable. This may have lead to some inaccurate measures in water levels.

The height of the gate should have been higher to better utilize the total height of the flume, but the height of the gate was made to fit the first experiment setup. But as the preliminary experiments were conducted, it was discovered that some rearrangements in the test procedure had to be done. This meant that a higher flushing gate could have been feasible, but then the gate had already been cut and mounted in the flume.

3.6.3 Numerical model study

The shear stress distribution in the flushing flume was estimated using a numerical model. It can be observed from Table 3.4 and 3.5 that observed flushing lengths in the flume

compares well with the estimated flushing length from the numerical model. In Star CCM+ is flushing length described as shear stresses which are transformed to flushing length by comparing critical shear stress from Shields. It can be seen from Figure 3.22 that the shear stress decreases rapidly upstream of the flushing gates. The numerical model confirms the observations from the flushing flume that flushing range only has an effect in small parts upstream of the flushing gate. Since observed flushing compares with numerical calculations, it suggests that numerical modelling is a good tool for evaluating the flushing length in a prototype model.

3.6.4 Verification of Shields to predict bedload transport

Observations from the model does not match perfectly with Shields calculation, but it indicates that Shields can be used with the assumption of average velocity and normal flow and still get good estimates for bed movement. If this simplified Shields approach is valid for dif Inaccurate meter reading or bad visual estimate of initial bed movement may be a source of error.

3.6.5 Modi Khola Hydroelectric Project

Results presented in chapter 3.5 supports the common experience that flushing driven by suction is effective only in small parts upstream of the flushing gate. The flushing range is also well documented by Figure 3.18 in section 3.4 that shows how the sediments has deposited in front of the intake. To keep the intake area free of sediments it is necessary to lower the water level so that bed movement is activated. Initiating bed movement requires the gates to open up much which gives increased velocity in the intake area. Increased velocity results in increased turbulence which again result in that less particles deposit and more particles enters in the gravel trap. A better solution would have been to have undersluice slots below the intake.

3.6.6 Lower Modi I Hydropower Project

The sediments will be difficult to flush out, knowing how short flushing range a small suction point will provide. Free surface flow through this gravel trap flushing channel is not possible during operation. The flushing gates can keep the closest intake screens free of sediments, but the most upstream intakes are depending on bed transport which requires large amounts of water. Another problem arises when gates are opened enough to enable bed transport, and that is increased velocity in the intake area. Increased velocity results in increased turbulence in the intake area which again may result in that less particles deposit and more particles enters the gravel trap.

3.6.7 Combination of settling basin and trash racks

In chapter 2, the concept of backflushing of intake trash racks has been evaluated. In chapter 3, the focus has been devoted to sediment handling at a power plants headworks. In reality, several projects are facing severe challenges with both floating debris and sediments at the same time. Hence, if proven reliable, the concept of backflushing would also be of interest for headworks that also includes settling basins.

The projects visited in Nepal were mostly situated at high elevations, with limited areas of forest and urbanization located in the catchment. Projects in the Himalayan region that are situated downstream of urban areas will always have to cope with amounts of human trash, in addition to natural debris.

The concept of backflushing depend on an available water volume to clean the trash rack. For headworks that also includes settling basins, it would have been favourable to be able to use parts of the water stored in the basin for the backflushing process.

The placement of the trash rack . For most projects, the settling basins would be placed in the open, and not covered from the surroundings. During periods of strong winds, trash and other objects should be expected to end up in the setting basins, and should be prevented from entering the waterways. Hence, a fine trash rack should always be installed after a settling basin located in the open. The amount of debris expected to be transported by wind is small, so a possibility for manual cleaning should be sufficient.

The trashrack designed for removal of floating debris transported with the river should have mechanisms for automatic cleaning. A normal practice at the observed projects in Nepal is to install a coarse trash rack upstream of the settling basin, and a fine trash rack with automatic cleaning in the downstream end.

If the project can afford two sets of fine trash racks, the main rack can be situated upstream of the settling basin. This would enable the possibility for backflushing, as the water from the settling basin can be used. The Middle Marsyangdi project, as well as Kali Gandaki (Figure 3.26), have both the fine trash racks installed upstream of the settling basin. Middle Marsyangdi uses an underground settling basin protected from wind-transported debris, while Kali Gandaki has installed an extra set of trash racks in the end of the settling basins.



Figure 3.26: The settling basins at Kali Gandaki

3.6.8 Further work

The model tests conducted in this chapter should be evaluated as rough estimations, to obtain a understanding of the big picture of sediment handling. To predict the performance of planned projects in detail, more accurate testing methods should be applied.

The model study of the Lower Manang Marsyangdi hydropower project at HydroLab in Kathmandu will give valuable indications of the performance of the suggested headworks design, and it would be of interest to compare the results with the suggestions presented in this report.

The economical aspect of sediment handling has not been looked at in this report. Optimizing the design of a headworks from an economical perspective is a complex task, and require understanding of the processes and costs connected to sediment induced wear on turbines and other mechanical equipment. It would be of interest to compare different efforts to reduce the costs due sediment induced wear on mechanical equipment, both from a civil- and mechanical-engineering point of view. As a possible collaboration project between students from different engineering disciplines, the areas of sediment handling could be approached in a larger context.

Chapter 4

Conclusion

Careful design of intake structures proves to be important for obtaining a reliable operation for hydropower projects facing challenging conditions. Observed projects in Nepal as well as small hydropower projects in Norway experience concentrated distributions of the available discharge, where a major fraction of the annual production takes place during seasonable floods. Hence, major operational problems during periods of high transport of sediments and/or floating debris is critical for the economy of the projects.

For rivers carrying large amounts of leaves, weed and branches, backflushing of trash racks seems to be a suitable concept to maintain production ability. Experiments conducted in the hydraulic laboratory at NTNU have shown that an induced gross water velocity of 0.1 - 0.2 m/s over the trash rack during flushing will be adequate to detach clogged debris from the rack. Based on the experiments in this master's thesis, it is recommended to design for a flushing velocity of at least 0.2 m/s. The clogged material tend to be able to resist a maximum pressure difference before it detaches from the trash rack, and not depend directly on the channel velocity.

Experience from the field as well as laboratory tests reveal that the flushing effect over the rack is reduced as parts of the rack is cleaned and openings in the clogged material have been made. Hence, to obtain an efficient cleaning for the whole rack, the flushing velocity over the gate should preferably be achieved by a rapid acceleration using a short opening time of the flushing gate. The pressure difference over the trash rack needed to detach the clogged debris is found to lie around 0.08 mWC, and can be shown to increase with the adhesion of the debris. As recommended further work on this topic, the appearing adhesion of the debris on trash racks at existing project sites should be evaluated, and compared with the results from the laboratory tests.

For hydropower projects operating in rivers with high sediment transport, the design of the intake area is of major importance to secure reliable operation and to avoid sediment induced wear on turbines and mechanical equipment. In addition to common functional requirements regarding passage of floods, handling of floating debris and entrained air, the headworks design should also include measures to deposit suspended sediments from the water transported to the turbine, and to avoid an uncontrolled accumulation of sediments

in front of the intake. The amounts of accumulated sediments that can be expected are in most cases too extensive for manual or mechanical removal, and systems for flushing should be included in the design.

Flushing can be obtained either by setting up local suction points, or by achieving a general water velocity high enough to enable bed transport over a larger area. For the flushing of settling basins, conventional gravity flushing using bed transport proves to be an inefficient method, with high water consumption and a long flushing duration. Methods for flushing of settling basins by using movable suction points proves to be more efficient, and can be operated continuously. The S4 system developed by prof. Haakon Støle and the SediCon Sluicer from the company Sedicon are both examples of intermittent flushing systems using movable suction points to continuously clean settling basins for accumulated sediments.

To avoid bed load from entering an intake, the bed level in front of the intake should be controlled. Placing undersluice openings directly in front of the intake is a reliable way of avoiding uncontrolled accumulation of sediments in front of the intake area. Undersluice flushing gates proves to be efficient in removing sediments only at a restricted length upstream of the gate, and can not be expected to be efficient for flushing a large area during a high water level.

To flush out accumulated sediments over a larger area, the water level would have to be lowered to enable flushing by bed transport. Uncontrolled accumulation of sediments can be reduced by operating the intake at a lower water elevation during periods of high sediment transport.

The proposed design of the planned 93 MW Lower Manang Marsyangdi has been evaluated based on observations from Nepal and model study. Tests indicate that the headworks may face severe problems in avoiding accumulation of sediments in front of the intake opening, as the intake is located far from the flushing gate. Placement of undersluices in front of the intake opening may have increase the probability of a reliable operation. The settling basins are designed with a conventional flushing system, and is designed to use 15 % of the operating discharge for flushing of sediments. The amount of water used for flushing could have been reduced by choosing an intermittent flushing system.

Bibliography

- Camp, T. (1946). Sedimentation and design of settling tanks. *Trans. ASCE, Vol III*.
- Converstation with Egil Berge, . (2012).
- Converstation with Kåre Natland, . (2012).
- Daly, S. F. (1991). Frazil ice blockage of intake trash racks. *Cold Regions Technical Digest*.
- Eltvik, M., B. S. Thapa, O. G. Dahlhaug, and K. Gjørseter (2012). Numerical analysis on effect of design parameters and sediment erosion on a francis runner.
- Goldman, S., K. Jackson, and T.A.Bursztynsky (1986). *Erosion & Sediment Control Handbook*. McGraw-Hill.
- Guttormsen, O. (1989). *Vassdragsteknikk II*. NTNU, Institutt for vann- og miljøteknikk.
- Hoerner, S. F. (1965). *Fluid Dynamic Drag*. Midland Park, NJ, USA: Independent Publisher.
- HydroLab (2011). Salient features of lower manang marsyangdi hydropower project.
- Jacobsen, T. (1997). *Sediment problems in reservoirs: control of sediment deposits*. Ph. D. thesis, NTNU.
- Jenssen, L., E. Tesaker, S. Lund, and D. Huber (2006). *Inntakshåndboken*. NVE.
- Lia, L. and L. Jenssen (2003). Grunne inntak. Technical report, NVE.
- Lvovich, M., G. Karasik, N. Bratseva, G. Medvedeva, and A. Maleshko (1991). Contemporary intensity of the world land intercontinental erosion. *USSR Academy of Sciences*.
- Lysne, D., B. Glover, H. Støle, and E. Tesaker (2003). *Hydraulic Design*, Volume 8 of *Hydropower Development*. Norwegian Institute of Technology, Division of Hydraulic Engineering.
- Lysne, D. K. (1982). Laws of similtude for model studies. Technical report, Norconsult.
- NVE (2005). *Retningslinjer for flomløp*.

- NVE (2009). Ressurskartlegging små vannkraftverk. <http://www.nve.no/no/Energi1/Fornybar-energi/Vannkraft/Ressurskartlegging/>. Hentet 07.10.2011.
- NVE (2010). *Vassdragshåndboka*. Tapir forlag.
- NVE (2011). Vannkraftpotensial 2011.
- Nøvik, H., L. Lia, and O. A. Jørstad (2011). Back flushing of trash racks at intakes to small hydro power plants.
- Rettedal, B. and L. E. Nielsen (2012). Inntak for småkraftverk med tilbakespylingsmulighet. Technical report, NTNU.
- Rouse, H. (1961). *Fluid Mechanics for Hydraulic Engineers*. Dover, NY: Engineering Societies Monographs.
- Shields, A. (1936). *Application of similarity principles and turbulence research to bed-load movement*.
- Støle, H. (1993). *Withdrawal of water from Himalayan rivers: sediment control at intakes*. Ph. D. thesis, NTNU.
- Vanoni, V. A. (1975). *Sedimentation Engineering*.

Appendix A

Trash rack adhesion tests at Bergedammen

The results from the adhesion measurements at Bergedammen are presented in Table A.1. The tests were conducted by using the PEETRA system as described in section 2.3.2. During the test series it was attempted on different measurement techniques, as the clogged material stuck to material in surrounding bar openings. It was attempted to cut the material of interest loose from the surrounding bars by using a knife, but it was found to disturb the adhesion of the material of interest.

The tests without the side-cutting of clogged material was decided to be the most representative to the tests in the laboratory, and the mean value was calculated based on test 5 - 9.

Table A.1: Trash rack adhesion at Bergedammen

Test nr	F_{gross} [N]	$F_{friction}$ [N]	Net Adhesion [mWC]	Comment
1	48	14	0.25	No cut
2	22	13	0.07	Cut
3	21	14	0.05	Cut
4	20	15	0.04	Cut
5	32	17	0.11	No cut
6	25	15	0.07	No cut
7	32	16	0.12	No cut
8	29	16	0.10	No cut
9	29	17	0.09	No cut

Appendix B

Adhesion test measurements from laboratory

In Table B.1, the manual clogging test series of the trash rack sections in the laboratory at NTNU is presented.

Table B.1: Adhesion test measurements from laboratory

	Test 1				Test 2				Test 3			
	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]
	22	7	15	0.11	38	7	31	0.23	32	9	23	0.17
	21	8	13	0.10	36	7	29	0.22	22	9	13	0.10
	23	8	15	0.11	36	7	29	0.22	44	8	36	0.27
	20	8	12	0.09	43	8	35	0.26	37	9	28	0.21
									29	8	21	
									27	9	18	
Number of pulls	4			0.03	4				6			
mean	21.50	7.75	13.75	0.10	38.25	7.25	31.00	0.23	31.83	8.67	23.17	0.17
Std.dev.	1.29		1.50	0.01	3.30		2.83	0.02	7.78		8.04	0.06
70%pred.int	1.80		2.10	0.02	3.24		3.95	0.03	6.23		10.03	0.08
max	23.30		15.85	0.12	41.49		34.95	0.26	38.06		33.20	0.25
min	19.70		11.65	0.09	35.01		27.05	0.20	25.61		13.13	0.10

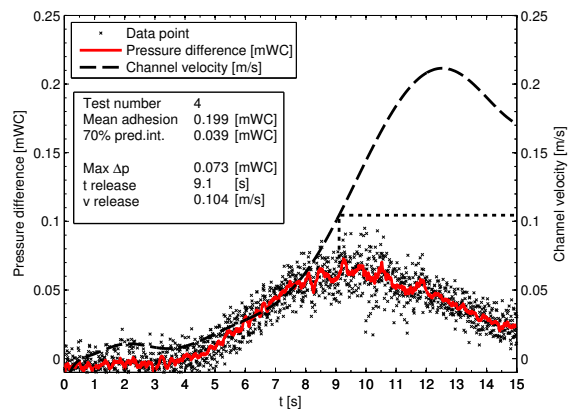
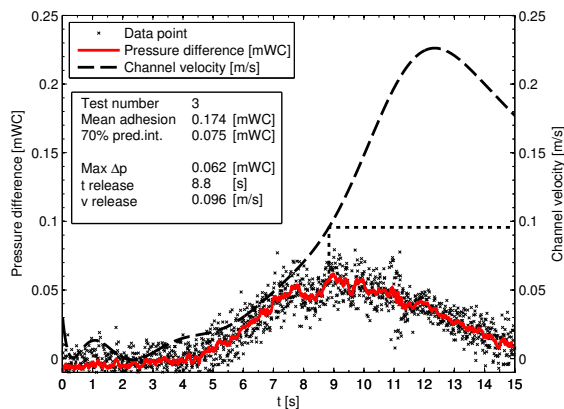
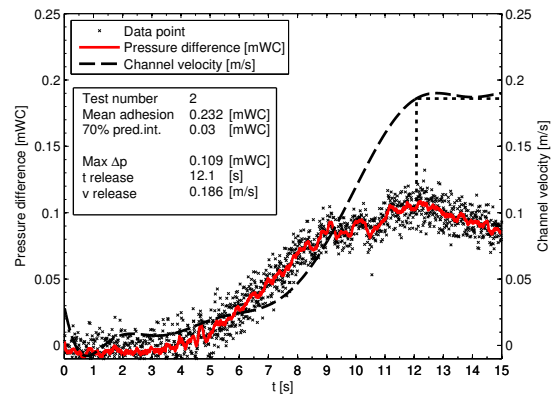
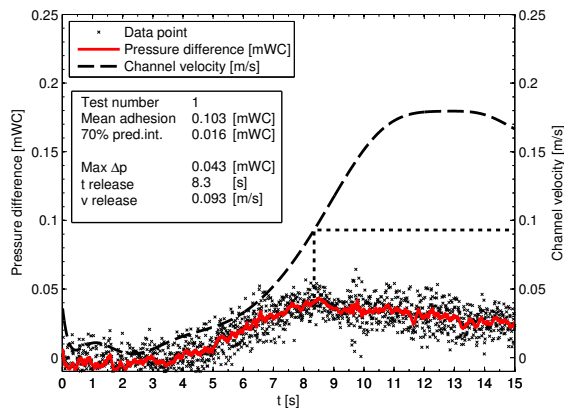
	Test 4				Test 5				Test 6			
	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]
	30	8	22	0.16	26	7	19	0.14	38	7	31	0.23
	35	9	26	0.19	23	8	15	0.11	35	8	27	0.20
	40	9	31	0.23	29	8	21	0.16	47	8	39	0.29
	37	10	27	0.20	24	8	16	0.12	42	7	35	0.26
Number of pulls	4				4				4			
mean	35.50	9.00	26.50		25.50	7.75	17.75		40.50	7.50	33.00	
Std.dev.	4.20		3.70	0.03	2.65		2.75	0.02	5.20		5.16	0.04
70%pred.int	4.12		5.17	0.04	2.59		3.85	0.03	5.09		7.22	0.05
max	39.62		31.67	0.24	28.09		21.60	0.16	45.59		40.22	0.30
min	31.38		21.33	0.16	22.91		13.90	0.10	35.41		25.78	0.19

	Test 7				Test 8				Test 9			
	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]	F_{gross} [N]	F_{friction} [N]	F_{net} [N]	σ_r [mWC]
	23	8	15	0.11	30	7	23	0.17	33	8	25	0.19
	25	8	17	0.13	28	7	21	0.16	40	8	32	0.24
	22	7	15	0.11	37	8	29	0.22	37	9	28	0.21
	21	7	14	0.10	23	7	16	0.12	37	9	28	0.21
					36	8	28		33	9	24	
Number of pulls	4				5				5			
mean	22.75	7.50	15.25	0.11	30.80	7.40	23.40	0.18	36.00	8.60	27.40	0.21
Std.dev.	1.71		1.26	0.01	5.81		5.32	0.04	3.00		3.13	0.02
70%pred.int	1.67		1.76	0.01	5.09		6.93	0.05	2.63		4.08	0.03
max	24.42		17.01	0.13	35.89		30.33	0.23	38.63		31.48	0.24
min	21.08		13.49	0.10	25.71		16.47	0.12	33.37		23.32	0.17

Appendix C

Results for backflushing test series

In this Appendix, the development of pressure difference over the trash rack and channel velocity during each of the nine flushing tests described in section 2.3.2 are presented. The figures display the results from the corresponding clogging series, to predict the adhesion on the trashrack before flushing. All the tests were conducted in a standardized way, where the flushing valve was opened at a constant in the time span from $t = 0$ to $t = 14$ s.



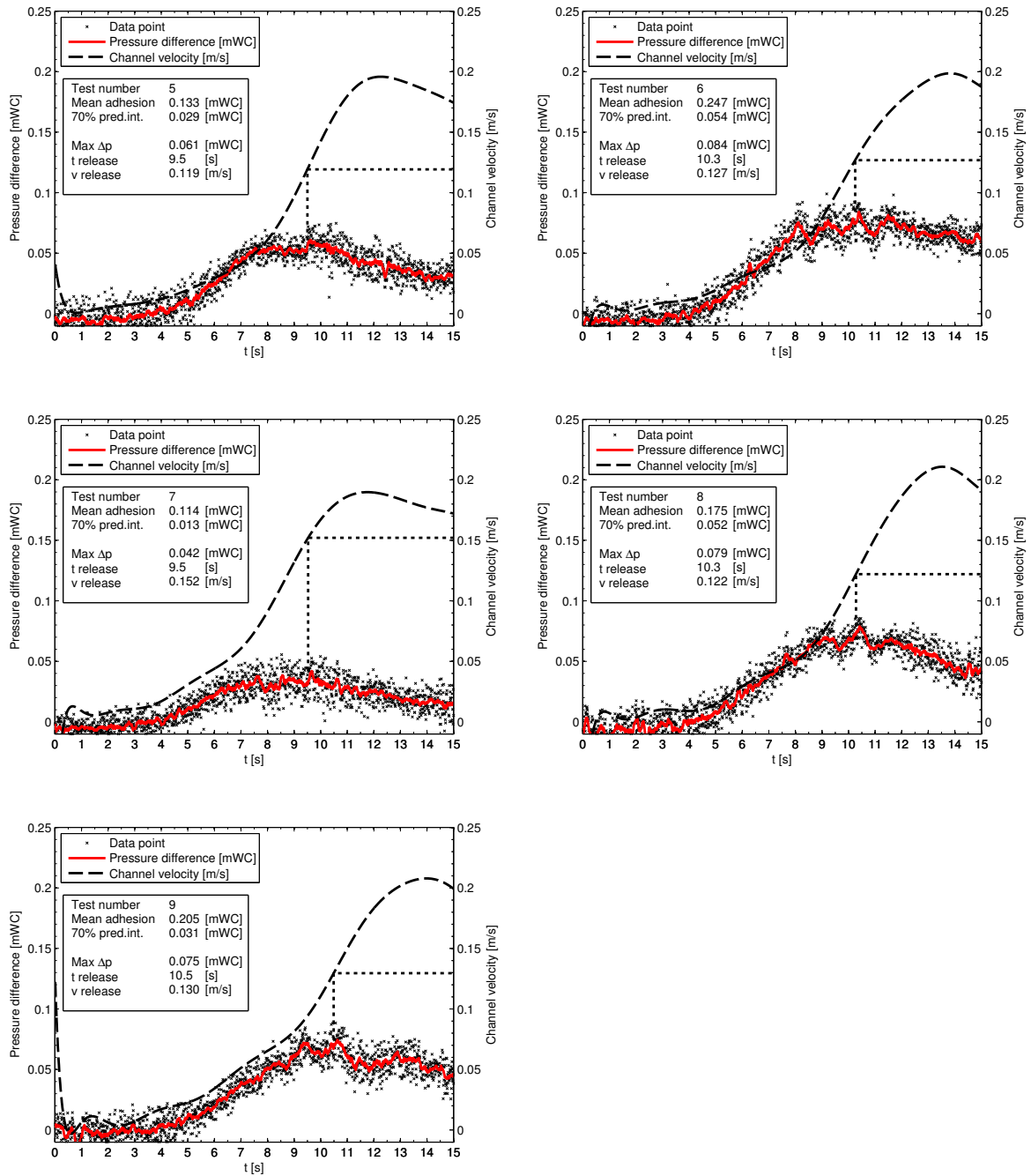


Figure C.1: Detailed results from backflushing test series

Appendix D

Photos of trash racks after flushing

After each backflushing experiment in the laboratory, the resulting flushing efficiency was evaluated based on the amount of clogged material left on the trash rack. The pictures show the trash rack after each flushing test. A photo of the flushed trash rack from test number 5 is not available.

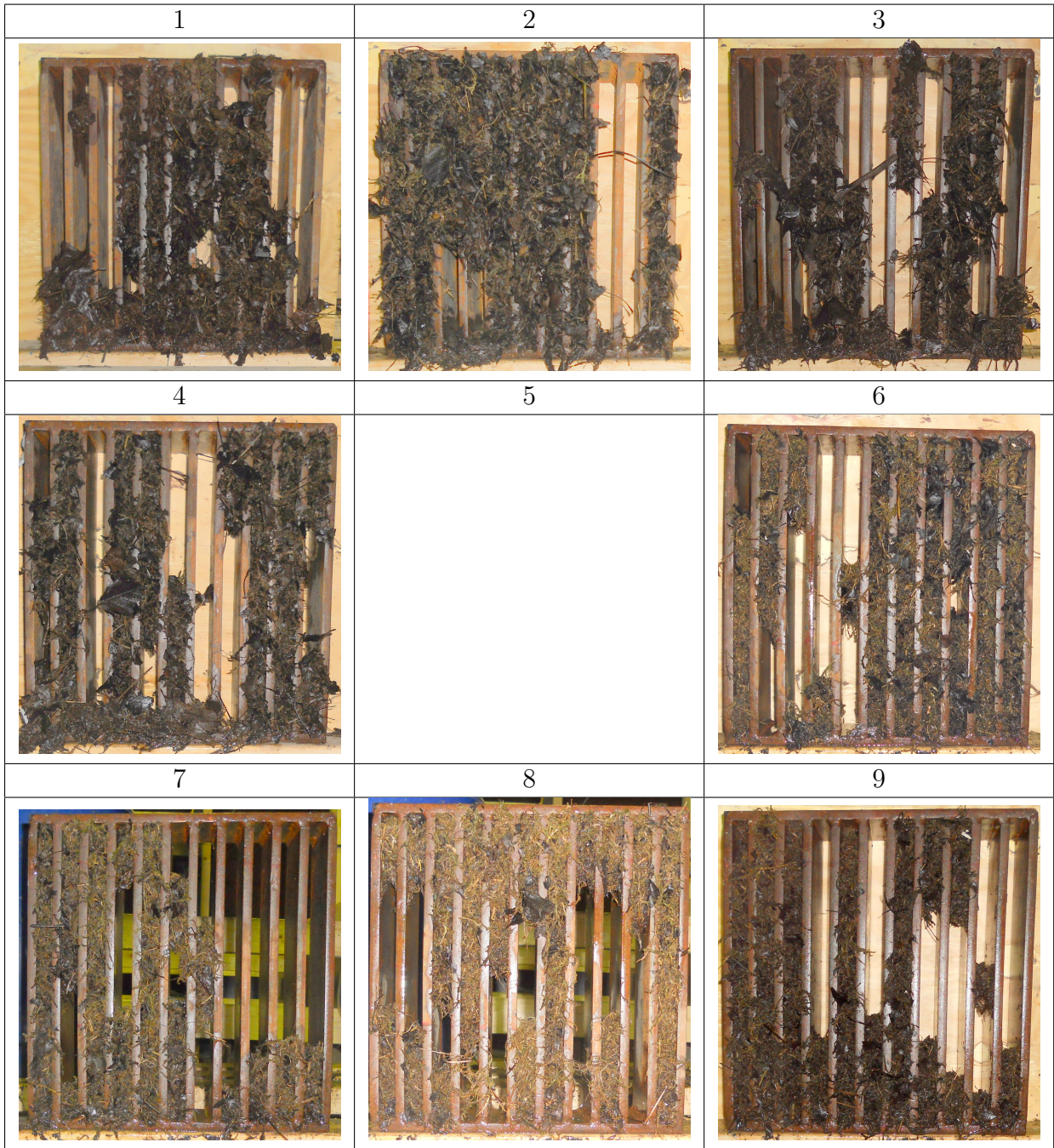


Figure D.1: Images of trash racks after flushing tests

Appendix E

Trap efficiency of settling basins

The trap efficiency in a settling basin is defined as follows:

$$\eta = \frac{Q_{s,in} - Q_{s,out}}{Q_{s,in}} \quad (\text{E.1})$$

where $Q_{s,in}$ is the amount of sediments entering the basin, and $Q_{s,out}$ is the amount of sediments that fail to settle and continue into the waterways.

A simple approach of estimating the trap efficiency of a settling basin would be to compare the required settling time for a threshold particle to the detention time of the water body in the basin. However, turbulence in the water body will always provide some movement in the upward direction and mixing of the water body, and the trap-efficiency will be reduced. Hence, reducing the level of turbulence to a minimum is a critical consideration when designing the a settling basin and its approach channels.

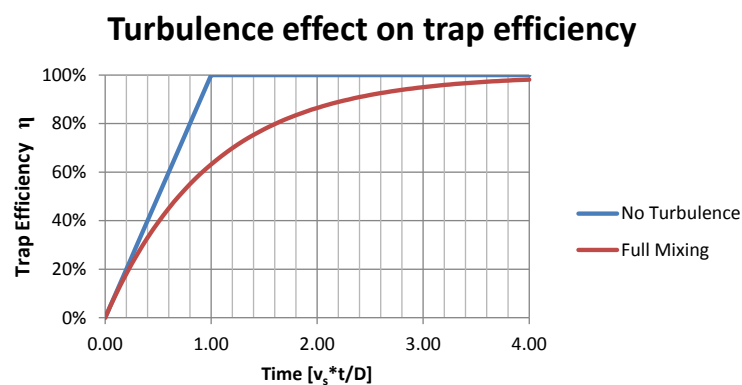


Figure E.1: Effect of turbulence on trap efficiency

Figure E.1 describes the effect of turbulence in a settling basin with two extreme scenarios; a quiescent water body with no turbulence (blue line), and a scenario with full mixing of

the water body in the settling tank (red line). For the ideal scenario with no turbulence, all particles of same size will have the same settling velocity, and the flux of sediments settling at the bottom will be constant. Hence, the trap efficiency will be a linear function reaching 100% when the settling length for a single particle reaches the basin depth. For the fully mixed scenario, the flux of sediments settling at the bottom will be reduced as the concentration of sediments in the water body goes down. Hence, it requires a long detention time to allow the last sediments to settle. It can be shown that for a theoretical fully mixed scenario, the required time reach a 95% trap efficiency would be 2,3 times the theoretical settling time for a single particle $t_s = D/V_s$, and to achieve a 99% trap efficiency it would require 4.6 times the settling time.

The turbulence in a real-world settling basin would however be somewhere in between the scenarios described in FigureE.1. There are various methods for evaluating the effect of turbulence on the trap efficiency, and their different approaches have been compared.

To evaluate the level of turbulence on the performance of the basin, Lysne et al. (2003) suggests the use of Camps diagram (Camp, 1946). This method uses a turbulence correction factor to the quiescent calculation assumption. Here, the turbulence is accounted for as the ratio between shear velocity and particle settling velocity.

Vetter's method, as described in Vanoni (1975) is another way of calculating the trap efficiency based on a full-mixing assumption. This method is based on a concentration approach, and hence it yields a longer settling time in order to settle out the last fraction of the suspended sediments.

$$W = W_0 \cdot e^{-\frac{wx}{q}}$$

Appendix F

Friction calibration of pulling device

To evaluate the effect of internal friction in the PEETRA system for measuring trash rack adhesion, it was performed a series of calibration tests.

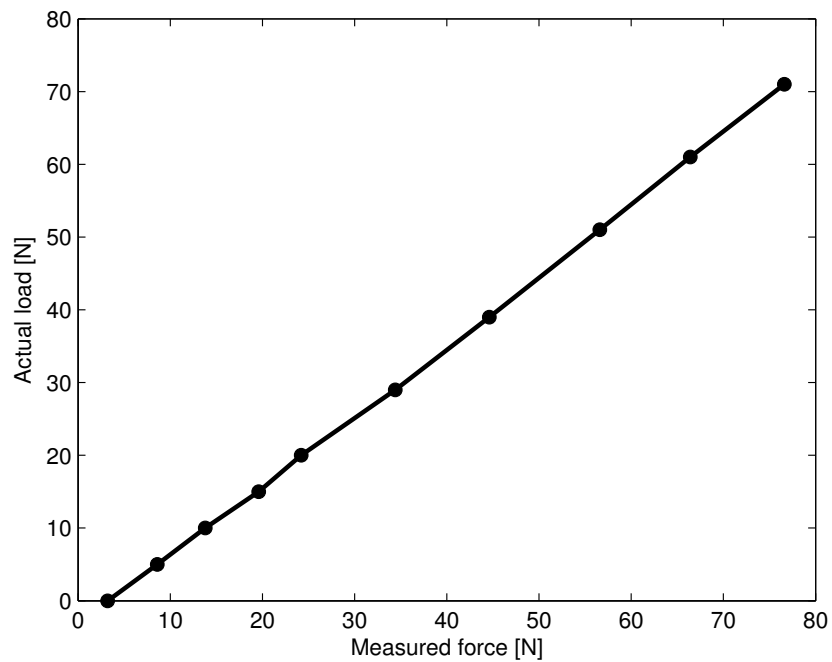


Figure F.1: Calibration measurements of the PEETRA system

Appendix G

Star CCM+ setup

In the following pages is a note written by Hanne Nøvik of the setup of the numerical model in Star CCM+.

Note

Written by: Hanne Nøvik

Date: 2012-05-23

Ownership: Department of Water and Environmental Engineering, NTNU

Distribution: Bjørnar Rettedal and Lars Eid Nielsen

TOPIC: BED SHEAR STRESS CALCULATION – STAR CCM+ CFD MODEL

The following note describes the model setup and the calculated bed shear stress results from two computational fluid dynamic (CFD) models conducted with the commercial program Star CCM+.

Geometry

Length: 1000 mm

Width: 150 mm

Slope: 1:100

Height: A) 370 mm and B) 390 mm

Gate opening: A) 25 mm and B) 36 mm

Gate 'thickness': 20 mm

Grid

Trimmed cells – (rectangular cells).

Minimum cell size: 2.5 mm

Maximum cell size: 25 mm

Target cell size: 5mm

Total number of cells: A) 75696 and B) 77396

Boundary conditions

Inlet velocity: A) Evenly distributed: $V = 0.10656$ m/s, giving $Q = 5,9$ l/s

B) Evenly distributed: $V = 0.1292$ m/s, giving $Q = 7,54$ l/s

Outlet velocity: A) Evenly distributed: $V = -1,577$ m/s,

B) Evenly distributed: $V = -1.39975$ m/s,

Top: Symmetry plan

Walls: Wall, Shear stress: No-slip, Wall surface specification: Smooth

Bottom: Wall, Shear stress: No-slip, Wall surface specification: roughness = 0.25mm

Solver

Three-dimensional solution of Reynolds-Averaged Navier-Stokes equation

Implicit unsteady 1st order temporal discretization

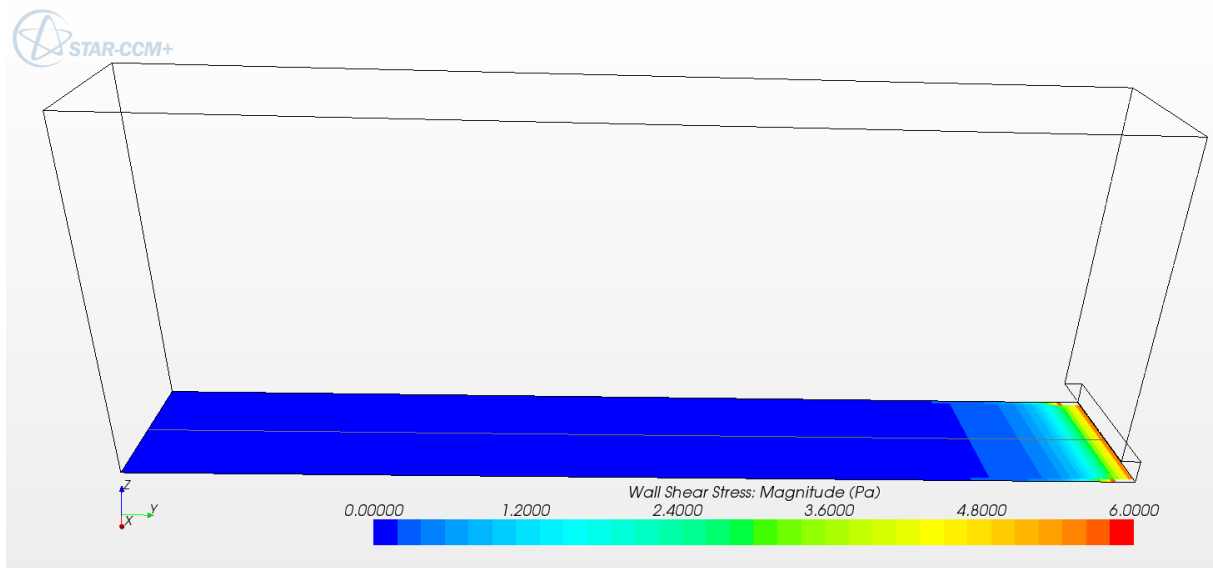
Timestep. 0.002 s

Realizable K-ε turbulence model, two-layer

Liquid phase (not free-surface flow)

Results – bed shear stress

Plot from Star CCM +



Figur 1 Calculated Bed shear stress B) h=370mm, gate opening= 25 mm, r=0.25 mm, Q=5,9 l/s

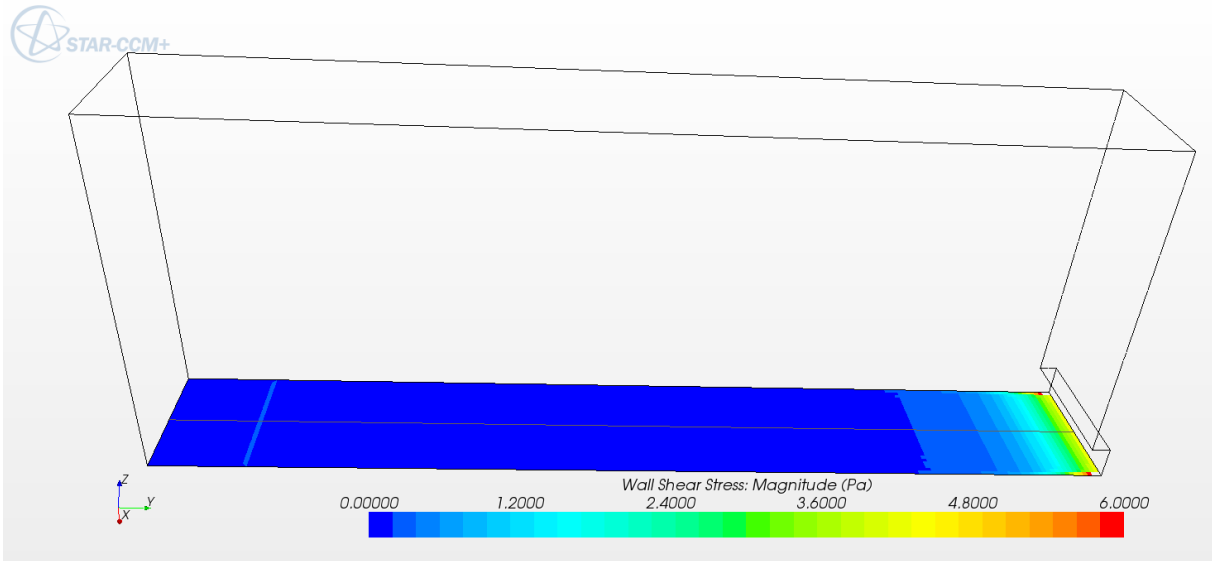


Figure 2 Calculated Bed shear stress B) $h=390\text{mm}$, gate opening= 36mm $r=0.25\text{ mm}$, $Q=7,54\text{ l/s}$

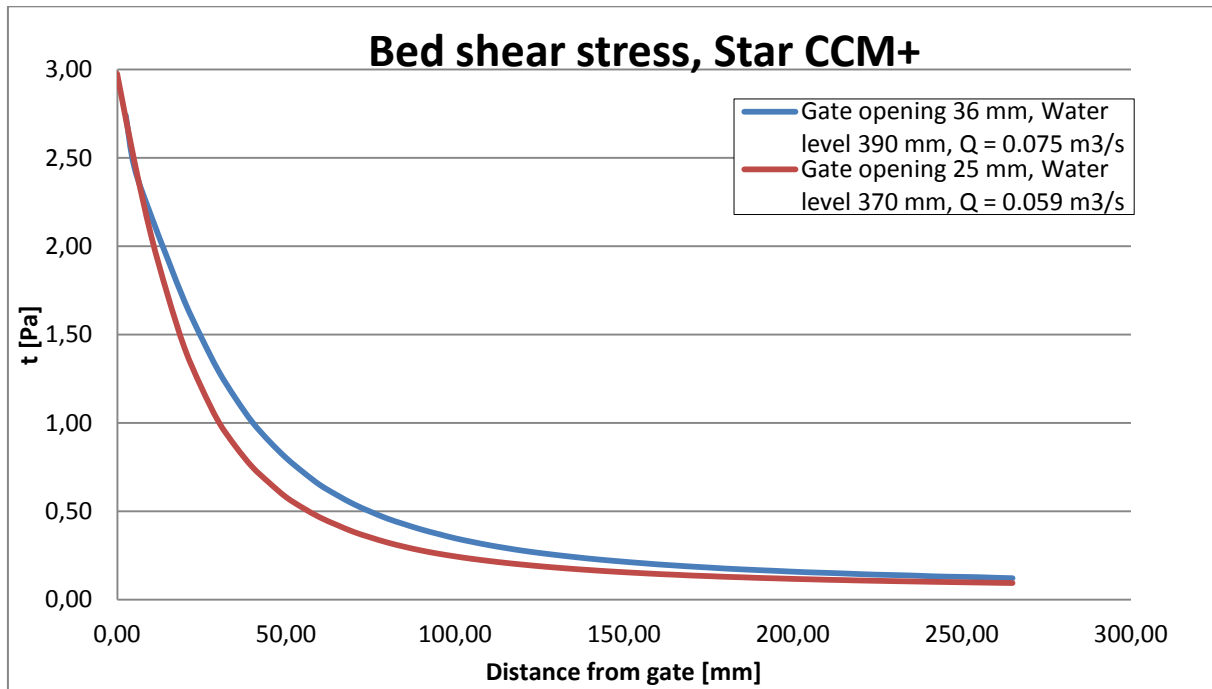


Figure 3 Calculated bed shear stress at different distance from the flushing gate

Appendix H

Test procedure for the flushing flume

1. Set the regulator to the minimum frequency (38 Hz) which gives $Q = 0.25$ l/s
2. Close the gate, and let the water fill up the tank and flow over the gate
3. Put in the fixed amount of sediments and spread it evenly out
4. Gradually increase the water discharge, while gradually lifting the gate, always keeping the water level in line with the top of the gate
5. The first experiment starts when the gate opening is big enough so that water level is in line with the top of the gate for the correct discharge
6. Note down the flushing length
7. Open the gate with approximately 5 mm and note down the flushing length
8. Let the water level stabilize
9. Open the gate more and note down the flushing length
10. Repeat point 7 to 9 till flushing stops
11. Note down the water level when the initial movement of the bed starts