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# Stabilitet av erosjonssikring med stein på elvebredder under overkritisk strømning – effekten av steinplassering

Stability of riverbank riprap in supercritical flow conditions – effect of stone placement Norges teknisk-naturvitenskapelige universitet - NTNU



# NVE Ekstern rapport nr. 21/2020

# Stabilitet av erosjonssikring med stein på elvebredder under overkritisk strømning – effekten av steinplassering

Stability of riverbank riprap in supercritical flow conditions - effect of stone placement

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Sammendrag:	Rapporten beskriver stabiliteten av erosjonssikring med stein på elvebredder. Stabiliteten ble testet under overkritisk strømning med fysiske modellforsøk i en målestokk på 1:10. Måten å plassere stein på ble variert. I forsøkene tålte plastring med lengste akse plassert normalt på strømningsretningen de høyeste belastningene med Froude tall mellom 2,1-3,0. Plastring med lengste akse parallelt med strømningsretningen var svakest og tålte Froude tall på 1,3-1,7. Den var dermed litt svakere enn når steinene ble vilkårlig dumpet som rauset steinsikring som tålte en belasting med Froude tall 1,6-2,0.
Emneord:	Erosjonssikring av stein, rauset steinsikring, plastring, modellforsøk

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# Forord

Dette er en forskningsrapport utarbeidet av institutt for bygg- og miljøteknikk på Norges teknisk-naturvitenskapelige universitet – NTNU. Modellforsøkene det vises til i rapporten, ble kjørt på vassdragslaboratoriet på NTNU.

Dimensjonering av erosjonssikring av stein er en utfordrende oppgave innenfor vassdragsteknikk. Både framgangsmåten for å dimensjonere og de hydrauliske prosessene bak, er beskrevet i <u>Vassdragshåndboka</u> fra 1998, andre utgave 2010. NVE veileder 4-2009 <u>Veileder for dimensjonering av erosjonssikringer av stein</u> utdyper dimensjoneringen ytterligere. Modellforsøkene i denne forskningsrapporten er et steg videre for å forstå og forbedre erosjonssikring av stein. Fokuset har vært på erosjonssikring i sideskråninger som blir belastet med overkritisk strømning. Dette er relevant fordi NVE ser utfordringer med dimensjonering og skadereduksjon for eksisterende erosjonssikringer langs veldig bratte vassdrag. Resultatene vil også bli inkludert i <u>Sikringshåndboka</u> som er en digital veileder om sikringstiltak mot flom og skred.

NTNU har gjennomført prosjektet for NVE. På NTNU har Michal Pavlíček, Leif Lia og Oddbjørn Bruland utført oppdraget med hjelp av Wolfgang Szentkereszty, Bibek Karki, Mahmoud Omer Awadallah og Geir Tesaker i vassdragslaboratoriet. NVEs prosjektgruppe har bestått av Paul Christen Røhr, Eirik Traae, Anders Jarle Muldsvor, Mads Eirik Hugo Johnsen og Priska Helene Hiller. Rapporten er skrevet av NTNU som står ansvarlig for konklusjonene. Rapporten er på engelsk med sammendrag på norsk og engelsk.

NVE takker NTNU for godt samarbeid.

Oslo, 15.10.2020

ann. Krister Larsen

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## STABILITET AV EROSJONSSIKRING MED STEIN PÅ ELVEBREDDER UNDER OVERKRITISK STRØMNING – EFFEKTEN AV STEINPLASSERING

#### SAMMENDRAG

Målet med forsøkene var å undersøke stabiliteten av plassert- og dumpet stein i plastring av elvebredder i overkritisk strømning. Resultatene ble sammenlignet med eksisterende formelverk for erosjonssikring. Forsøkene ble kjørt i skala 1:10. En glideluke med understrømning i en 1,0 m bred horisontal renne ble brukt for å skape overkritisk strømning. Froude-tallet ble brukt som felles parameter for strømning i bratte elver og strømning i den horisontale renna nedstrøms luka. Elvebredden (dvs. et halvt trapesformet tverrsnitt) med sideskråning 1: 1,5 (33,7 °) ble bygget oppstrøms- og nedstrøms luka med hjelp av en trekonstruksjon. Erosjonssikringen besto av ensartede steiner på d<sub>50</sub> = 0,057 m, og filterlaget under erosjonssikringen ble bygget med ensartede steiner på d<sub>50</sub> = 0,020 m. Tre typer plastring ble testet: i) dumpet stein ('rauset steinsikring'), ii) plassert plastring parallelt ('flatplastring', den lengste aksen til steinene er parallell med strømningsretningen) og iii) plassert plastring normalt ('damplastring', steinens lengste akse mot sidehellingen/skråningen).

Vannføringen, nivået og hastigheten på vannoverflaten og punkthastigheten ble målt i modellen. Nivået på vannoverflaten ble målt ved hjelp av ultralydsensorer. Målinger av vannoverflatehastighet ble utført ved bruk av bordtennisballer og kamera. Punkthastighet ble målt i flere punkter langs et tverrsnitt ved bruk av ADV (dvs. Akustisk Doppler Velocimetry) sideprober og propell (dvs. strømmåler). To videokamera (dvs. sideveis og ovenfra) ble brukt for å identifisere når- og hvordan erosjonssikringen gikk til brudd.

Forsøkene inkluderte innledende tester, stabilitetsforsøk for erosjonssikring og målinger av punkthastighet. For hvert forsøk med erosjonssikring ble steinene plassert/dumpet i modellen. Deretter ble vannføringen økt trinnvis til erosjonssikringen sviktet. Årsaken til at punkthastighetsforsøkene ble utført separat, var at både ADV og propellmålinger kan påvirke forholdene nedstrøms probene (dvs. påvirke stabiliteten til plastringen).

*Kritiske* forhold for erosjonssikringen (dvs. forhold som forårsaker bruddet) er satt til når feilen initieres (dvs. øyeblikket hvor kontinuerlig erosjon av steinene begynner og fører til total svikt). Total svikt regnes når erosjonssikringen glir ned fra skråningen og filteret blir eksponert langs hele den nedsenkede delen av skråningen.

Resultatene for kritiske forhold for plastringen (dvs. vannføring og Froude-tall) viser tydelig at plassert plastring normalt er den mest stabile plastringen. Den tåler strømning med mye høyere Froude-tall (dvs. 2,1 – 3,0 med vannføring  $Q_{lab} = 450$  l/s, tilsvarande Q = 140 m<sup>3</sup>/s i fullskala, enn de andre utformingene. Plassert plastring parallelt er den minst stabile plastringen med Froude-tall på 1,3 – 1,7 med vannføring på  $Q_{lab} = 250 - 275$  l/s, Q = 80 - 90 m<sup>3</sup>/s i fullskala. Dumpet erosjonssikring er litt mer stabil enn plassert plastring parallelt med Froude-tall mellom 1,6 – 2,0 med  $Q_{lab} = 275 - 300$  l/s, 90 – 100 m<sup>3</sup>/s i fullskala. I fullskala (1:10) gjelder verdiene for steindiameter på d = 0.57 m.

Når det gjelder hastighetsmålinger, gir punkthastigheter målt med ADV og Propell rimelig form på hastighetsprofilene. ADV ser imidlertid ut til å undervurdere punkthastighetsverdiene i forhold til Propell. Det ble utført sammenligning av gjennomsnittshastighet oppnådd med alle metodene (dvs. ADV, Propell, overflatehastighet, ultralydsensorer). Fra sammenligningen er det klart at bruk av Propell gir best samsvar med gjennomsnittshastighet fra ultralydsensorer. ADV undervurderer vannhastigheten og målingene av hastighet på overflaten overvurderer den reelle vannhastigheten.

Evaluering av Robinsons formel for plastring i bratte elver ble utført. Steindiameter i forhold til Robinsons formel ble bestemt for kritiske strømningsforhold for alle forsøkene. Sammenligning av de beregnede

verdiene på steindiameter som ble brukt i forsøkene, indikerer at Robinsons formel gir større nødvendig steindiameter i alle tilfeller.

# STABILITY OF RIVERBANK RIPRAP IN SUPERCRITICAL FLOW CONDITIONS – EFFECT OF STONE PLACEMENT

#### PROJECT SUMMARY

The goal of the experiment is to investigate stability of placed and dumped riverbank riprap in the supercritical flow and the comparison of the results with existing riprap design formulas. The conceptual scale of the model is 1:10. A sluice gate in 1 m wide horizontal flume was used to generate supercritical flow. Froude number was used as a common parameter for flow in steep rivers and flow in the horizontal flume downstream a sluice gate. Riverbank (i.e. half trapezoidal cross section) with side slope 1:1.5 (33.7°) was built upstream and downstream the gate using a wooden structure. The riprap consisted of uniform stones of  $d_{50}$ =0.057 m and the filter layer below riprap was built with uniform stones of  $d_{50}$ =0.020 m. Three types of riprap configuration were tested: i) dumped riprap ('rauset steinsikring'), ii) placed riprap parallel ('flatplastring', the longest axis of the stones is parallel to the flow direction), iii) placed riprap inclined ('damplastring', the longest axis of the stones is towards the side slope/bed).

The discharge, water surface elevation, water surface velocity and point velocity were measured on the model. Water surface elevation was measured by ultrasonic sensors. Measurements of water surface velocity were carried out using table tennis balls and camera. Point velocity was measured in several points along a cross section using ADV (i.e. Acoustic Doppler Velocimetry) side probes and Propeller (i.e. current meter). In order to identify the riprap failure, video documentation with 2 cameras (i.e. side and top view) was recorded.

The experiments included preliminary tests, riprap stability experiments and point velocity measurement experiments. For each riprap stability experiment, the riprap stones were placed/dumped to the model and then the discharge was increased stepwise until the failure of riprap occurred. The reason of separation of the point velocity experiments is that both ADV and Propeller measurements are intrusive methods and can influence the flow conditions downstream the probes (i.e. effect the riprap stability).

Critical flow conditions (i.e. conditions which caused the failure) were obtained for failure initiation stage (i.e. the moment when continuous erosion of the riprap stones begins and leads to the total failure). Total failure occurs when riprap stones slide down from the bank and the filter is exposed along the whole submerged height of the bank.

From the results of critical flow conditions (i.e. discharge and Froude number) is clear that placed riprap inclined is the most stable riprap configuration and it can withstand much higher Froude number (i.e. ca. 2.1 to 2.95 with discharge of ca. 450 l/s, ca. 142 m<sup>3</sup>/s in prototype) than the other configurations. Placed riprap parallel is the least stable configuration with the range of Froude number ca. 1.3 to 1.7 with discharge of ca. 250 to 275 l/s (ca. 79 to 87 m<sup>3</sup>/s in prototype). Dumped riprap is a bit more stable than placed riprap parallel with Froude number range ca 1.6 to 2.0 with discharge of ca. 275 to 300 l/s (ca. 87 to 95 m<sup>3</sup>/s in prototype). The stones diameter in prototype (1:10) is 0.57 m.

Regarding velocity measurements, point velocities measured by ADV and Propeller give reasonable shapes of velocity profiles. However, ADV seems to underestimate the point velocity values in comparison to Propeller. Comparison of mean velocity obtained from all methods was done (i.e. ADV, Propeller, surface velocity, ultrasonic sensors). From the comparison it is clear, that use of Propeller gives the best match with mean velocity from ultrasonic sensors, ADV underestimates the values and the values obtained from surface velocity measurements are overestimated.

Evaluation using Robinson's formula for riprap design in steep rivers was performed. Stone diameter according to Robinson's formula was determined for critical flow conditions for all riprap stability

experiments. Comparison of the designed values with stone diameters used in the experiment indicates that Robinson's formula overestimates the stone diameter in all cases.



Department of civil and environmental engineering

Date O 02.07.2020 L

Our reference Leif Lia

# Memo To: NVE, Region Midt-Norge/ Priska Helene Hiller, Mads Johnsen

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From: Michal Pavlíček, Leif Lia, Oddbjørn Bruland

Signature:

# STABILITY OF RIVERBANK RIPRAP IN SUPERCRITICAL FLOW CONDITIONS – EFFECT OF STONE PLACEMENT

## (STABILITET AV EROSJONSSIKRING MED STEIN PÅ ELVEBREDDER UNDER OVERKRITISK STRØMNING – EFFEKTEN AV STEINPLASSERING)

## FINAL REPORT



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The literature review on the topic was done and can be seen in Appendix 5.

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The goal of the experiment is to investigate stability of placed and dumped riverbank riprap in the supercritical flow and the comparison of the results with existing riprap design formulas. A sluice gate in a horizontal flume was used to generate supercritical flow. Different types of riprap placement/dumping were tested. The discharge, water surface elevation and point velocity were measured on the model. The conceptual scale of the model is 1:10. The riprap consisted of uniform stones of  $d_{50}$ =0.057 m. The experience gained from this experiment can be applied on a more detailed experiment on the same topic.

## 2 MODEL GEOMETRY

Experiments were performed in the hydraulic laboratory at NTNU. The horizontal flume is 25 m long, 2 m high and 1 m wide. The maximum discharge on the inlet is theoretically 0.6 m<sup>3</sup>/s, but for practical purposes 0.50 m<sup>3</sup>/s. The discharge is regulated by manual valves. The sluice gate was installed in upstream part of the flume. The model scheme can be seen in Figure 1 and Figure 2. The sluice gate was operated by a manual crank. The gate opening was fixed on 230 mm (i.e. distance from the upstream bed wooden plate to the gate edge), the opening dimensions can be seen in Figure 2. Riverbank (i.e. half trapezoidal cross section) with side slope 1:1.5 (33.7°) was built upstream and downstream the gate using a wooden structure, the dimension of the bank can be seen in Figure 2. The riverbank downstream the gate consisted of the fixed riprap and the test section with a mobile riprap placed on the layer of filter and metal grid (see Figure 3). The wooden structure was fixed by screws to the concrete bottom of the flume (see Figure 4). The structure was sealed on the upstream end and on the sides, the downstream end was kept open in order to allow the tailwater to fill the structure (see Figure 4). For the purpose of avoiding the side flow from the gate, which would influence the flow conditions in the test section, the inclined part of the opening was closed and sealed with a wooden plate and guiding walls were installed upstream (see Figure 5) and downstream of the gate (see Figure 6). A metal sieve was installed in the outlet of the flume to catch the stones (see Figure 7). The position of the camera while taking the pictures in Figure 3 to Figure 6 is marked in Figure 1.



Figure 1 Longitudinal section and plan view of the model (water flows from left to right), dimension in mm)



Figure 2 Cross sections of the model and the gate opening, dimensions in mm



Figure 3 The final set-up of the model with the test section without filter and riprap stones



Figure 4 Downstream end of the model with the supports to the concrete bed of the flume



Figure 5 Upstream guiding wall and closing of the inclined gate opening

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Figure 6 Guiding wall downstream the gate



Figure 7 The outlet of the flume with metal grid

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#### 3 ANALOGY BETWEEN FLOW IN STEEP RIVERS AND FLOW DOWNSTREAM A SLUICE GATE

To use the measurements from the flow in the horizontal flume to determine the stone size in steep rivers, the analogy between the phenomena have to be found. Froude number (1) will be used as a common parameter for flow in steep rivers (i.e. slope) and flow in the horizontal flume downstream a sluice gate (i.e. gate).

$$Fr = \frac{v_{slope}}{\left(g \cdot \frac{A_{slope}}{T_{slope}}\right)^{0.5}} = \frac{v_{gate}}{\left(g \cdot \frac{A_{gate}}{T_{gate}}\right)^{0.5}}$$
(1)

where  $v_{slope}$  and  $v_{gate}$  are mean flow velocities in the steep river and downstream sluice gate,  $A_{slope}$  and  $A_{gate}$ are the flow areas,  $T_{slope}$  and  $T_{gate}$  are free surface widths and g is the gravitational acceleration.

In order to obtain longitudinal slope corresponding to flow conditions downstream sluice gate, Manning's formula was used for the test section geometry and the slope was found to match the values of mean velocity, water depth and therefore Froude number measured in the lab.

Formula according to (C. E. Rice K. C. Kadavy, 1999) was used for Manning's roughness coefficient (2).

$$n = 0.029 (d_{50}s)^{0.147}$$

Where *n* is Manning's roughness coefficient,  $d_{50}$  median stone diameter and *s* bed slope.

Froude number in the rivers with longitudinal slope in the range from 1:50 - 1:10 (i.e. 2 to 10 %) could range between 1 and 3. The sluice gate is used to generate supercritical flow conditions in the flume. Since the slope range is up to 10 %, the effect of the different direction of the forces acting on the riprap stones in the set-up with horizontal or tilted flume is neglected.

#### 4 SCALING

Froude's model law was used for scaling the model (3), (4), (5), (6), (7) (Lysne, 2003).

$$\frac{v_p}{(gL_p)^{0.5}} = \frac{v_m}{(gL_m)^{0.5}}$$
(3)

$$L_r = \frac{L_m}{L_p} \tag{4}$$

$$V_r = \sqrt{L_r} \tag{5}$$

$$T_r = \sqrt{L_r} \tag{6}$$

$$Q_r = L_r^{\frac{5}{2}} \tag{7}$$

where  $v_p$  and  $v_m$  are the prototype and the model flow velocity,  $L_p$  and  $L_m$  are lengths in prototype and model, g is the gravitational acceleration,  $L_r$  geometric scale,  $V_r$  velocity scale and  $T_r$  time scale.

The conceptual scale was set to 1:10.

(2)

Date

#### 5 **STONE SIZE**

As can be seen in Figure 2, the test section consisted of the metal grid, layer of filter and mobile riprap stones.

#### 5.1 Mobile riprap stones (test section)

Mobile riprap stone size has to be designed in the way that the failure will occur in desired flow conditions (i.e. Froude number in the range ca. between 1 and 3). (Jafarnejad, 2016) conducted an experiment with similar set-up and flow conditions. She used uniform stones of  $d_{50}=37-47$  mm. Her results are used as an outline to design the mobile riprap stones diameter in this experiment.

Based on the outline from (Jafarnejad, 2016), uniform natural stones of  $d_{50}=57$ mm were chosen for the initial tests. The conclusion of these tests was, that the failure occurs for all riprap dumping/placements and the stones can be used to perform the experiment.

The mobile riprap stones characteristics were taken from (Hiller, 2017) and can be seen in Table 1. The stones were classified as uniform  $(d_{60}/d_{10}=1.17)$ , angular to subangular and slightly oblong (a/b = 1.17).

Table 1 Riprap stones parameters						
$d_{min}$ [mm]	<i>d</i> <sub>15</sub> [mm]	<i>d</i> <sub>50</sub> [mm]	<i>d</i> <sub>65</sub> [mm]	$d_{max}$ [mm]	$\rho_s [kg/m^3]$	
41	52	57	60	74	2710	

#### 5.2 **Filter design**

The filter was designed according to NVE guidelines (NVE, 2009). The grain size distribution curve of the filter stones (Figure 8) was determined using the same procedure as in (Hiller, 2017). 100 stones were measured with calliper (a, b and c axis were measured). The nominal diameter d were used (8) (Bunte & Abt, 2001) and the mass of each stone was estimated using formula (9) with  $C_f = 0.6$ , (NVE, 2012). The filter stones were classified as uniform  $(d_{60}/d_{10}=1.56)$ , (11), (NVE, 2009).

$$d = (abc)^{\frac{1}{3}} \tag{8}$$

$$d = \left(\frac{m}{C_f \rho_s}\right)^{\frac{1}{3}} \tag{9}$$

where d is nominal diameter, m stone mass,  $C_f$  stone shape factor and  $\rho_s$  stone density.

The density of the stones was obtained by weighing the stones in the container with known volume. Immersed and non-immersed stones were weighted and based on volume and mass values the density was calculated. Filter stone parameters can be seen in Table 2.

Table 2 Filter stones parameters					
<i>d</i> <sub>10</sub> [mm]	<i>d</i> 15 [mm]	<i>d</i> <sub>50</sub> [mm]	<i>d</i> <sub>60</sub> [mm]	<i>d</i> <sub>85</sub> [mm]	$\rho_s [kg/m^3]$
13.7	14.7	20.4	21.4	30.5	3042

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Figure 8 Filter stones grain size distribution curve

The filter stones size fulfilled the criteria from NVE guidelines for river riprap design (10), (11), (12), (NVE, 2009).

$$\frac{d_{15,riprap}}{d_{15,filter}} = 3.5 > 1.5 \tag{10}$$

$$\frac{d_{15,riprap}}{d_{85,filter}} = 1.7 < 5 \tag{11}$$

$$\frac{d_{60,filter}}{d_{10,filter}} = 1.56 < 10 \tag{12}$$

The minimum thickness of the filter layer was 100 mm on the bed and 90 mm on the bank (based on the riprap type, see chapter 6), which also corresponds to the criteria in (NVE, 2009), (13).

 $90(100) \text{ mm} > 4 \, d_{50} = 81.6 \text{ mm} \tag{13}$ 

#### 5.3 Metal grid

The metal grid was used to create a rough surface below the filter stones. The grid was fixed to the flume bottom and the wooden plate on the riverbank (see Figure 9).

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Figure 9 Metal grid below the filter stones

#### 5.4 Fixed stones

The purpose of the fixed stones is to create a rough surface on the river bed and bank (i.e. create more natural conditions at the test section with similar turbulence due to bed roughness upstream the test section as within the test section) and to prevent erosion in the zone with too high Froude numbers (i.e. close to the gate). Therefore, the requirements on the size and the uniformity of the material are not as strict as for the mobile riprap and filter. The stones have been chosen by hand from the stones which are not used in the other ongoing experiment. Stone size is ca. from 26 to 63 mm (i.e. the length of intermediate stone axis). The stones were glued to the model structure. The example of the fixed stones can be seen in Figure 10.



Figure 10 Example of the fixed stones

### 6 **RIPRAP PLACEMENT**

As required by NVE, three types of riprap configuration were tested: i) dumped riprap ('rauset steinsikring'), ii) placed riprap parallel ('flatplastring', the longest axis of the stones is parallel to the flow direction), iii) placed riprap inclined ('damplastring', the longest axis of the stones is towards the side slope/bed). The ripraps for all scenarios were built by the same person. The stone colours in the pictures have no relevance.

#### 6.1 Filter

The filter stones were dumped to the test section and aligned using a shovel and by hand. Due to the different thickness of each riprap layer, the thickness of the filter layer was 100 mm on the bed and 90 mm on the bank for dumped riprap and placed riprap inclined and 150 mm for the bed and 140 mm for the bank for placed riprap parallel. Figure 11 and Figure 12 show examples of the filter layer.



Figure 11 Example of the filter layer for dumped riprap (D) and placed riprap inclined (T)



Figure 12 Example of the filter layer for placed riprap parallel (P)

### 6.2 Dumped riprap (D)

Dumped riprap was randomly placed by hand (3-5 stones at once) from downstream to upstream of the test section without interlocking between stones. Dumped riprap consisted of 468 stones in average of all dumped riprap experiments. The thickness of the riprap layer was 90 mm, which is ca. 1.5  $d_{50}$  and fulfils the requirements in (NVE, 2009). The examples can be seen in Figure 13, Figure 14 and Figure 15.

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Figure 13 Dumped riprap (D04)



Figure 14 Dumped riprap (D02)

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Figure 15 Dumped riprap (D03)

#### 6.3 Placed riprap parallel (P)

Placed riprap parallel stones were placed one by one from the ladder to the test section from downstream to upstream. The orientation of the stones was with the longest axis parallel to the flow direction. Interlocking pattern was made between the stones. Once the stone was randomly picked from the bucket, it was placed to the riprap, putting stones back and picking the one with better shape was not allowed during the construction. This rule was introduced in order to prevent building of more resistant riprap that can be done in practice by using excavator. Placed riprap parallel consisted of 344 stones in average of all placed riprap parallel experiments. The thickness of the riprap layer was 40 mm, which corresponds to the height of lying stones (i.e. b or c axis). The examples can be seen in Figure 16, Figure 17 and Figure 18.



Figure 16 Placed riprap parallel (P01)

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Figure 17 Placed riprap parallel (P03)



Figure 18 Placed riprap parallel (P04)

#### 6.4 Placed riprap inclined (T)

The same procedure and rules as for placed riprap parallel were used for placed riprap inclined, but the orientation of the stones was with the longest axis towards (perpendicular) to the bed or bank. The riprap consisted of 556 stones in average of all placed riprap inclined experiments. The thickness of the riprap layer was 90 mm, which corresponds the height of standing stones (i.e. *a* axis). The examples can be seen in Figure 19, Figure 20 and Figure 21.



Figure 19 Placed riprap inclined (T05)



Figure 20 Placed riprap inclined (T04)



Figure 21 Placed riprap inclined (T03)

#### 6.5 Packing density

Packing density could be appropriate parameter to describe the quality of different riprap configurations. To quantify packing density, packing factor  $P_c$  was introduced by (Olivier, 1967), (14). Low packing factor indicates high packing density (Hiller, 2017).

$$P_c = \frac{1}{Nd_s^2} \tag{14}$$

where N is amount of stones per m<sup>2</sup> and  $d_s^2$  area of the average stone assuming cubical shape of stones.

The average value of packing factor for placed riprap inclined in this experiment is  $P_c = 0.49$ . (Hiller, 2017) used the same stones, orientation of stones and construction rules for riprap on downstream slope of rockfill dam. The average values of eight of her experiments was  $P_c=0.53$ . Comparison of those two  $P_c$  values shows, that the packing density of the placed riprap for both experiments is very similar.

However, packing factor  $P_c$  allows comparison of the packing density of ripraps with different stone sizes, but with the same stone orientation. Since the different riprap configurations in this experiment have a different thickness and orientation of stones, volumetric packing factor  $P_{cv}$  has to be used to describe packing density (15).  $P_{cv}$  was also introduced by (Olivier, 1967).

$$P_{cv} = \frac{1}{N_v d_s^3} \tag{15}$$

where  $N_v$  is amount of stones per m<sup>3</sup> and  $d_s^3$  volume of the average stone assuming cubical shape of stones.

Average values of  $P_{cv}$  for all configurations can be seen in Table 3. The riprap thickness of 40 mm for placed riprap parallel and 90 mm for dumped riprap and placed riprap inclined were assumed. From the table it is clear, that the packing density is highest for placed riprap parallel and lowest for dumped riprap, which could be also evaluated by looking at Figure 22, where the highest packing density can be seen for placed riprap parallel (center).

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exp		Number of	P <sub>cv</sub>
		stones	[-]
	$\mathbf{D}_{\mathrm{avg}}$	468	0.93
	$\mathbf{P}_{avg}$	344	0.56
	$T_{avg}$	556	0.78

Table 3 Average values of volumetric packing factor  $P_{cv}$  for all riprap configurations



Figure 22 Top view of dumped riprap (left), placed riprap parallel (center) and placed riprap inclined (right)

### 7 EXPERIMENT SCENARIOS

In order to avoid damage of point velocity probes (i.e. ADV, Propeller) and preliminary testing of the model, the experiments were divided into three groups: i) preliminary experiments, ii) riprap stability experiments with only ultrasonic sensors in the test section and ii) point velocity measurement experiments with ADV probes and the propeller upstream the test section and ultrasonic sensors.

### 7.1 **Preliminary experiments**

Preliminary tests (F01-03) were carried out to observe the flow conditions, test the ultrasonic and discharge sensors and estimate the critical values for the failure for dumped riprap and placed riprap inclined.

For each experiment, the riprap stones were placed/dumped to the test section and then the discharge was increased stepwise until the failure of riprap occurred. The initial discharge was 200 l/s, then the discharge was increased with steps of 25 l/s every 15 minutes.

#### 7.2 Riprap stability experiments

The procedure was the same as for preliminary tests, but the discharge was increased every 50 minutes with the step of 25 l/s. Time interval of 6 minutes was chosen to achieve the initial discharge 200 l/s from 0 l/s at the start of each experiment.

Three experiments for each riprap configuration were performed with two ultrasonic sensors in the test section (D01-03, P01-03, T01-03) and one experiment each with one ultrasonic sensor in the test section (D04, P04, T04). One extra experiment was done for placed riprap inclined (T05).

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### 7.3 Point velocity measurement experiments

The reason of separation of the point velocity experiments is that both ADV and Propeller measurements are intrusive methods and can influence the flow conditions downstream the probes (i.e. effect the riprap stability).

For these experiments (ADV01-07, Propeller), placed riprap inclined was built in the test section. The initial discharge was 100 l/s, then 200 l/s and next the discharge was step-wise increased with the step of 25 l/s until the probe parameters gave reasonable values. For each discharge the time interval of 5 minutes was taken for ADV measurements and 1 minute for propeller measurements. See chapters 8.4 for detailed description of the point velocity measurements.

To get similar flow conditions upstream the test section for all experiments the gate opening was fixed on 230 mm (i.e. distance from the upstream bed wooden plate to the gate edge), see Figure 2. Table 4 shows the experiments schedule.

Date	Experiment	Riprap	Downstream ultrasonic sensors
11/12/2019	F-01	dumped riprap	2
12/12/2019	F-02	placed riprap inclined	2
12/12/2019	F-03	dumped riprap	2
15/12/2019	D-01	dumped riprap	2
17/12/2019	P-01	placed riprap parallel	2
18/12/2019	T-01	placed riprap inclined	2
04/01/2020	D-02	dumped riprap	2
05/01/2020	P-02	placed riprap parallel	2
06/01/2020	P-03	placed riprap parallel	2
08/01/2020	D-03	dumped riprap	2
09/01/2020	T-02	placed riprap inclined	2
11/01/2020	T-03	placed riprap inclined	2
12/01/2020	D-04	dumped riprap	1
13/01/2020	P-04	placed riprap parallel	1
14/01/2020	T-04	placed riprap inclined	1
15/01/2020	T-05	placed riprap inclined	2
18/01/2020	ADV 01-04	placed riprap inclined	2
19/01/2020	ADV 05-07	placed riprap inclined	2
19/01/2020	Propeller	placed riprap inclined	2

Table 4 Exp	periments	schedule
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### 8 MEASURED VARIABLES

#### 8.1 Discharge

The discharge on the inlet was measured with Siemens Sitrans Mag 5000 discharge meters.

#### 8.2 Sluice gate opening

The ruler fixed to the reference point and a measure tape were used to measure the gate opening.

#### 8.3 Water surface level

The water surface level upstream (i.e. upstream sensor) and downstream (i.e. upper and lower downstream sensor) the gate was measured with ultrasonic sensors (Microsonic mic +340). The upstream sensor was placed ca. 0.8 m upstream of the sluice gate in the flume centre. For the different scenarios, one or two downstream sensors were used in the test section (see Figure 23). The sensors were mounted on the metal traverse fixed to the top of the flume, as can be seen in Figure 24. For experiment T05, the measurements of the lower downstream sensor were influence by placing the sensor too close to the bank, so the results from this sensor were excluded from the further results analysis.

Paper scales were fixed on the window outside the flume to estimate the water depth during the experiments. The origin of the scales was fixed to the top of the wooden plate on the bed. The scales can be seen in Figure 40.



Figure 23 Plan view of the ultrasonic sensor positions

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Figure 24 Mounting of ultrasonic sensors (ADV01-07, Propeller)

#### 8.3.1 Discharge and upstream water depth

The data from discharge and ultrasonic sensors were processed using R programming language and MS Excel.

The water depth upstream the gate was calculated from water surface level (i.e. the distance from the ultrasonic sensor to the water surface) and the flume bed level (i.e. the distance from the concrete flume bed to the sensor). Discharge and upstream water depth in time is plotted in Figure 25. As mentioned in chapter 7.2, each discharge was kept for 50 minutes for riprap stability tests. However, it takes some time to reach the constant discharge along the whole model. Stabilization of upstream depth was used as a criterion for constant discharge (i.e. when the upstream water depth is constant, constant discharge in the test section is assumed). Figure 26 presents 10 minutes interval, where the increasing of the discharge from 275 to 300 l/s can be seen. The increasing starts after 205 minutes and from the graph is clear that the upstream water depth is stabilized 5 minutes later (i.e. 210 min). Hence, the time interval for constant discharge 300 l/s is 45 minutes. The analysis of the stabilization time of the upstream water depth was done for all riprap stability experiments and the results of critical time intervals are shown in Table 7. For further results analysis the average discharge value from the constant discharge time interval was taken.





Figure 25 Upstream water depth and discharge in time (T02)



Figure 26 Upstream water depth and discharge in time for increasing the discharge from 275 to 300 l/s (T02)

#### 8.3.2 Water surface level in the test section

The operational range of Mic +340 ultrasonic sensors in the test section (ca. 2 m from the sensor) was a circle with the diameter of 400 mm (Microsonic, 2020). Therefore, in the experiments with two sensors, they influence each other which resulted in spikes in the data. According to the constant volume time interval obtained in chapter 8.3.1, the data was filtered and de-spiked using R programming de-spike function. An example of de-spiked data for upper downstream sensor can be seen in Figure 27.

To evaluate the influence of the sensors to each other, one round of experiments was performed just with one sensor in the test section (i.e. D04, P04, T04), where no spikes were observed.



Figure 27 Example of de-spiking of the ultrasonic sensors data (D01, Q = 225 l/s)

For further results analysis the average sensor distance (i.e. distance from the sensor to the water surface) value from the constant discharge time interval was taken. For the critical discharge (see chapter 8.7), the average values for critical time interval was used.

#### 8.3.3 Bed level in the test section

The distance from the ultrasonic sensors to the top of the riprap stones was obtained in order to get the water depth. The values were estimated based on ultrasonic sensors measurements of dry model, measurements of water surface in the level of tops of the stones during shutting down of the experiments after failure (Figure 28 and Figure 29) and reading on the window scales in video documentation. The upper downstream sensor distance to the top of the stones is 1860 mm and it is 1850 mm for the lower downstream sensor.



Figure 28 Water surface on the level of top of the riprap stones for bed level estimation (D02)



Figure 29 Water surface on the level of top of the riprap stones for bed level estimation (T01)

To obtain water depth from water level measurements, the effective bed level of the rough bed has to be located (i.e. reference bed level for water depth). According to (Hughes & Flack, 1984) the effective bed level is located 0.2  $d_{65}$  below the physical top of the riprap stones. The same approach was used also in the lab experiment with the similar stone size as in this experiment (Pagliara, Das, & Carnacina, 2008). The effective bed was located 12 mm below the top of the stones and the effective bed width  $b_{ef}$  = 542 mm (bed width b=550 mm is the width of the wooden plate below fixed stones).

#### 8.4 Point velocity

#### 8.4.1 ADV

Nortek Vectrino side ADV (i.e. Acoustic Doppler Velocimetry) probes were used for point velocity measurements. The configuration of the probes requires the values of certain parameters (i.e. control volume, nominal velocity range and frequency of data collection). More detailed description of the parameters can be found in (Nortek AS, 2018) or (Fernández, 2019). The configuration for this experiment was: control volume 5.5. mm, nominal velocity range 4 m/s and frequency 200 Hz. According to (Fernández, 2019), 5 minutes time interval was measured for each discharge.

Two probes were mounted to the vertical threaded rods screwed to the wooden horizontal beam fixed to the flume walls (see Figure 30 and Figure 31). The probes were placed 60 mm upstream the start of the test section. Measurements in the test section were not possible, because moving stones could damage the probes.

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Figure 30 Mounting of ADV probes - streamwise view



Figure 31 Mounting of ADV probes – top view

14 points along the ADV cross section were measured. Figure 32 shows the position of the points, the coordinate system is adopted from the ADV probe. The notation of the points indicates the ADV (A) or Propeller (P) measurement, profile number (1-6) and z coordination value in mm. Table 5 shows the profiles station. The lower points were placed as close to the bed or bank as possible (see Figure 33). Figure 34 presents an example of ADV measurements with discharge of 400 l/s.

Table 5 Profiles station		
Profile	<i>y</i> [mm]	
1	700	
2	600	
3	450	
3b	500	
4	350	
5	372	
6	304	

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Figure 32 Point velocity measurement points



Figure 33 Position of ADV point A5-131

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Figure 34 ADV 06 experiment, Q=400 l/s

The data from ADV measurements were processed using WinADV software (Wahl, 2000), R programming language and MS Excel. The data was despiked using (Goring & Nikora, 2002) despiking function in WinADV and the average values of three velocity components were obtained from the software ( $v_x$ ,  $v_y$ ,  $v_z$ , see coordinate system in Figure 32). The average discharge values were obtained from discharge meters for time interval 5 min of the ADV measurements. Point velocity was measured for discharge up to 400 l/s, but due to non-reasonable results, the data for discharge greater than 300 l/s were excluded from the analysis.

Mean velocity was obtained based on ADV measurements. The cross section was divided into six sections based on the measured profiles (see Figure 35). Depth-averaged velocity ( $v_{avg}$ ) of each section was assumed as velocity at 40 % of the total depth measured up from the bed (i.e. one-point method, assuming turbulent velocity profile) (Julien, 1995), (Rantz, 1982). 40 % velocity corresponds to  $z'/z_{21}=0.4$ , where  $z'=z-(40-0.2d_{65})$ , i.e. the water depth for velocity measurement point measured up from effective bed, top of the riprap stones is assumed 40 mm above the wooden plate and  $z_{21}$  is the water depth. In sections with more velocity points measured along the water depth, linear interpolation was used to obtain the mean velocity. In some sections, only one point was measured (due to low water depth and lack of time), so this point velocity was taken as a mean velocity, even when  $z'/z_{21} \neq 0.4$ . Using depth-average velocity and flow area, the discharge was obtained for each section. Then the total discharge was divided by total flow area of the cross section to obtain cross section mean velocity. The ultrasonic measurements during ADV and Propeller tests were influenced by intrusive probes and could not be used, therefore average values from placed riprap inclined experiments (T01-T04) of water depth (i.e. upper ultrasonic sensor) and discharges were used to obtain flow areas and mean velocity.
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Figure 35 Discharge sections for determination of mean velocity for ADV

# 8.4.2 Propeller

OTT C2 current meter (i.e. Propeller) was also used for the point velocity measurements. The propeller was placed 110 mm upstream the start of the test section.



Figure 32. Examples of propeller measurement can be seen in Figure 36 and Figure 37.

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Figure 36 Propeller measurement in point P1-86



Figure 37 An example of propeller measurement

The propeller records number of rotations in the time interval (i.e. 1 min) and then the point velocity is calculated according to calibration formula. Two different propellers were used according to their velocity limit, one with the velocity limit up to 2 m/s (i.e. Propeller 2) and the other with the limit up to 2.5 m/s (i.e. Propeller 3). For some points with velocity around 2 m/s, both propellers were used and the results were compared. The average discharge values for each discharge step were obtained from the discharge meters.

Same procedure for determination of mean velocity as for ADV measurements was used for Propeller (see chapter 8.4.1). The discharge sections can be seen in Figure 38.



Figure 38 Discharge sections for determination of mean velocity for Propeller

# 8.5 Water surface velocity

Water surface velocity was measured by adding the table tennis balls to the stream (see Figure 39). The measurements were done for one scenario (i.e. T05). Six balls were added to the model upstream the test section and the video was recorded from the top camera for each discharge. Fudaa-LSPIV (i.e. Large Scale Particle Image Velocimetry) software was planned to use to obtain the velocity, but the software could not recognize the tracers due to the light reflection on the water surface (even after reduction of the reflection by modifying the camera configuration and lightning of the model). In the end the water surface velocity was calculated by hand from the time between the video frames and the specific distance in the model (i.e. 1.5 m).

According to (Rantz, 1982), mean velocity can be obtained from surface velocity using velocity index 0.85. Therefore,  $v_{mean} = 0.85 v_{surf.}$ 



Figure 39 Table tennis balls in the experiment

#### 8.6 Video documentation

Two SONY RX0 cameras were used for video documentation of the experiment. The first one was mounted on the top of the flume as can be seen in Figure 24. The second one was placed behind the window outside the flume to record the moving stones under water level. A black cloth "tent" was built around the side camera to eliminate the light reflection on the video. Figure 40 and Figure 41 show the examples of video documentation.



Figure 40 Example of side camera documentation (D02)



Figure 41 Example of top camera documentation (T02)

### 8.7 Failure of the riprap

Three stages of failure were defined: i) first stone movement, ii) failure initiation, iii) total failure. For each riprap stability experiment, the number of stones leaving the bed and bank of the test section was manually counted for each discharge step (see Appendix 3).

#### 8.7.1 First stone movement

First stone movement is defined as a moment when the first riprap stone leaves the test section. For all experiments the discharge of the first stone movement was much lower than for the other failure stages. Especially for dumped riprap, where the first stones moved at the initial discharge (i.e. 200 l/s), but it did not seem to influence the riprap stability. Therefore, the first stone movement is not a good indicator of the riprap failure and it was not used for the failure identification. The discharge values for the first stone movement ( $Q_{stm}$ ) can be seen in Table 6.

### 8.7.2 Failure initiation

Failure initiation is a moment when continuous erosion of the riprap stones begins and leads to the total failure. This stage of the failure is the most important in terms of critical values of the measured variables, because the flow conditions are not influenced by the failure itself (i.e. the conditions which caused the failure, not the conditions during the failure). The values of failure initiation time  $(t_{fi})$  and the critical discharge  $(Q_c)$  can be seen in Table 6.

The critical time interval  $t_c$ , is defined as the interval between the time point when constant discharge is reached for certain discharge step and the failure initiation time (i.e. time to failure for certain discharge). The critical values of the variables (i.e. discharge, water surface level) were taken as an average value in the critical time interval  $t_c$ .

The exception is when the failure initiation occurred during increasing of the discharge. In these cases, the critical upstream water depth  $(z_{1,c})$  was taken in the failure initiation time (i.e. average value of 10 s) and the critical discharge  $(Q_c)$  was obtained using linear interpolation from the relation of constant upstream water depth and constant discharge for each discharge step. The example for P02 experiment can be seen in Figure 42. Then the values of downstream ultrasonic sensors were obtained as the average of 10 s interval before failure initiation time.

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Figure 42 Linear interpolation of the critical discharge value for the failure initiation during discharge increasing

#### 8.7.3 Total failure

Total failure occurs when riprap stones slide down from the bank and the filter is exposed along the whole submerged height of the bank. In some cases, total failure occurred in the next discharge step after the failure initiation. The values of total failure time  $(t_{totf})$  and discharge  $(Q_{totf})$  can be seen in Table 6. The table shows that it could take tens of minutes to reach the failure after the initiation stage. Therefore, it is important to use the initial failure values as the critical values for further results analysis.

### 9 **RESULTS**

#### 9.1 Failure of the riprap

Critical values (i.e. values during the failure stages) of the variables were obtained, critical time and discharge for each stage of the failure can be seen in Table 6.

exp	First stone movement	Failure ini	tiation	Total failure		
	[l/s]	t <sub>fi</sub> [hh:mm:ss]	<i>Q</i> <sub>c</sub> [l/s]	t <sub>totf</sub> [hh:mm:ss]	Q <sub>totf</sub> [l/s]	
D01	200	03:32:18	297.89	03:40:44	300	
D02	200	03:36:45	299.12	03:57:42	300	
D03	200	03:11:02	274.70	04:20:32	325	
D04	200	02:49:00	275.19	03:38:08	300	
P01	200	02:40:28	273.60	02:42:09	275	
P02	200	02:37:04	269.28	02:40:07	250-275	
P03	225	02:40:27	250.47	02:44:10	275	
P04	225	02:56:33	274.63	02:56:56	275	
T01	375	09:13:45	449.80	09:16:45	450-475	
T02	350	09:09:00	450.77	09:10:28	450	
T03	325	08:29:17	447.59	08:34:25	450	
T04	300	08:46:51	450.15	09:07:28	450	
T05	350	08:40:00	450.34	08:44:40	450	

# 9.2 Test section flow conditions

#### 9.2.1 Flow conditions

For all experiments water depth in the test section for upper downstream sensor  $(z_{21})$  and lower downstream sensor  $(z_{22})$  was obtained from the ultrasonic sensor measurements. Mean flow velocity (v=Q/A) and Froude number (1) were calculated. The values of the variables can be found in Appendix 1.

Figure 43 compares water depth for all riprap stability experiments. Figure 44, Figure 45 and Figure 46 show water depth vs. discharge for each riprap configuration.

For all cases, the difference between water depth for upper and lower ultrasonic sensor can be seen in the graphs. For lower discharges (i.e. 200-250 l/s), water depth was higher for upper sensor, but for higher discharges (i.e. > 250 l/s), the water depth was higher for the lower sensor. The exception was experiment P01 (placed rip-rap parallel), where it was the other way around (see Figure 45) which could be caused by filter exposure on the bank (i.e. 2 riprap stones left the test section) during discharge of 200 l/s. The water depth distribution along the test section was influenced by the model set-up (i.e. sluice gate and the step on the end of the wooden platform, see Figure 1). The riprap placement could also influence the water level distribution over the test section. Uniform flow can not be developed in this set-up, hence the flow conditions in the test section were not constant. The difference in Froude number values is shown in Figure 47.

As for dumped riprap, decreasing of water depth with increasing discharge can be seen in Figure 44. This was caused by continuous erosion of riprap stones during the whole experiment (i.e. the bed level decreased). There was much more movement of the stones for dumped riprap, than for the placed ones.

Regarding placed riprap inclined, the water depth for lower sensor started to increase for discharges greater than 300 l/s (see Figure 46). This was caused by white water coming from the bank. Furthermore, the water depth for T05 is lower than for the other placed riprap inclined experiments. In this case, the upper sensor was placed to the spot, where no white water was observed (i.e. upstream part of the test section and close to the flume wall, see Figure 23). Thus, the white water influenced all water elevation measurements for placed riprap inclined. For upper sensor, the decrease of water depth was observed for discharges greater than 375 l/s.



Figure 43 Water depth vs. discharge for all riprap stability experiments



Figure 44 Water depth vs. discharge for dumped riprap (D)



Figure 45 Water depth vs. discharge for placed riprap parallel (P)

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Figure 46 Water depth vs. discharge for placed riprap inclined (T)



Figure 47 Froude number vs. discharge for all riprap stability experiments

# 9.2.2 Critical flow conditions

After the failure stages identification (see chapter 9.1), the critical values of variables were obtained (i.e. the values which caused the failure initiation). Table 7 shows the critical values of discharge ( $Q_c$ ), critical time interval ( $t_c$ ), water depth for upper ( $z_{21,c}$ ) and lower ( $z_{22,c}$ ) ultrasonic sensor, velocities ( $v_{21,c}$ ,  $v_{22,c}$ ), Froude numbers ( $Fr_{21,c}$ ,  $Fr_{22,c}$ ) and longitudinal slopes ( $s_{21,c}$ ,  $s_{11,c}$ ) for all riprap stability experiments. Longitudinal slopes corresponding the flow conditions were obtained using Manning' formula (see chapter 3).

exp	Qc [l/s]	t <sub>c</sub> [hh:mm:ss]	<i>z<sub>21,c</sub></i> [mm]	Z22,c [mm]	z <sub>22,c</sub> -z <sub>21,c</sub> [mm]	<i>v</i> <sub>21,c</sub> [m/s]	v <sub>22,c</sub> [m/s]	Fr <sub>21,c</sub> [-]	Fr <sub>22,c</sub> [-]	\$ <sub>21,c</sub> [%]	\$ <sub>22,c</sub> [%]
D01	297.9	00:01:48	183.0	190.9	7.9	2.40	2.28	1.96	1.83	11.2	9.1
D02	299.1	00:07:15	185.8	193.5	7.7	2.36	2.25	1.92	1.80	10.4	8.6
D03	274.7	00:31:02	182.8	191.1	8.3	2.21	2.10	1.81	1.69	8.9	7.2
D04	275.2	00:13:00	195.2	-	-	2.05	-	1.63	-	6.5	-
P01	273.6	-	191.5	193.9	2.5	2.09	2.05	1.67	1.64	7.1	6.6
P02	269.3	-	212.6	209.4	-3.2	1.81	1.84	1.39	1.42	4.0	4.3
P03	250.5	-	211.8	198.8	-13.1	1.69	1.82	1.30	1.44	3.3	4.5
P04	274.6	00:16:33	210.8	-	-	1.86	-	1.43	-	4.4	-
T01	449.8	00:41:15	199.9	222.9	23.0	3.25	2.85	2.56	2.14	23.1	13.4
T02	450.8	00:37:30	198.5	220.6	22.0	3.29	2.89	2.60	2.18	24.2	14.3
T03	447.6	-	193.4	219.1	25.7	3.37	2.89	2.69	2.19	27.0	14.5
T04	450.2	00:15:21	203.5	-	-	3.19	-	2.49	-	21.3	-
T05	450.3	00:06:00	184.0	-	-	3.60	-	2.94	-	35.1	-

Table 7 Critical flow conditions for all riprap stability experiments

Figure 46 compares critical water depths for all riprap stability experiments. Due to white water, the highest differences occurred for placed riprap inclined (T). This difference is also an origin of high range of Froude numbers in Figure 49.

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Figure 49 compares Froude numbers for all riprap stability experiments for failure initiation. It is apparent from the graph, that placed riprap inclined is the most stable riprap configuration and it can withstand much higher Froude number (i.e. ca. 2.1 to 2.95 with discharge of ca. 450 l/s) than the other configurations. Placed riprap parallel is the least stable configuration with the range of Froude number ca. 1.3 to 1.7 with discharge of ca. 250 to 275 l/s. Dumped riprap is a bit more stable than placed riprap parallel with Froude number range ca 1.6 to 2.0 with discharge of ca. 275 to 300 l/s.

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Figure 49 Critical Froude numbers vs. discharge

Figure 50 shows longitudinal slope corresponding to the critical flow conditions. From the graph is clear, that differences in water depth for ultrasonic sensors (see Table 6) caused significant differences in the longitudinal slopes, especially for placed riprap inclined configuration



### 9.3 Velocity measurements

Velocity perpendicular to the cross section  $(v_x)$  is the dominant velocity component in this experiment setup. Therefore, values of  $v_x$  from ADV measurements were used for the further analysis of the results. The propeller measured velocity perpendicular to the cross section, hence the values are considered as  $v_x$ . Measured values of the velocity can be seen in Appendix 2. Values of point velocity  $v_x$  along the water depth from ADV and Propeller measurements are presented in Figure 53 to Figure 56.

### 9.3.1 ADV

Summary of depth-averaged and mean velocity calculations for ADV measurements can be seen in Table 8.

Profile	<i>z</i> 21 [mm]	z'/z <sub>21</sub> [-]	v <sub>avg</sub> [m/s]	$\begin{bmatrix} A_i \\ [\mathbf{m}^2] \end{bmatrix}$	<i>Qi</i> [l/s]	Q <sub>adv</sub> [l/s]	<i>v<sub>adv</sub></i> [m/s]	Qus [l/s]	A [m <sup>2</sup> ]	<i>v<sub>us</sub></i> [m/s]
1	206.9	0.40	1.12	0.072	80.89	157.07	1.09	200.11	0.144	1.39
2	206.9	0.40	1.25	0.021	25.90					
3b	206.9	0.37	1.21	0.016	18.74					
3	206.9	0.40	1.11	0.010	11.54					
5	206.9	0.43	0.81	0.013	10.69					
6	206.9	0.56	0.78	0.012	9.31					
1	205.6	0.40	1.55	0.072	111.53	208.49	1.46	225.00	0.143	1.57
2	205.6	0.40	1.67	0.021	34.39					
3b	205.6	0.37	1.71	0.015	26.44					
3	205.6	0.40	1.49	0.010	15.37					
5	205.6	0.43	0.95	0.013	12.51					
6	205.6	0.57	0.70	0.012	8.24					
1	203.3	0.40	1.78	0.071	126.86	237.45	1.68	250.11	0.141	1.77
2	203.3	0.40	1.88	0.020	38.20					
3b	203.3	0.37	1.99	0.015	30.27					
3	203.3	0.40	1.79	0.010	18.29					
5	203.3	0.44	1.12	0.013	14.51					
6	203.3	0.58	0.82	0.011	9.31					
1	201.4	0.40	1.92	0.070	135.27	259.21	1.86	274.66	0.139	1.97
2	201.4	0.40	2.01	0.020	40.44					
3b	201.4	0.40	2.20	0.015	33.22	$v_x$ value f	for P3 use	d (no $v_x$ val	ue for P3b	o for this Q)
3	201.4	0.40	2.20	0.010	22.25					
5	201.4	0.45	1.30	0.013	16.57					
6	201.4	0.60	1.04	0.011	11.46					
1	201.7	0.40	2.18	0.071	154.25	293.43	2.10	299.99	0.140	2.15
2	201.7	0.40	2.28	0.020	45.96					
3b	201.7	0.38	2.54	0.015	38.46					
3	201.7	0.40	2.34	0.010	23.72					
5	201.7	0.45	1.36	0.013	17.33					
6	201.7	0.60	1.24	0.011	13.71					

Table 8 Discharge and mean velocity obtained from ADV ( $Q_{adv}$ ,  $v_{adv}$ ) measurements and water level measurements ( $O_{uv}$ ,  $v_{uv}$ )

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#### 9.3.2 Propeller

Summary of depth-averaged and mean velocity calculations for propeller measurements can be seen in Table 9.

Profile	<i>z</i> <sub>2</sub> [mm]	z'/z <sub>2</sub> [-]	<i>v<sub>avg</sub></i> [m/s]	$A_i$ [m <sup>2</sup> ]	<i>Qi</i> [l/s]	Q <sub>prop</sub> [l/s]	v <sub>prop</sub> [m/s]	Qus [l/s]	A [m <sup>2</sup> ]	<i>v<sub>us</sub></i> [m/s]
1	206.9	0.40	1.42	0.072	102.61	197.72	1.37	200.11	0.144	1.39
2	206.9	0.40	1.53	0.026	39.47					
3	206.9	0.40	1.27	0.025	31.45					
4	206.9	0.28	1.14	0.021	24.20					
1	205.6	0.40	1.93	0.072	138.55	252.93	1.77	225.00	0.143	1.57
2	205.6	0.40	1.84	0.026	47.26					
3	205.6	0.40	1.69	0.025	41.48					
4	205.6	0.28	1.23	0.021	25.64					
1	203.3	0.29	1.68	0.071	119.61	250.00	1.77	250.11	0.141	1.77
2	203.3	0.29	1.98	0.025	50.24					
3	203.3	0.40	1.97	0.024	47.80					
4	203.3	0.29	1.59	0.020	32.35					
1	201.7	0.29	2.20	0.071	155.42	303.22	2.17	299.99	0.140	2.15
2	201.7	0.29	2.25	0.025	56.63					
3	201.7	0.29	2.17	0.024	52.26					
4	201.7	0.29	1.95	0.020	38.91					

Table 9 Discharge and mean velocity obtained from Propeller measurements (Qprop, vprop) and water level measurements  $(Q_{us}, v_{us})$ 

#### 9.3.3 Water surface velocity

The results of the manual computation of the water surface velocity for experiment T05 can be seen in Table 10. According to (Rantz, 1982), mean velocity can be obtained from surface velocity using velocity index 0.85. Therefore,  $v_{mean} = 0.85 v_{surf.}$ 

0.0.1/-1		
Q [I/S]	<i>v<sub>surf</sub></i> [ <b>m</b> / <b>s</b> ]	V <sub>mean</sub> [ <b>m</b> /S]
199.84	1.80	1.53
224.74	2.34	1.99
249.48	2.62	2.23
274.99	2.97	2.52
300.63	3.15	2.68
323.88	3.27	2.78
349.63	3.86	3.28
375.18	4.08	3.47
399.74	5.22	4.44
423.76	5.51	4.68
450.34	5.77	4.90

Table 10 Results of water surface velocity measurements

#### 9.3.4 Depth average velocity distribution

Figure 51 shows depth-average velocity distribution along velocity measurements cross section. Regarding horizontal distribution in trapezoidal channel, the highest velocity occurs in the center of the channel and decrease closer to the bank (Chow, 1959). Decrease of the velocity along riverbank can be seen in Figure 51, which corresponds to the expected velocity distribution in a trapezoidal channel. However, there is velocity decrease also on the bed in the direction to channel center, where the highest velocity is expected. The cross section is half trapezoidal, so the decreasing could be caused by vertical glass wall. Velocity distribution could be also influenced by flow pattern generated by sluice gate. Flow pattern for discharge 300 l/s can be roughly seen in Figure 52.



Figure 51 Horizontal distribution of depth-averaged velocity along velocity measurement cross section



Figure 52 Example of flow pattern from the gate, Q=300 l/s (T02)

#### 9.3.5 Point velocity comparison

Values of  $v_x$  velocity from ADV and Propeller measurements are shown in Figure 53 to Figure 56. Furthermore, the values of surface velocity were plotted to Figure 53 to Figure 55. The position of the profiles can be seen in Table 5 and Figure 32. From the comparison it is clear that propeller measurements give higher values of the velocity than ADV measurements in all cases. As can be seen in Table 8 and Table 9, propeller measurements give better results in comparison with the discharge obtained from water level measurements. Therefore, it seems that ADV measurements underestimate the velocity, which could be caused by configuration of the probe. As for the propeller, one round of the measurements was done for all points. Two propellers with different calibration formulas were used according to expected maximum velocity (see chapter 8.4.2). For discharge 250 l/s, both propellers were tested in the same points and Figure 53 to Figure 56 show a good match in velocity values for Profile 1, 3, 4, but the values vary for Profile 2. Hence, repetition of the measurements should be done in order to verify the results. The values of surface velocity profiles.

These differences could be caused by configuration of ADV probes, only one configuration was tested. Propeller measurements give the better results in comparison with discharge obtained from water level measurements, as can be seen in Table 8 and Table 9.



Figure 53 Point velocity  $v_x$  along the water depth in Profile 1









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Figure 56 Point velocity  $v_x$  along the water depth in Profile 4

#### 9.3.6 Mean velocity comparison

Mean velocity values were obtained for all velocities (i.e. ADV, Propeller, surface velocity) and water level measurements (i.e. ultrasonic sensors). Figure 57 compares mean velocity values vs. discharge. Average values from placed riprap inclined experiments (T01-T04) of water depth (i.e. upper ultrasonic sensor) and discharges were used to obtain mean velocity based on ultrasonic sensors measurements. The values of mean and point velocity obtained from the different methods can be seen in Appendix 2.

The graphs show that there is very good match between mean velocity obtain from water level measurements (i.e. ultrasonic sensors) and propeller measurements (with exception for 225 l/s).

Mean velocity obtained from ADV measurements is a bit underestimated in comparison to mean velocity from ultrasonic sensors. Except discharge 200 l/s, the difference is ca. 0.1 m/s.

Mean velocity obtained from surface velocity measurements is much higher than velocity measured by the other methods. However, the surface velocity compared to the point velocities in Figure 53 to Figure 55 gives a reasonable result as an extension of the velocity profiles. Therefore, the suitability of velocity index 0.85 is questionable.

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Figure 57 Comparison of mean velocity obtained from different measurement methods

### 9.3.7 Velocity on the stones

To measure the velocity close to the stones ADV probes were placed as close as possible to the bed and bank stones (see Figure 32 and Figure 33). Table 11 presents the points closest to the stones, z' is the water depth measured up from effective bed to the point and  $z_{21}$  is the water depth.  $z'/z_{21}$  is the average value for all discharges. Profile 5 and 6 are placed on the bank.

Profile	Point	<i>z</i> [mm]	z' [mm]	$z'/z_{21}$ [-]	Bed/Bank
1	A1-60	60	32	0.16	Bed
2	A2-50	50	22	0.11	Bed
3	A3-82	82	54	0.27	Bed (Toe)
5	A5-131	131	51	0.44	Bank
6	A6-175	175	49.7	0.59	Bank

Table 11 Points used for measurement of velocity on the stones

Figure 58 shows velocity on the stones vs. mean velocity obtained from ultrasonic sensors. Figure 59 presents the ratio of velocity on the stones to mean velocity obtained from ultrasonic sensors vs. mean velocity from ultrasonic sensors. Due to the water depth of the velocity point in Profile 3 (see Table 11), velocity on the stones in Profile 3 is higher than for the other profiles, as can be seen in the graphs. The values of the velocity measured on the bank (i.e. Profile 5 and 6) are comparable with the values measured on the bed (i.e. Profile 1 and 2). However, the velocity on the bank was measured in the points with higher water depth z' (i.e. distance from the effective bed level) than on the bed, so the velocity on the bank is expected lower closed to the stones.

Figure 59 shows that the ratio of velocity on the stones to mean velocity is slightly increasing with increasing mean velocity. Excluding Profile 3, the values of the ratio are in the range ca. 0.47 to 0.68 for the bed.



Figure 58 Velocity on the stones vs. mean velocity obtained from ultrasonic sensors



Figure 59 Velocity on the stones over mean velocity vs. mean velocity obtained from ultrasonic sensors

### 9.4 Stones counting

Numbers of counted stones which left the test section for all riprap stability experiment can be found in Appendix 3.

#### **10 PROTOTYPE**

The critical flow conditions described in chapter 9.2.2 were scaled up using conceptual scale of 1:10. The prototype is assumed with the same geometry as the model (i.e. half trapezoidal cross section with the bed width ca. 5 m and side slope 1:1.5 in prototype). The prototype values of stones diameter  $d_{50}$ , discharge Q, water depth z and mean velocity v were obtained using equations (4), (5), (6) and (7) (see Table 12, m for model and p for prototype).

According to Froude's model law the Froude number is equal for model and prototype. The diameter of the stones  $d_{50}$  in the prototype is 0.57 m. It can be seen in Table 12 that the critical discharge in prototype for placed riprap inclined is ca. 142 m<sup>3</sup>/s, for dumped riprap ca. 87 to 95 m<sup>3</sup>/s and for placed riprap parallel ca. 79 to 87 m<sup>3</sup>/s. The water depth *z* in prototype is ca. 1.8 to 2.1 m. The prototype velocity starts ca. on the value of 5 m/s for placer riprap parallel and ends at the values of about 10-11 m/s for placer riprap inclined.

				MODEL					PROTOTYPE					
exp	<i>Fr</i> <sub>21</sub> [-]	Fr <sub>22</sub> [-]	d <sub>50,m</sub> [m]	$Q_m$ [m <sup>3</sup> /s]	Z2,1m [ <b>m</b> ]	Z22,m [ <b>m</b> ]	<i>v</i> 21,m [ <b>m/s</b> ]	<i>v</i> <sub>22,m</sub> [m/s]	<i>d</i> <sub>50,p</sub> [m]	$Q_p$ [m <sup>3</sup> /s]	Z2,1p [ <b>m</b> ]	Z22,p [ <b>m</b> ]	<i>v</i> <sub>21,p</sub> [m/s]	<i>v</i> <sub>22,p</sub> [m/s]
D01	1.96	1.83	0.057	0.30	0.18	0.19	2.40	2.28	0.57	94.20	1.83	1.91	7.58	7.21
D02	1.92	1.80	0.057	0.30	0.19	0.19	2.36	2.25	0.57	94.59	1.86	1.94	7.48	7.12
D03	1.81	1.69	0.057	0.27	0.18	0.19	2.21	2.10	0.57	86.87	1.83	1.91	7.00	6.64
D04	1.63	-	0.057	0.28	0.20	-	2.05	-	0.57	87.02	1.95	-	6.48	-
P01	1.67	1.64	0.057	0.27	0.19	0.19	2.09	2.05	0.57	86.52	1.91	1.94	6.60	6.50
P02	1.39	1.42	0.057	0.27	0.21	0.21	1.81	1.84	0.57	85.15	2.13	2.09	5.71	5.82
P03	1.30	1.44	0.057	0.25	0.21	0.20	1.69	1.82	0.57	79.21	2.12	1.99	5.34	5.77
P04	1.43	-	0.057	0.27	0.21	-	1.86	-	0.57	86.84	2.11	-	5.89	-
T01	2.56	2.14	0.057	0.45	0.20	0.22	3.25	2.85	0.57	142.24	2.00	2.23	10.29	9.00
T02	2.60	2.18	0.057	0.45	0.20	0.22	3.29	2.89	0.57	142.55	1.99	2.21	10.40	9.14
T03	2.69	2.19	0.057	0.45	0.19	0.22	3.37	2.89	0.57	141.54	1.93	2.19	10.66	9.15
T04	2.49	-	0.057	0.45	0.20	-	3.19	-	0.57	142.35	2.03	-	10.08	-
T05	2.94	-	0.057	0.45	0.18	-	3.60	-	0.57	142.41	1.84	-	11.39	-

Table 12 Critical flow condition for all riprap stability experiments for model and prototype in scale 1:10

### 11 COMPARISON WITH ROBINSON'S FORMULA

Evaluation using Robinson's formula for riprap design in steep rivers was performed in this chapter. Stone diameter according to Robinson's formula (16), (17), (Robinson et al., 1993), (NVE, 2009) was determined for critical flow conditions for all riprap stability experiments. Hydraulic calculations can be seen in Appendix 4.

$$d_{50} = 1.5s^{0.79}q^{0.53} \quad \text{for } s < 1:10 \tag{16}$$

$$d_{50} = 0.5s^{0.31}q^{0.53} \quad \text{for } 1:10 \le s \le 1:2.5 \tag{17}$$

$$q = \frac{Q}{b_{ef}} \tag{18}$$

$$b_{ef} = b + \frac{mz}{2} \tag{19}$$

where  $d_{50}$  is design median stone diameter, *s* longitudinal slope, *q* unit discharge, *Q* discharge,  $b_{ef}$  effective width of the channel, *b* width of horizontal bed of half trapezoidal channel, 1:*m* is a side slope of the bank and *z* is the water depth.

Table 13 show the determined stone diameters according to Robinson's formula ( $d_{50rob}$ ). Comparison of the designed values with stone diameters used in the experiment indicates that Robinson's formula overestimates the stone diameter in all cases. The values of determined diameters are ca. 2-3 times higher than diameters used in the experiment for dumped riprap and ca. 4-5 times higher for placed riprap inclined. As for placed riprap parallel the overestimation is not that significant and the diameters are rather similar, however Robinson's formula assumes riprap layer thickness  $2d_{50}$ , but placed riprap parallel was built with one layer of lying stones so the thickness was even lower than  $d_{50}$ .

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exp	$Q_c [\mathrm{m^3/s}]$	<i>q</i> <sub>21,c</sub> [m <sup>3</sup> /s/m]	<i>q</i> <sub>22,c</sub> [m <sup>3</sup> /s/m]	s <sub>21,c</sub> [%]	s <sub>22,c</sub> [%]	<i>d</i> 50 [mm]	d <sub>50rob,21</sub> [mm]	d <sub>50rob,22</sub> [mm]
D01	0.298	0.439	0.435	11.2	9.1	57	164	145
D02	0.299	0.439	0.436	10.4	8.6	57	160	139
D03	0.275	0.404	0.401	8.9	7.2	57	137	115
D04	0.275	0.400	-	6.5	-	57	107	-
P01	0.274	0.400	0.398	7.1	6.6	57	114	107
P02	0.269	0.385	0.385	4.0	4.3	57	71	75
P03	0.250	0.358	0.363	3.3	4.5	57	59	76
P04	0.275	0.392	-	4.4	-	57	77	-
T01	0.450	0.650	0.635	23.1	13.4	57	253	211
T02	0.451	0.653	0.638	24.2	14.3	57	257	215
T03	0.448	0.652	0.634	27.0	14.5	57	265	216
T04	0.450	0.649	-	21.3	-	57	246	-
T05	0.450	0.662	-	35.1	-	57	291	-

Table 13 Parameters for flow in steep rivers and designed stones diameter according to Robinson's formula

# 12 RECOMMEDATIONS FOR THE ADVANCED EXPERIMENT

To increase the quality in future experiments, the flowing is recommended:

- Model set-up with steep slope would eliminate issues connected to sluice gate and analogy between sluice gate and steep rivers. Furthermore, uniform flow would be developed in the inclined flume. For steep slopes very long flumes are required.
- Use of ultrasonic sensors with the smaller range closer to the water surface, so they don't influence each other
- Measure the bed level in order to obtain the bed level using more methods (i.e. not only ultrasonic sensors).
- Put more focus on point velocity measurements (i.e. more rounds of measurements, more ADV configurations, Ultrasonic Velocity Profiler).

# **13 CONCLUSION**

From results analysis it is clear that placed riprap inclined is the most stable riprap configuration. With discharge of ca. 450 l/s (ca. 142 m<sup>3</sup>/s in prototype) placed riprap inclined withstands Froude number of ca. 2.1 to 2.95. Placed riprap parallel is the least stable configuration. This configuration withstands Froude number of ca. 1.3 to 1.7 with discharge of ca. 250 to 275 l/s (ca. 79 to 87 m<sup>3</sup>/s in prototype). Dumped riprap is a bit more stable than placed riprap parallel. With discharge of ca. 275 to 300 l/s (ca. 87 to 95 m<sup>3</sup>/s in prototype) dumped riprap withstands Froude number of ca. 1.6 to 2.0. The stones diameter in prototype (1:10) is 0.57 m. Since, placed riprap parallel is a placed riprap, higher stability than dumped riprap could be expected. However, placed riprap parallel was built as one layer of stones, so less stones than for dumped riprap were used (see chapter 6).

Regarding velocity measurements, point velocities measured by ADV and Propeller give reasonable shapes of velocity profiles. However, ADV seems to underestimate the point velocity values in comparison to Propeller. Comparison of mean velocity obtained from all methods was done (i.e. ADV, Propeller, surface

velocity, ultrasonic sensors. From the comparison it is clear, that use of Propeller gives the best match with mean velocity from ultrasonic sensors, ADV underestimates the values and the values obtained from surface velocity measurements are overestimated. Due to lack of time, only one round of measurements was done for Propeller and surface velocities and one configuration of ADV probes was tested. Therefore, more experiments would be required for more precise investigation of velocity profiles downstream sluice gate.

The report contains all necessary data to build a numerical model based on the experiments.

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**APPENDIX 1** 

# FLOW CONDITIONS

exp	Q [l/s]	<i>t</i> [min]	WSL <sub>21</sub> [mm]	WSL <sub>22</sub> [mm]	<i>Z21</i> [ <b>mm</b> ]	Z22 [mm]	<i>v</i> <sub>21</sub> [m/s]	<i>v</i> <sub>22</sub> [m/s]	<i>Fr</i> <sub>21</sub> [-]	Fr <sub>22</sub> [-]
D01	200.75	48.50	1664.87	1653.49	207.13	208.51	1.39	1.38	1.08	1.07
D01	224.97	46.00	1672.77	1657.15	199.23	204.85	1.63	1.58	1.29	1.23
D01	249.35	45.00	1682.96	1666.56	189.04	195.44	1.93	1.85	1.56	1.47
D01	274.12	45.00	1688.21	1671.55	183.79	190.45	2.20	2.10	1.79	1.69
D01	297.89	1.80	1689.01	1671.10	182.99	190.90	2.40	2.28	1.96	1.83
D02	199.75	48.50	1664.44	1655.70	207.56	206.30	1.38	1.39	1.07	1.08
D02	224.85	35.00	1663.57	1650.51	208.43	211.49	1.55	1.52	1.20	1.17
D02	250.50	46.00	1675.64	1662.73	196.36	199.27	1.85	1.82	1.47	1.44
D02	274.41	44.50	1685.84	1666.51	186.16	195.49	2.16	2.04	1.76	1.62
D02	299.12	7.25	1686.16	1668.48	185.84	193.52	2.36	2.25	1.92	1.80
D03	200.00	49.50	1666.30	1656.96	205.70	205.04	1.40	1.40	1.09	1.09
D03	225.48	45.50	1669.76	1654.54	202.24	207.46	1.61	1.56	1.26	1.21
D03	250.32	47.00	1677.16	1663.10	194.84	198.90	1.87	1.82	1.49	1.44
D03	274.70	31.03	1689.22	1670.87	182.78	191.13	2.21	2.10	1.81	1.69
D04	77.86	0.50	1852.90	2899.81	19.10	-	7.34	-	17.16	-
D04	200.14	50.00	1666.55	2899.23	205.45	-	1.40	-	1.09	-
D04	226.09	46.00	1671.52	2899.24	200.48	-	1.63	-	1.28	-
D04	249.11	46.00	1676.22	2899.23	195.78	-	1.85	-	1.47	-
D04	275.19	13.00	1676.82	2899.24	195.18	-	2.05	-	1.63	-
P01	199.85	49.50	1680.62	1663.46	191.38	198.54	1.52	1.46	1.22	1.15
P01	224.65	44.00	1683.60	1668.88	188.40	193.12	1.75	1.69	1.41	1.36
P01	251.63	51.47	1682.76	1671.38	189.24	190.62	1.95	1.93	1.57	1.55
P01	273.60	increase	1680.54	1668.09	191.46	193.91	2.09	2.05	1.67	1.64
P02	200.94	49.00	1659.78	1660.28	212.22	201.72	1.35	1.44	1.04	1.13
P02	225.30	47.80	1655.45	1654.31	216.55	207.69	1.48	1.56	1.13	1.21
P02	251.22	46.57	1656.61	1653.67	215.39	208.33	1.66	1.73	1.27	1.34
P02	269.28	increase	1659.35	1652.56	212.65	209.44	1.81	1.84	1.39	1.42
P03	200.11	49.50	1664.54	1666.80	207.46	195.20	1.38	1.49	1.07	1.19
P03	225.12	47.00	1660.99	1661.60	211.01	200.40	1.52	1.62	1.17	1.28
P03	250.24	46.50	1663.82	1663.03	208.18	198.97	1.72	1.82	1.33	1.44
P03	275.94	0.95	1661.05	1655.80	210.95	206.20	1.87	1.92	1.44	1.49
P03	250.47	increase	1660.16	1663.24	211.84	198.76	1.69	1.82	1.30	1.44
P04	26.85	0.50	1831.92	2899.39	40.08	-	1.17	-	1.92	-
P04	200.60	50.00	1666.67	2899.24	205.33	-	1.40	-	1.09	-
P04	225.23	46.50	1658.77	2899.24	213.23	-	1.51	-	1.15	-
P04	250.10	46.50	1658.35	2899.24	213.65	-	1.67	-	1.28	-
P04	274.63	16.55	1661.22	2899.22	210.78	-	1.86	-	1.43	-

 Table A 1.1 Flow conditions for riprap stability experiments (D01-D04, P01-P04)

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Discharge  $Q_c$ , time interval *t*, distance from ultrasonic sensor to the water level *WSL*, water depth *z*, mean velocity *v*, Froude number *Fr*, upper ultrasonic sensor 21, lower ultrasonic sensor 22.

Table A 1.2 Flow conditions for riprap stability experiments (T01-T03)

exp	Q [l/s]	<i>t</i> [min]	WSL <sub>21</sub> [mm]	WSL <sub>22</sub> [mm]	<i>Z21</i> [ <b>mm</b> ]	z <sub>22</sub> [mm]	<i>v</i> <sub>21</sub> [m/s]	<i>v</i> <sub>22</sub> [m/s]	<i>Fr</i> <sub>21</sub> [-]	<i>Fr</i> <sub>22</sub> [-]
T01	200.46	50.00	1664.49	1658.21	207.51	203.79	1.39	1.42	1.07	1.11
T01	224.82	46.00	1665.52	1657.86	206.48	204.14	1.56	1.59	1.21	1.24
T01	250.35	46.00	1668.19	1660.20	203.81	201.80	1.77	1.79	1.38	1.40
T01	273.67	45.50	1671.96	1658.09	200.04	203.91	1.98	1.93	1.56	1.51
T01	299.31	45.00	1671.56	1658.46	200.44	203.54	2.16	2.12	1.70	1.66
T01	324.75	45.00	1669.41	1654.20	202.59	207.80	2.31	2.24	1.81	1.74
T01	348.86	45.00	1669.45	1650.18	202.55	211.82	2.48	2.35	1.95	1.81
T01	375.08	44.00	1669.72	1645.77	202.28	216.23	2.67	2.47	2.10	1.88
T01	397.76	44.50	1671.12	1642.60	200.88	219.40	2.86	2.57	2.25	1.94
T01	424.87	44.00	1672.54	1642.61	199.46	219.39	3.08	2.74	2.43	2.08
T01	449.80	17.50	1672.06	1639.06	199.94	222.94	3.25	2.85	2.56	2.14
T02	199.97	49.00	1665.58	1657.31	206.42	204.69	1.39	1.41	1.08	1.10
T02	224.90	47.00	1667.69	1659.52	204.31	202.48	1.58	1.60	1.24	1.25
T02	249.79	46.00	1670.52	1663.05	201.48	198.95	1.79	1.82	1.41	1.43
T02	274.68	46.00	1671.55	1661.06	200.45	200.94	1.98	1.97	1.56	1.55
T02	299.91	45.00	1671.92	1660.04	200.08	201.96	2.17	2.14	1.71	1.68
T02	325.47	44.00	1670.23	1656.53	201.77	205.47	2.33	2.28	1.83	1.77
T02	349.45	45.00	1669.58	1651.78	202.42	210.22	2.49	2.38	1.95	1.83
T02	374.83	44.00	1672.19	1649.39	199.81	212.61	2.71	2.52	2.14	1.93
T02	398.78	45.00	1672.60	1645.23	199.40	216.77	2.89	2.61	2.28	1.99
T02	424.65	45.00	1674.23	1646.38	197.77	215.62	3.11	2.80	2.46	2.14
T02	450.77	37.50	1673.49	1641.44	198.51	220.56	3.29	2.89	2.60	2.18
T03	199.76	44.00	1663.82	1656.06	208.18	205.94	1.38	1.39	1.06	1.08
T03	224.95	53.00	1663.86	1654.83	208.14	207.17	1.55	1.56	1.20	1.21
T03	250.41	46.00	1666.35	1658.42	205.65	203.58	1.75	1.77	1.36	1.38
T03	275.37	45.50	1669.19	1657.25	202.81	204.75	1.96	1.93	1.53	1.51
T03	300.86	44.50	1668.25	1657.46	203.75	204.54	2.13	2.12	1.66	1.65
T03	325.83	44.00	1667.08	1653.05	204.92	208.95	2.29	2.23	1.78	1.73
T03	350.19	45.50	1667.04	1650.05	204.96	211.95	2.46	2.36	1.92	1.81
T03	374.99	45.00	1669.52	1647.75	202.48	214.25	2.67	2.49	2.09	1.91
T03	400.18	44.00	1671.83	1644.71	200.17	217.29	2.89	2.61	2.28	1.99
T03	425.76	49.28	1678.93	1644.82	193.07	217.18	3.21	2.78	2.57	2.12
T03	447.59	increase	1678.59	1642.90	193.41	219.10	3.37	2.89	2.69	2.19
Discha velocit	arge <i>Q</i> , tim ty <i>v</i> , Froud	e interval <i>t</i> le number <i>l</i>	, distance fi Fr, upper ul	rom ultraso trasonic se	nic sensor nsor 21, lo	to the wate wer ultrase	er level onic sen	WSL, wa sor 22.	ater depth	z, mean

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exp	Q [l/s]	<i>t</i> [min]	WSL <sub>21</sub> [mm]	WSL <sub>22</sub> [mm]	<i>Z21</i> [ <b>mm</b> ]	<i>Z22</i> [mm]	<i>v</i> <sub>21</sub> [m/s]	<i>v</i> <sub>22</sub> [m/s]	<i>Fr</i> <sub>21</sub> [-]	<i>Fr</i> <sub>22</sub> [-]
T04	26.82	0.50	1866.03	2900.00	5.97	-	8.22	-	34.11	-
T04	200.25	50.00	1666.61	2899.24	205.39	-	1.40	-	1.09	-
T04	225.34	46.00	1668.38	2899.24	203.62	-	1.59	-	1.25	-
T04	249.88	46.50	1669.73	2899.24	202.27	-	1.78	-	1.40	-
T04	274.93	44.50	1669.71	2899.24	202.29	-	1.96	-	1.54	-
T04	299.86	46.50	1669.28	2899.25	202.72	-	2.13	I	1.67	-
T04	324.84	44.50	1666.71	2899.22	205.29	-	2.28	-	1.77	-
T04	350.19	45.00	1664.95	2899.23	207.05	-	2.43	-	1.88	-
T04	374.78	45.50	1664.07	2899.23	207.93	-	2.58	-	2.00	-
T04	400.09	44.00	1665.34	2899.22	206.66	-	2.78	I	2.16	-
T04	425.36	45.00	1667.91	2899.22	204.09	-	3.00	-	2.34	-
T04	450.15	15.35	1668.53	2899.18	203.47	-	3.19	-	2.49	-
T05	199.84	49.70	1661.68	-	210.32	-	1.36	-	1.05	-
T05	224.74	47.00	1660.59	-	211.41	-	1.52	-	1.17	-
T05	249.48	47.50	1667.42	-	204.58	-	1.75	-	1.37	-
T05	274.99	45.50	1675.74	-	196.26	-	2.03	-	1.62	-
T05	300.63	46.00	1682.68	-	189.32	-	2.32	-	1.87	-
T05	323.88	46.00	1683.76	-	188.24	-	2.52	-	2.04	-
T05	349.63	44.50	1687.49	-	184.51	-	2.79	-	2.27	-
T05	375.18	45.00	1688.92	-	183.08	-	3.02	-	2.47	-
T05	399.74	44.00	1688.93	-	183.07	-	3.22	-	2.63	-
T05	423.76	45.50	1688.96	-	183.04	-	3.41	-	2.79	-
T05	450.34	6.00	1688.03	-	183.97	-	3.60	-	2.94	-
Discha	prop () tim	a interval t	distance fr	om ultraga	nio concor	to the wate	r lovol V		or donth	7 moon

Table A 1.3 Flow conditions for riprap stability experiments (T04-T05)

Discharge Q, time interval t, distance from ultrasonic sensor to the water level WSL, water depth z, mean velocity v, Froude number Fr, upper ultrasonic sensor 21, lower ultrasonic sensor 22.

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# **APPENDIX 2**

# **VELOCITY MEASUREMENTS**

Point	Q [l/s]	$v_x$ [m/s]	ν <sub>y</sub> [m/s]	<i>v</i> <sub>z</sub> [m/s]	<i>COR</i> [%]	SNR [dB]				
A1-60	200	0.65	-0.02	-0.06	78.98	59.20				
A1-60	225	0.95	0.00	-0.08	67.08	54.84				
A1-60	250	1.04	-0.02	-0.10	61.60	50.62				
A1-60	275	1.26	0.01	-0.06	57.87	55.70				
A1-60	300	1.45	-0.03	-0.10	52.10	52.31				
A1-60	325	1.44	-0.02	-0.09	50.92	53.53				
A1-60	350	1.66	-0.07	-0.22	44.52	48.87				
A1-60	375	1.75	-0.07	-0.31	41.22	48.02				
A1-60	400	1.61	-0.10	-0.48	39.28	45.92				
A1-120	100	0.89	0.02	0.03	89.38	58.78				
A1-120	200	1.17	0.02	-0.01	75.93	58.90				
A1-120	225	1.64	0.07	0.05	67.55	56.65				
A1-120	250	1.88	0.07	0.06	61.21	53.52				
A1-120	275	2.09	0.09	0.10	54.62	50.45				
A1-120	300	2.30	0.09	0.08	48.24	43.70				
A1-120	325	1.93	0.12	0.34	44.15	38.26				
A1-120	350	1.90	0.13	0.22	41.57	32.56				
A1-120	375	1.41	0.13	0.29	39.52	27.89				
A1-120	400	1.21	0.07	-0.20	35.46	23.07				
A1-170	100	1.04	-0.04	0.08	95.28	59.19				
A1-170	200	1.63	-0.06	-0.02	84.15	59.26				
A1-170	225	2.11	-0.02	0.08	81.34	56.96				
A1-170	250	2.41	-0.04	0.12	78.50	51.65				
A1-170	275	2.74	-0.09	0.16	73.91	45.59				
A1-170	300	3.14	-0.12	0.16	72.57	36.90				
A1-170	325	3.07	-0.11	0.38	38.52	12.22				
A1-170	350	3.04	-0.11	0.13	32.22	11.17				
A2-170	100	1.01	0.00	0.06	95.14	58.53				
A2-170	200	1.72	-0.03	0.02	84.97	58.38				
A2-170	225	2.22	0.05	0.03	82.99	55.95				
A2-170	250	2.49	0.07	0.05	79.65	51.05				
A2-170	275	2.73	0.03	0.11	72.03	42.97				
A2-170	300	2.96	0.06	0.54	47.31	18.29				
A2-170	325	1.02	-0.03	-0.11	14.32	16.64				
Discharge <i>SNR</i> .	Discharge $Q$ , point velocity $v_x$ , $v_y$ , $v_z$ ; correlation $COR$ , signal noise ratio $SNR$ .									

Table A 2.1 Results of ADV point velocity measurements (1)

Table A 2.2 Results of ADV point velocity measurements (2)

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Point	Q [l/s]	<i>v<sub>x</sub></i> [m/s]	<i>v</i> <sub>y</sub> [m/s]	<i>v</i> <sub>z</sub> [m/s]	<i>COR</i> [%]	SNR [dB]
A2-120	100	0.91	-0.02	0.04	94.41	61.04
A2-120	200	1.33	-0.06	0.01	80.51	58.41
A2-120	225	1.80	-0.02	0.01	75.75	54.81
A2-120	250	2.03	-0.01	0.02	70.64	49.41
A2-120	275	2.19	-0.01	0.04	64.59	44.68
A2-120	300	2.48	-0.03	0.05	60.11	35.33
A2-120	325	2.64	-0.03	0.03	55.11	33.83
A2-120	350	2.94	-0.05	0.02	51.83	30.02
A2-50	200	0.73	0.06	-0.04	78.43	59.11
A2-50	225	0.88	0.09	-0.03	70.27	52.99
A2-50	250	1.06	0.10	-0.04	64.69	48.82
A2-50	275	1.14	0.16	-0.03	57.30	47.85
A2-50	300	1.33	0.12	-0.05	53.94	40.32
A2-50	325	1.28	0.12	-0.03	51.14	37.69
A2-50	350	1.59	0.12	-0.06	43.59	38.41
A2-50	375	1.69	0.11	-0.07	40.67	37.21
A2-50	400	1.69	0.10	-0.06	38.07	38.41
A1-86	200	0.96	-0.03	-0.01	80.78	59.70
A1-86	225	1.32	-0.01	0.01	74.08	54.46
A1-86	250	1.57	-0.03	0.00	69.74	48.41
A1-86	325	2.00	-0.05	0.01	58.01	31.54
A1-86	350	2.31	-0.07	-0.04	51.13	25.75
A1-86	300	1.99	-0.07	0.00	61.24	34.05
A3-120	200	1.18	-0.07	0.02	78.22	61.10
A3-120	225	1.56	-0.07	0.00	73.02	56.21
A3-120	250	1.90	-0.09	-0.01	68.32	50.92
A3-120	275	2.31	-0.12	-0.07	57.27	57.75
A3-120	300	2.44	-0.12	-0.05	52.95	56.38
A3-120	325	2.34	-0.20	0.10	40.18	57.53
A3-120	350	2.52	-0.16	0.25	34.89	58.16
A3-120	375	2.35	-0.17	0.33	28.77	59.23
A3-120	400	1.87	-0.15	0.71	24.69	59.14
Discharge <i>SNR</i> .	Q, point	velocity v <sub>s</sub>	$v_x, v_y, \overline{v_z; \mathbf{C}}$	orrelation	COR, signal	noise ratio

Point	Q [l/s]	<i>v<sub>x</sub></i> [m/s]	<i>v</i> <sub>y</sub> [m/s]	<i>v<sub>z</sub></i> [m/s]	<i>COR</i> [%]	SNR [dB]				
A3-82	100	0.76	-0.01	-0.02	89.00	60.66				
A3-82	200	0.90	-0.06	0.00	79.04	60.85				
A3-82	225	1.26	-0.06	-0.02	71.74	58.62				
A3-82	250	1.52	-0.07	-0.03	66.02	55.16				
A3-82	275	1.96	-0.07	-0.07	54.24	58.95				
A3-82	300	2.14	-0.12	-0.10	52.32	55.40				
A3-82	325	2.32	-0.07	-0.08	44.10	57.44				
A3-82	350	2.44	-0.08	-0.11	41.22	57.83				
A3-82	375	2.60	-0.05	-0.03	32.78	61.64				
A3-82	400	2.76	-0.05	-0.04	30.10	62.27				
A3-170	100	0.99	-0.08	0.03	91.95	60.15				
A3-170	200	1.59	-0.18	0.08	79.49	59.90				
A3-170	225	2.01	-0.19	0.06	75.84	58.56				
A3-170	250	2.36	-0.23	0.06	72.15	54.27				
A3-170	275	2.31	-0.44	-0.01	47.55	51.06				
A3-170	300	2.44	-0.43	0.14	39.32	50.41				
A3-170	325	1.51	-0.45	0.15	29.31	49.11				
A3-170	350	1.36	-0.39	0.14	25.72	48.91				
A4-170	100	0.70	-0.02	0.01	87.93	59.91				
A4-170	200	0.87	-0.07	-0.01	75.06	58.87				
A4-170	225	0.90	-0.07	-0.06	67.68	56.76				
A4-170	250	1.17	-0.07	-0.20	53.57	53.30				
A4-170	275	1.16	0.01	-0.48	39.40	48.29				
A4-170	300	1.10	0.06	-0.35	17.18	43.76				
A4-170	325	1.09	0.10	-0.63	24.49	41.95				
Discharge SNR.	Discharge $Q$ , point velocity $v_x$ , $v_y$ , $v_z$ ; correlation $COR$ , signal noise ratio $SNR$ .									

Table A 2.3 Results of ADV point velocity measurements (3)

Point	Q [l/s]	<i>v<sub>x</sub></i> [m/s]	<i>v</i> <sub>y</sub> [m/s]	<i>v</i> <sub>z</sub> [m/s]	<i>COR</i> [%]	SNR [dB]				
A5-131	100	0.70	-0.03	0.04	91.07	62.14				
A5-131	200	0.81	-0.04	0.08	78.51	60.30				
A5-131	225	0.95	-0.02	0.10	73.24	57.74				
A5-131	250	1.12	-0.01	0.14	65.53	55.91				
A5-131	275	1.30	0.00	0.17	54.78	61.28				
A5-131	300	1.36	0.00	0.32	45.95	60.84				
A5-131	325	1.38	0.00	0.44	39.12	61.83				
A5-131	350	1.41	-0.01	0.62	30.78	62.35				
A6-175	200	0.78	-0.08	0.06	78.23	61.95				
A6-175	225	0.70	-0.05	0.04	74.13	60.04				
A6-175	250	0.82	-0.03	0.04	65.07	60.94				
A6-175	275	1.04	-0.03	0.10	52.67	62.67				
A6-175	300	1.24	-0.04	0.13	41.49	63.51				
A6-175	325	1.27	-0.03	0.19	18.20	63.43				
A6-175	350	1.40	-0.05	0.25	24.20	63.79				
A6-175	375	1.28	-0.04	0.42	18.17	63.38				
A6-175	400	1.04	-0.03	0.52	15.18	62.35				
A3b-86	200	1.21	-0.13	-0.02	77.12	61.10				
A3b-86	225	1.71	-0.16	-0.05	72.39	56.00				
A3b-86	250	1.99	-0.18	-0.07	67.26	49.76				
A3b-86	300	2.54	-0.21	-0.12	53.83	43.95				
A3b-86	325	2.81	-0.19	0.11	44.94	50.37				
A3b-86	350	2.85	-0.20	-0.01	42.60	43.30				
Discharge <i>SNR</i> .	Discharge $Q$ , point velocity $v_x$ , $v_y$ , $v_z$ ; correlation $COR$ , signal noise ratio $SNR$									

Table A 2.4 Results of ADV point velocity measurements (4)

Point	Q [l/s]	<i>v<sub>x</sub></i> [m/s]
P1-170	200	1.80
P1-170	225	2.34
P1-86	200	1.26
P1-86	225	1.76
P1-86	250	1.72
P1-86	250	1.64
P1-86	300	2.20
P2-170	200	1.97
P2-170	225	2.40
P2-86	200	1.34
P2-86	225	1.61
P2-86	250	2.11
P2-86	250	1.84
P2-86	300	2.25
P3-170	200	1.80
P3-170	225	2.16
P3-170	250	2.46
P3-86	200	1.05
P3-86	225	1.50
P3-86	250	1.78
P3-86	250	1.77
P3-86	300	2.17
P4-153	200	1.14
P4-153	225	1.23
P4-153	250	1.63
P4-153	250	1.55
P4-153	300	1.95
Discharge	Q, point	velocity $v_{x}$ .

Table A 2.5 Results of Propeller point velocity measurements

Method	Profile	Q [l/s]	$v_x$ [m/s]	<i>v<sub>p</sub></i> [m/s]	<i>v<sub>u</sub></i> [m/s]	<i>v</i> <sub>s</sub> [m/s]	<i>v<sub>x,stone</sub></i> [m/s]
ADV	1	200	1.12	1.09	1.39	1.53	0.65
ADV	1	225	1.55	1.46	1.57	1.99	0.95
ADV	1	250	1.78	1.68	1.77	2.23	1.04
ADV	1	275	1.92	1.86	1.97	2.52	1.26
ADV	1	300	2.18	2.07	2.15	2.68	1.45
ADV	2	200	1.25	1.09	1.39	1.53	0.73
ADV	2	225	1.67	1.46	1.57	1.99	0.88
ADV	2	250	1.88	1.68	1.77	2.23	1.06
ADV	2	275	2.01	1.86	1.97	2.52	1.14
ADV	2	300	2.28	2.07	2.15	2.68	1.33
ADV	3	200	1.11	1.09	1.39	1.53	0.90
ADV	3	225	1.49	1.46	1.57	1.99	1.26
ADV	3	250	1.79	1.68	1.77	2.23	1.52
ADV	3	275	2.20	1.86	1.97	2.52	1.96
ADV	3	300	2.34	2.07	2.15	2.68	2.14
ADV	3b	200	1.21	1.09	1.39	1.53	-
ADV	3b	225	1.71	1.46	1.57	1.99	-
ADV	3b	250	1.99	1.68	1.77	2.23	-
ADV	3b	275	2.20	1.86	1.97	2.52	1.96
ADV	3b	300	2.54	2.07	2.15	2.68	-
ADV	5	200	0.81	1.09	1.39	1.53	0.81
ADV	5	225	0.95	1.46	1.57	1.99	0.95
ADV	5	250	1.12	1.68	1.77	2.23	1.12
ADV	5	275	1.30	1.86	1.97	2.52	1.30
ADV	5	300	1.10	2.07	2.15	2.68	1.36
ADV	6	200	0.78	1.09	1.39	1.53	0.78
ADV	6	225	0.70	1.46	1.57	1.99	0.70
ADV	6	250	0.82	1.68	1.77	2.23	0.82
ADV	6	275	1.04	1.86	1.97	2.52	1.04
ADV	6	300	1.24	2.07	2.15	2.68	1.24

Table A 2.6 Results of point and mean velocity obtained from different methods (ADV)

Discharge Q, point velocity  $v_x$ , mean velocity obtained from point velocity measurements  $v_p$ , mean velocity obtained from water level measurements (i.e. ultrasonic sensors)  $v_u$ , mean velocity obtained from surgace velocity measurements  $v_s$ , point velocity on the stones  $v_{x,stone}$ .

Method	Profile	Q [l/s]	$v_x$ [m/s]	$v_p$ [m/s]	$v_u$ [m/s]	$v_s$ [m/s]	$v_{x,stone}$ [m/s]
Propeller	1	200	1.42	1.37	1.39	1.53	-
Propeller	1	225	1.93	1.77	1.57	1.99	-
Propeller	1	250	1.68	1.77	1.77	2.23	-
Propeller	1	300	2.20	2.17	2.15	2.68	-
Propeller	2	200	1.53	1.37	1.39	1.53	-
Propeller	2	225	1.84	1.77	1.57	1.99	-
Propeller	2	250	1.98	1.77	1.77	2.23	-
Propeller	2	300	2.25	2.17	2.15	2.68	-
Propeller	3	200	1.27	1.37	1.39	1.53	-
Propeller	3	225	1.69	1.77	1.57	1.99	-
Propeller	3	250	1.97	1.77	1.77	2.23	-
Propeller	3	300	2.17	2.17	2.15	2.68	-
Propeller	4	200	1.14	1.37	1.39	1.53	-
Propeller	4	225	1.23	1.77	1.57	1.99	-
Propeller	4	250	1.59	1.77	1.77	2.23	-
Propeller	4	300	1.95	2.17	2.15	2.68	-

Table A 2.7 Results of point and mean velocity obtained from different methods (Propeller)

Discharge Q, point velocity  $v_x$ , mean velocity obtained from point velocity measurements  $v_p$ , mean velocity obtained from water level measurements (i.e. ultrasonic sensors)  $v_u$ , mean velocity obtained from surgace velocity measurements  $v_s$ , point velocity on the stones  $v_{x,stone}$ .
## APPENDIX 3 STONES COUNTING

		Nu	mber of le stones	eaving			Nu	aving	
exp	Q [l/s]	Bed	Bank	Total	exp	Q [l/s]	Bed	Bank	Total
D01	200	-	-	4	T01	375	5	0	5
D01	225	-	-	8	T01	400	1	0	1
D01	250	-	-	18	T01	425	4	3	7
D01	275	-	-	31	T01	450	29	1	30
D01	300	-	-	>30	T02	350	0	1	1
D02	200	3	2	5	T02	375	0	0	0
D02	225	3	1	4	T02	400	1	0	1
D02	250	23	5	28	T02	425	1	0	1
D02	275	27	24	51	T02	450	0	0	0
D02	300	74	42	116	T02	475	2	0	2
D02	325	4	0	4	T03	325	0	1	1
D03	200	7	0	0	T03	350	1	0	1
D03	225	15	2	17	T03	375	1	1	2
D03	250	61	28	89	T03	400	1	1	2
D03	300	25	7	32	T03	425	15	3	18
D03	325	2	0	2	T03	450	41	18	59
D04	200	3	0	3	T04	300	1	0	1
D04	225	4	3	7	T04	325	0	1	1
D04	250	45	45	90	T04	350	2	1	3
D04	275	66	16	82	T04	375	0	0	0
P01	200	9	2	11	T04	400	2	1	3
P01	225	1	0	1	T04	425	8	10	18
P01	250	4	3	7	T04	450	8	36	44
P01	275	18	54	72	T05	350	2	0	2
P02	200	1	1	2	T05	375	1	0	1
P02	225	7	1	8	T05	400	1	0	1
P02	250	4	7	11	T05	425	1	1	2
P03	200	2	45	47	T05	450	33	2	35
P03	225	0	0	0					
P03	250	2	1	3	1				
P03	275	5	5	10	1				
P04	300	21	11	32	1				
P04	200	0	0	0					
P04	225	2	1	3					
P04	250	4	1	5					
P04	275	14	5	19					

Table A 3.1 Number of stones leaving test section for riprap stability experiments

## DateOur reference15.05.2020Leif Lia

## APPENDIX 4 COMPARISON WITH ROBINSON'S FORMULA

Table A 4.1 Analogy with steep slope and	d comparison with Robinson's form	ula for upper downstream ultrasonic	sensor (21)
	1	11	

exp	Q [l/s]	<i>Z21</i> [mm]	<i>v</i> <sub>21</sub> [m/s]	<i>Fr</i> <sub>21</sub> [-]	<i>b<sub>ef,21</sub></i> [m]	<i>q</i> <sub>21</sub> [m <sup>3</sup> /s/m]	Frs <sub>21</sub> [-]	\$21 [%]	n [-]	A <sub>21</sub> [m <sup>2</sup> ]	<i>O</i> <sub>21</sub> [m]	<i>R</i> 21 [m]	v <sub>slope,21</sub> [m/s]	Q <sub>slope,21</sub> [m <sup>3</sup> /s]	Fr <sub>slope,21</sub> [-]	d <sub>50rob,21</sub> [mm]
D01	297.89	182.99	2.40	1.96	0.679	0.44	10.30	11.2	0.038	0.12	0.871	0.143	2.40	0.298	1.96	164
D02	299.12	185.84	2.36	1.92	0.681	0.44	10.31	10.4	0.038	0.13	0.877	0.144	2.36	0.299	1.92	160
D03	274.70	182.78	2.21	1.81	0.679	0.40	9.50	8.9	0.037	0.12	0.871	0.142	2.21	0.274	1.81	137
D04	275.19	195.18	2.05	1.63	0.688	0.40	9.39	6.5	0.035	0.13	0.893	0.150	2.05	0.275	1.63	107
P01	273.60	191.46	2.09	1.67	0.685	0.40	9.37	7.1	0.036	0.13	0.887	0.148	2.09	0.274	1.68	114
P02	269.28	212.65	1.81	1.39	0.701	0.38	9.01	4.0	0.033	0.15	0.925	0.161	1.81	0.270	1.39	71
P03	250.47	211.84	1.69	1.30	0.700	0.36	8.39	3.3	0.032	0.15	0.923	0.161	1.69	0.251	1.30	59
P04	274.63	210.78	1.86	1.43	0.700	0.39	9.21	4.4	0.033	0.15	0.921	0.160	1.86	0.274	1.43	77
T01	449.80	199.94	3.25	2.56	0.691	0.65	15.26	23.1	0.042	0.14	0.902	0.153	3.25	0.449	2.56	253
T02	450.77	198.51	3.29	2.60	0.690	0.65	15.32	24.2	0.043	0.14	0.899	0.152	3.29	0.451	2.60	257
T03	447.59	193.41	3.37	2.69	0.687	0.65	15.30	27.0	0.043	0.13	0.890	0.149	3.37	0.447	2.69	265
T04	450.15	203.47	3.19	2.49	0.694	0.65	15.22	21.3	0.042	0.14	0.908	0.155	3.19	0.451	2.49	246
T05	450.34	183.97	3.60	2.94	0.679	0.66	15.55	35.1	0.045	0.13	0.873	0.143	3.60	0.450	2.94	291

Discharge Q, water depth z, mean velocity v, Froude number Fr, effective width of the channel  $b_{ef}$ , unit discharge q, longitudinal slope s, Manning's roughness coefficient n, flow area A, wetted perimeter O, hydraulic radius R, median stone diameter designed according to Robinson's rormula  $d_{50rob}$ .

#### Norwegian University of Science and Technology

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exp	Q [l/s]	y22 [mm]	<i>v</i> <sub>22</sub> [m/s]	Fr <sub>22</sub> [-]	<i>b<sub>ef,22</sub></i> [m]	<i>q</i> <sub>22</sub> [m <sup>3</sup> /s/m]	Frs <sub>22</sub> [-]	\$22 [%]	n [-]	A <sub>22</sub> [m <sup>2</sup> ]	<i>O</i> <sub>22</sub> [m]	<i>R</i> 22 [m]	Vslope,21 [m/s]	Q <sub>slope,22</sub> [m <sup>3</sup> /s]	Fr <sub>slope,22</sub> [-]	<i>d</i> <sub>50rob,22</sub> [mm]
D01	297.89	190.90	2.28	1.83	0.685	0.44	10.21	9.1	0.037	0.13	0.886	0.148	2.28	0.298	1.83	145
D02	299.12	193.52	2.25	1.80	0.687	0.44	10.22	8.6	0.037	0.13	0.890	0.149	2.25	0.299	1.80	139
D03	274.70	191.13	2.10	1.69	0.685	0.40	9.41	7.2	0.036	0.13	0.886	0.148	2.10	0.275	1.69	115
D04	275.19	-	-	-	-	-	_	-	-	-	-	_	-	-	-	-
P01	273.60	193.91	2.05	1.64	0.687	0.40	9.34	6.6	0.035	0.13	0.891	0.149	2.05	0.273	1.64	107
P02	269.28	209.44	1.84	1.42	0.699	0.39	9.04	4.3	0.033	0.15	0.919	0.159	1.84	0.269	1.42	75
P03	250.47	198.76	1.82	1.44	0.691	0.36	8.51	4.5	0.033	0.14	0.900	0.153	1.82	0.250	1.44	76
P04	274.63	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
T01	449.80	222.94	2.85	2.14	0.709	0.63	14.89	13.4	0.039	0.16	0.943	0.167	2.85	0.450	2.14	211
T02	450.77	220.56	2.89	2.18	0.707	0.64	14.96	14.3	0.039	0.16	0.939	0.166	2.89	0.451	2.18	215
T03	447.59	219.10	2.89	2.19	0.706	0.63	14.88	14.5	0.040	0.15	0.936	0.165	2.89	0.448	2.19	216
T04	450.15	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
T05	450.34	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Table A 4.2 Analogy with steep slope and comparison with Robinson's formula for upper downstream ultrasonic sensor (22)

Discharge Q, water depth z, mean velocity v, Froude number Fr, effective width of the channel  $b_{ef}$ , unit discharge q, longitudinal slope s, Manning's roughness coefficient n, flow area A, wetted perimeter O, hydraulic radius R, median stone diameter designed according to Robinson's rormula  $d_{50rob}$ .

$$Frs = \frac{q}{(gd_{50}^3)^{0.5}}$$

where q is unit discharge,  $d_{50}$  median stone diameter and g is the gravitational acceleration.

## STEINDIAMETER I ELVEPLASTRING (STONE DIAMETER IN RIVER RIPRAP)

## LITERATURE REVIEW

## SUMMARY

The literature review is divided into three chapters. The first one shows publications investigating erosion protection on the river bed and also on the river banks. The second chapter describes publications related to overtopping and river bed riprap design and the third chapter focused on the impulse concept of bed particle entrainment in turbulent flow.

Most of the experiments are related to bed or overtopping embankment riprap. And any experiments on the riverbank protection in steep rivers in the river bend have not been found.

The experimental procedure to determine size of the riprap protection is quite similar along the experiments. The riprap is constructed in the flume and then the discharge is slowly increased until the failure of riprap happens. In this moment, the variables are measured and the data are used for the result analysis. The failure criteria could be subjective and there are different qualitative methods along the experiments.

Measured variables in the experiments usually discharge, mean velocity or local velocity, water surface elevation and observation of the riprap failure.

According to evaluation of overtopping riprap design relationships by (Abt et al.,2013), There are relationships that adequately predict median stone size for ripraps in slope from 2 to 50 %, stone sizes ranging from 15 to 254 mm, and stone layer thicknesses of  $2xD_{50}$  or greater.

## **BRIEF DESCRIPTION OF THE LITERATURE**

## (S. R. Abt & Johnson, 1991) Riprap design for overtopping flow

Stability of riprap on steep slope was explored on the laboratory experiment.

(May & Escarameia, 1992), (May & Escarameia, 1995) Channel protection: Turbulence downstream of structures

Laboratory experiment in the horizontal flume with sluice gate to create turbulent flow was performed.

(Froehlich & Benson, 1996) Sizing dumped rock riprap

Dumped rock riprap on the river bed and banks was investigated on the laboratory experiment with unknown slope.

## (K. M. Robinson, C. E. Rice, & K. C. Kadavy, 1998) Design of rock chutes

Laboratory experiments were performed on the bed and banks of rock chutes.

# (Peirson, Figlus, Pells, & Cox, 2008) Placed Rock as protection against erosion by flow down steep slopes

Stability of placed and dumped riprap was compared based on results from laboratory experiment.

(Ahmet O. Celik, Diplas, Dancey, & Valyrakis, 2010), (Ahmet Ozan Celik, Diplas, & Dancey, 2013), (Ahmet O Celik, Diplas, & Dancey, 2014) **Impulse and particle dislodgement under turbulent flow** conditions

Quantification of the hydrodynamic forces on exposed spherical particle was experimentally investigated.

(B. S. R. Abt, Thornton, Scholl, & Bender, 2013) Evaluation overtopping riprap design relationships

21 stone-sizing relationships were evaluated against the dataset with results of 96 experiments.

### (Falkenberg, 2013) Erosjossikring av elvebunn nedstrøms dammer fundamentert på løsmasser

Laboratory experiments on bed riprap downstream a sluice gate.

### (Langmaak & Basson, 2015) Incipient motion of riprap on steep slopes

The applicability of Liu's theory was investigated experimentally on steep riprap slope.

(Jafarnejad, Franca, Pfister, & Schleiss, 2017), (Jafarnejad, Franca, Pfister, & Schleiss, 2018) **Timebased failure analysis of compressed riverbank riprap** 

An empirical prediction of riprap time to failure is developed. The experiment on riverbank riprap on steep slope was performed.

## (Dey & Ali, 2018) Review Article: Advances in modeling of bed particle entrainment sheared by turbulent flow

The paper reviews mathematical models of the bed particle entrainment caused by turbulent wall-shear flow.

#### (Fernández, 2019) Experimental characterization of turbulence in steep rough streams-PhD thesis

The thesis is focused on the description of turbulent flow and incipient motion of the particles in steep slopes.

# $(Marin-Esteve,\ 2019)\ \textbf{Theoretic-experimental study on the morphological behavior of rivers with steep \ slopes-ongoing\ PhD$

The PhD candidate investigates theoretically and experimentally threshold of the particles motion in steep slopes.

## DESIGN OF THE POTENTIAL EXPERIMENT

Based on the literature review, the future research should be focused more on the stability of river banks than river bed. According to (B. S. R. Abt et al., 2013), there are already existing relationships, which can be used to design the river bed riprap on steep slopes.

As for the flume geometry, highly turbulent flow could be created by increasing longitudinal slope of the flume or including sluice gate upstream the test area of the flume. The dimensions of the flume depend on the limitations of the laboratory, where the experiment will be performed. An analogy between flow in steep slope and flow downstream a sluice gate can be expressed by Froude number.

To monitor the fluctuations of hydrodynamic forces on the particles, the point velocity should be measured on the river bed and along the bank. Failure of the riprap and the monitoring methodology need to be specified. Exposing of the filter could be used as a failure criterion, but also incipient motion of the particles should be monitored. Then can be evaluated if the movement of single particles leads to the riprap failure.

## **RIVERBANKS RIPRAP**

## TIME-BASED FAILURE ANALYSIS OF COMPRESSED RIVERBANK RIPRAP

(Jafarnejad et al., 2017); Laboratory of Hydraulics Constructions of École Polytechnique Fédérale de Lausanne, Switzerland

The authors experimentally investigated the stability of well-positioned riverbank riprap. An empirical prediction of the riprap time to failure is developed based on time-based analysis of the results. The prediction could be used in the range of applications corresponding to the experimental set-up of this study. The results show that the time of the failure depends on longitudinal slope and the diameter of riprap stones.

**Experimental set-up:** straight tilting flume with the single riverbank (Figure 1), 10x1.5x0.5 m (LxBxH), longitudinal slope 1.5;3;5.5 %, bank slope 1:1.43 (V:H).

Riprap material: uniform crushed limestone D<sub>50</sub>= 37;42;47 mm.

Measured variables: discharge, mean velocity (Q/A), erosion (tracking and counting of eroded blocks).

**Riprap failure:** "The total collapse of all blocks in a section over the whole bank is considered as a failure, thus the failure mechanism considered herein is sliding. This failure type is due to the slumping or sliding of riprap from the top to the toe of the bank, causing full exposure of the filter to the flowing water. This causes bank instability and consequently a downstream or upstream continuation of the riprap failure and a collapse of the bank." The discharge was increased subsequently until the failure of riprap happened. An example of the failure can be seen in Figure 2.

The dimensionless time to failure depends on the friction velocity ( $u^*$ ), time of the failure ( $t_f$ ) and the block size diameter (D) or the water depth (h).

Influence of the second layer of riprap stones was investigated on similar experimental set-up (Jafarnejad et al., 2018).





Figure 1 Photo of the experimental flume, streamwise view

Figure 2 Example of riprap before (a) and after failure

#### CHANNEL PROTECTION: TURBULENCE DOWNSTREAM OF STRUCTURES

(May & Escarameia, 1995), (May & Escarameia, 1992), HR Wallingford, UK

Lab experiment on stability of riprap and concrete blocks for river bed and bank protection in high turbulent environments were carried out. Sluice gate in the horizontal flume was used to create the uniform turbulent flow. An equation (developed from Izbash equation) for sizing riprap under normal and high turbulence conditions (downstream of a hydraulic jump) was obtained from the results. The equation can be applied on the flat bed and riverbanks and takes into account the turbulence level. The range of turbulent intensity (TI) is 5 - 30 %. Correlation between Froude number and turbulence intensity was also investigated.

**Experimental set-up:** straight rectangular horizontal flume with sluice gate 28 x 1.21x 0.6 m, 2.6 m long test section, bank slopes 1:2, 1:2.5 (V:H). Figure 3 and Figure 4 show the examples of the experiments.

**Riprap material:** angular stones  $D_{50}$ =4.6 - 11.8 mm; round stones  $D_{50}$ = 7.3 – 9.3 mm.

**Measured variables:** discharge, water surface level (simple scale upstream the sluice gate, micrometer screw point gauge downstream the test section), point values of instantaneous flow velocity in the test section (xyz, ultrasonic current meter).

**Threshold of movement criteria:** The rectangular area was marked in the test section. "The threshold of movement was reached when a fixed number of stones would roll on the marked area during a fixed period of time." The velocity measurements were made, once the flow conditions stabilized and the threshold was reached.



Figure 3 Experiment on riprap on the bed



Figure 4 Experiment with concrete blocks on the bank slope

## **DESIGN OF ROCK CHUTES**

(K. M. Robinson et al., 1998), USA

Lab experiments on stability of rock chutes were performed. The tests were carried out on the bed (slope 2-40%) and on the banks with side slope 2:1(slope 2-6%). The experiments on two full size prototype structures were also conducted.

The equation was developed to design the particle size of riprap. The equation is relating the highest stable discharge to median stone size and the bed slope.

An empirical relationship to predict Manning roughness coefficient based on the median stone size and the slope was also developed.

**Experimental set-up:** 3 separate straight rectangular flumes: width = 0.76, 1.07, 1.83 m, prototype structures: width = 2,74 m, side slopes 2:1, ; longitudinal slope 2-40%; tests with the side slopes 2-6%;

**Riprap material:** angular crushed limestone D<sub>50</sub>=15 - 278 mm.

Riprap failure: "Failure was defined as the flow condition that exposed the underlying geofabric or bedding material."

Measured variables: discharge (manometers, Parshall flumes), failure of the chute (observation).

## **OVERTOPPING AND RIVER BED RIPRAP**

## EROSJONSSIKRING AV ELVEBUNN NEDSTRØMS DAMMER FUNDAMENTERT PÅ LØSMASSER (EROSION PROTECTION OF THE RIVER BED DOWNSTREAM THE DAMS) – MASTER THESIS

(Falkenberg, 2013)

The thesis is focused on the scour of river revetments in supercritical flow. Both dumped and placed riprap was tested separately in the horizontal flume. The sluice gate was installed to the flume to create supercritical flow. The main goal was to investigate the functionality of the erosion protection of Checras dam in Peru. The author concluded that erosion protection was stable for all combinations of gate openings at the highest regulated water level. Gate operation curve is presented as a result of the measurements.

**Experimental set-up:** Straight horizontal flume 25x1.0x2.0 m (Figure 5). Sluice gate to create the turbulent flow (Figure 6).

Riprap material: crushed rock D=45 mm.

**Riprap failure:** The failure is defined as an exposure of the filter below the riprap stones.

Measured variables: discharge, water surface elevation, riprap failure (observation).



Figure 5 Flume geometry scheme



Figure 6 Example of the experiment, Q=495 l/s.

## EVALUATION OF OVERTOPPING RIPRAP DESIGN RELATIONSHIPS

(Abt et al., 2013), CSU, USA

The authors evaluated 21 stone-sizing relationships to determine the median stone size of a riprap layer. The relationships were evaluated against the dataset with 96 discrete data points (results of 10 experiments). The compared parameters are discharge at failure, median stone size and bed slope.

"The findings presented herein indicate that for slopes ranging from 2 to 50%, stone sizes ranging from 1.5 to 25.4 cm, and stone layer thicknesses of 2xD50 or greater, there currently exists a relationship(s) that adequately predict(s) median stone sizes for a breadth of uses and applications."

## **RIPRAP DESIGN FOR OVERTOPPING FLOW**

(S. R. Abt & Johnson, 1991), CSU, USA

The experimental studies of overflowed riprap protected embankments were conducted. A riprap design relationship was developed to sizing the riprap stone on the basis of unit discharge and embankment slope. The experiments were carried out in two flumes.

**Experimental set-up – outdoor flume:** straight rectangular flume, ca. 55 x 6.1 x 2.4 m, longitudinal slope 10-20% (Figure 7).

**Experimental set-up – indoor flume:** straight rectangular flume, ca. 61 x 2.4 x 1.2 m, longitudinal slope 2-10%.

**Riprap material:** limestone D<sub>50</sub>=from 25.9 mm to 157.5 mm.

**Measured variables:** discharge, water surface elevation (manometers), flow velocity (pitot tube, magnetic flowmeter).

**Riprap failure:** "The failure criterion of the riprap layer was when the filter blanket, or more often, the geofabric, was exposed."



Figure 7 Test facility

### PLACED ROCK AS PROTECTION AGAINST EROSION BY FLOW DOWN STEEP SLOPES

(Peirson et al., 2008), Australia

The authors investigated the influence of placing rock instead of dumping randomly on the riprap stability on the lab experiment. The flow through and down two layers of rock armor on steep embankment was also explored.

The conclusion is that placing rock increased failure flow by 30% of that achieved with the same type of randomly dumped material but the total armor mass per unit surface area increased by 35%.

Experimental set-up: straight rectangular flume 4.2 – 8.4 x 0.9 x 0.6 m, longitudinal slope 20, 30, 40 %,

**Rock material:** sandstone and basalt D<sub>50</sub>=76, 94, 109 mm.

Measured variables: discharge, water surface elevation (manometers), motion of the rocks (observation).

Riprap failure: Three values of discharge were measured:

- Initial displacement of a single stone anywhere on the test surface.
- Significant rock motion, defined as displacement of five rocks over a distance of more than 5 diameters.
- Armor failure, that is, exposure of the filter layer.

#### **INCIPIENT MOTION OF RIPRAP ON STEEP SLOPES**

(Langmaak & Basson, 2015); University of Stellenbosch, South Africa

The paper investigated the applicability of Liu's theory (estimation of the incipient motion point of sediments) on steep riprap slope. The theory states that for large particles the movability number (defined as the ratio of the shear velocity to the settling velocity), is constant (assumption of the flat bed slope and uniform flow). The results of the experiments indicate, that Liu's theory can be a suitable tool to estimate minimum riprap size, even slopes up to 40 %. The author proposed value of 0.18 for movability number for design purposes. All experiments were performed under uniform flow conditions.

**Experimental set-up:** straight rectangular flume 30.0 x 1.0 x 1.25 m, 1,6 (1,0) m long test area, longitudinal slope 20, 30, 40 %. Side view of the flume can be seen in

Figure 8.

Rip rap material: angular rocks  $D_{50}$ = 67, 100 mm.

**Measured variables:** discharge, water surface level, bed level, incipient motion of the gravel (observation, video).

**Riprap failure:** "The discharge was then slowly increased at a rate of 5 l/s per minute, while carefully observing the structure from the side of the flume, until particles started dislodging that were of sufficient size to locally provoke instability of the riprap layer (video recordings were used to verify this afterwards), without actually causing global failure of the structure. At that instant, water levels were measured, after which the water supply was shut off."



Figure 8 Side view of the experimental flume

## SIZING DUMPED ROCK RIPRAP

(Froehlich & Benson, 1996), Kentucky, USA

Lab experiment on the evaluation of the resistance of riprap rocks to hydrodynamic forces in uniform flow conditions was carried out in this paper. The probabilistic model of particle resistance using Weibull distribution is developed to determine the size of loosely dumped rock riprap (river bed and banks).

Experimental set-up: straight rectangular flume, ca. 2.5 x 0.15 m; increasing slope (not mentioned).

Riprap material: rounded and subrounded particles (D=1.85 to 15.88 mm).

Measured variables: discharge; number of displaced particles (recording).

## **IMPULSE CONCEPT**

## EXPERIMENTAL CHARACTERIZATION OF TURBULENCE IN STEEP ROUGH STREAMS– PHD THESIS

(Fernández, 2019); UPC, Barcelona

PhD thesis is focused on the description of the development of turbulence in highly turbulent flows in steep streams. The stochastic method to determine threshold of motion (includes also fluctuation, does not use only time-averaged values of variables) were investigated and the simple conceptual model of incipient motion was developed. Impulse concept was followed. The impulse is product of magnitude and duration of the force on the particle. All experiments were performed under uniform flow conditions.

**Experimental set-up:** straight horizontal flume with the trapezoidal cross section 14.0 x 1.2 x 1.0 m, 1.5 m long test area (Figure 10), longitudinal slope 3%, bank slope 1:1 (V:H).

**Characteristics of the material:** rounded gravel  $D_{50}$ = 55 mm, crushed gravel  $D_{50}$ = 17.82; 30.02; 51.08 mm.

**Measured variables:** discharge, water depth (piezometers), velocity (ADV), incipient motion of the gravel (observation, painted particles).

**Threshold of movement:** "It is important to determine the fraction of particles (%) that move out for each flow rate, so it is decided to discretize the test area by painting the surface of the stones, and placing a particle trap downstream of the test area. The criterion established to start taking the velocity is based on considering the incipient motion discharge as the one that sets in motion at least a 2% of the particles placed in the test area during the 10 minutes of application. Once this discharge is established, velocity profiles and hydraulic data is taken."

The conceptual model (Figure 9) uses simple arguments from mechanics to insight the bursting cycle and incipient motion. "The conceptual model is extremely simple and does not consider additional forces of holding between particles." The model could not be evaluated on the lab experiment in terms of incipient motion, i.e. recorded videos had not enough resolution to detect at which velocity the particles started to move (also unsynchronized video and ADV measurements and a big study area).



Figure 9 Conceptual model



Figure 10 test area in the experimental flume

# THEORETIC-EXPERIMENTAL STUDY ON THE MORPHOLOGICAL BEHAVIOUR OF RIVERS WITH STEEP SLOPES –ONGOING PHD

(Marín-Esteve, 2019)

PhD candidate from UPC in Barcelona, she continues on work of (Fernández, 2019).

The aim of her PhD is to improve the knowledge of the steep rivers, trying to know how they behave. So far, she is focused on the threshold of motion equation in steep slopes (development of the theoretical formulation, which will be tested experimentally).

## CELIK, A.O. – PUBLICATIONS ON IMPULSE AND PARTICLE DISLODGEMENT UNDER TURBULENT FLOW CONDITIONS

(Ahmet O. Celik et al., 2010), (Ahmet Ozan Celik et al., 2013), (Ahmet O Celik et al., 2014), USA

Initial movement of an exposed spherical particle in rapidly fluctuating hydrodynamic forces was explored. In the first experiment (Ahmet O. Celik et al., 2010), the local flow velocity was measured one diameter upstream the particle. In the next experiment (Ahmet Ozan Celik et al., 2013), (Ahmet O Celik et al., 2014), the pressures were measured directly on four points on the particle. Unsteady wake-flow conditions are also included by using horizontal cylinder spanning across the channel upstream the test area.

The publications show that the impulse is the more propriet parameter to describe the role of the turbulence fluctuations on the incipient movement of the particle.

The comparison of determining instantaneous hydrodynamic forces using velocity and pressure measurement supports the use of velocity measurement. Measuring of the pressure fluctuations directly is more accurate, but far more demanding to use in field and lab experiments.

Experimental set-up: straight rectangular flume 14 x 0.6 m, longitudinal slope 0.25 %.

**Characteristics of the material:** Teflon spherical particles d = 12.7 mm (Figure 11).

**Measured variables:** discharge, local flow velocity (LDV), recording of the entrainment of the particle (laser-based system), pressure in four points of the particle surface (pressure transducers).



Figure 11 Side view of the experimental set-up

# **REVIEW ARTICLE: ADVANCES IN MODELING OF BED PARTICLE ENTRAINMENT SHEARED BY TURBULENT FLOW**

#### (DEY & ALI, 2018)

The paper reviews mathematical models of the bed particle entrainment caused by turbulent wall-shear flow. It contains the description of impulse concept.

Publications focused on the impulse concept use the approach that the magnitude of hydrodynamic force on particles is necessary, but not sufficient to determine the threshold of motion. It needs to be supported also by the duration of applied force.

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# STABILITY OF RIVERBANK RIPRAP IN SUPERCRITICAL FLOW CONDITIONS – EFFECT OF STONE PLACEMENT

#### PROJECT SUMMARY

The goal of the experiment is to investigate stability of placed and dumped riverbank riprap in the supercritical flow and the comparison of the results with existing riprap design formulas. The conceptual scale of the model is 1:10. A sluice gate in 1 m wide horizontal flume was used to generate supercritical flow. Froude number was used as a common parameter for flow in steep rivers and flow in the horizontal flume downstream a sluice gate. Riverbank (i.e. half trapezoidal cross section) with side slope 1:1.5 (33.7°) was built upstream and downstream the gate using a wooden structure. The riprap consisted of uniform stones of  $d_{50}$ =0.057 m and the filter layer below riprap was built with uniform stones of  $d_{50}$ =0.020 m. Three types of riprap configuration were tested: i) dumped riprap ('rauset steinsikring'), ii) placed riprap parallel ('flatplastring', the longest axis of the stones is parallel to the flow direction), iii) placed riprap inclined ('damplastring', the longest axis of the stones is towards the side slope/bed).

The discharge, water surface elevation, water surface velocity and point velocity were measured on the model. Water surface elevation was measured by ultrasonic sensors. Measurements of water surface velocity were carried out using table tennis balls and camera. Point velocity was measured in several points along a cross section using ADV (i.e. Acoustic Doppler Velocimetry) side probes and Propeller (i.e. current meter). In order to identify the riprap failure, video documentation with 2 cameras (i.e. side and top view) was recorded.

The experiments included preliminary tests, riprap stability experiments and point velocity measurement experiments. For each riprap stability experiment, the riprap stones were placed/dumped to the model and then the discharge was increased stepwise until the failure of riprap occurred. The reason of separation of the point velocity experiments is that both ADV and Propeller measurements are intrusive methods and can influence the flow conditions downstream the probes (i.e. effect the riprap stability).

Critical flow conditions (i.e. conditions which caused the failure) were obtained for failure initiation stage (i.e. the moment when continuous erosion of the riprap stones begins and leads to the total failure). Total failure occurs when riprap stones slide down from the bank and the filter is exposed along the whole submerged height of the bank.

From the results of critical flow conditions (i.e. discharge and Froude number) is clear that placed riprap inclined is the most stable riprap configuration and it can withstand much higher Froude number (i.e. ca. 2.1 to 2.95 with discharge of ca. 450 l/s, ca. 142 m<sup>3</sup>/s in prototype) than the other configurations. Placed riprap parallel is the least stable configuration with the range of Froude number ca. 1.3 to 1.7 with discharge of ca. 250 to 275 l/s (ca. 79 to 87 m<sup>3</sup>/s in prototype). Dumped riprap is a bit more stable than placed riprap parallel with Froude number range ca 1.6 to 2.0 with discharge of ca. 275 to 300 l/s (ca. 87 to 95 m<sup>3</sup>/s in prototype). The stones diameter in prototype (1:10) is 0.57 m.

Regarding velocity measurements, point velocities measured by ADV and Propeller give reasonable shapes of velocity profiles. However, ADV seems to underestimate the point velocity values in comparison to Propeller. Comparison of mean velocity obtained from all methods was done (i.e. ADV, Propeller, surface velocity, ultrasonic sensors). From the comparison it is clear, that use of Propeller gives the best match with mean velocity from ultrasonic sensors, ADV underestimates the values and the values obtained from surface velocity measurements are overestimated.

Evaluation using Robinson's formula for riprap design in steep rivers was performed. Stone diameter according to Robinson's formula was determined for critical flow conditions for all riprap stability

experiments. Comparison of the designed values with stone diameters used in the experiment indicates that Robinson's formula overestimates the stone diameter in all cases.

## STABILITET AV EROSJONSSIKRING MED STEIN PÅ ELVEBREDDER UNDER OVERKRITISK STRØMNING – EFFEKTEN AV STEINPLASSERING

### SAMMENDRAG

Målet med forsøkene var å undersøke stabiliteten av plassert- og dumpet stein i plastring av elvebredder i overkritisk strømning. Resultatene ble sammenlignet med eksisterende formelverk for erosjonssikring. Forsøkene ble kjørt i skala 1:10. En glideluke med understrømning i en 1,0 m bred horisontal renne ble brukt for å skape overkritisk strømning. Froude-tallet ble brukt som felles parameter for strømning i bratte elver og strømning i den horisontale renna nedstrøms luka. Elvebredden (dvs. et halvt trapesformet tverrsnitt) med sideskråning 1: 1,5 (33,7 °) ble bygget oppstrøms- og nedstrøms luka med hjelp av en trekonstruksjon. Erosjonssikringen besto av ensartede steiner på d<sub>50</sub> = 0,057 m, og filterlaget under erosjonssikringen ble bygget med ensartede steiner på d<sub>50</sub> = 0,020 m. Tre typer plastring ble testet: i) dumpet stein ('rauset steinsikring'), ii) plassert plastring parallelt ('flatplastring', den lengste aksen til steinene er parallell med strømningsretningen) og iii) plassert plastring normalt ('damplastring', steinens lengste akse mot sidehellingen/skråningen).

Vannføringen, nivået og hastigheten på vannoverflaten og punkthastigheten ble målt i modellen. Nivået på vannoverflaten ble målt ved hjelp av ultralydsensorer. Målinger av vannoverflatehastighet ble utført ved bruk av bordtennisballer og kamera. Punkthastighet ble målt i flere punkter langs et tverrsnitt ved bruk av ADV (dvs. Akustisk Doppler Velocimetry) sideprober og propell (dvs. strømmåler). To videokamera (dvs. sideveis og ovenfra) ble brukt for å identifisere når- og hvordan erosjonssikringen gikk til brudd.

Forsøkene inkluderte innledende tester, stabilitetsforsøk for erosjonssikring og målinger av punkthastighet. For hvert forsøk med erosjonssikring ble steinene plassert/dumpet i modellen. Deretter ble vannføringen økt trinnvis til erosjonssikringen sviktet. Årsaken til at punkthastighetsforsøkene ble utført separat, var at både ADV og propellmålinger kan påvirke forholdene nedstrøms probene (dvs. påvirke stabiliteten til plastringen).

*Kritiske* forhold for erosjonssikringen (dvs. forhold som forårsaker bruddet) er satt til når feilen initieres (dvs. øyeblikket hvor kontinuerlig erosjon av steinene begynner og fører til total svikt). Total svikt regnes når erosjonssikringen glir ned fra skråningen og filteret blir eksponert langs hele den nedsenkede delen av skråningen.

Resultatene for kritiske forhold for plastringen (dvs. vannføring og Froude-tall) viser tydelig at plassert plastring normalt er den mest stabile plastringen. Den tåler strømning med mye høyere Froude-tall (dvs. 2,1 – 3,0 med vannføring  $Q_{lab} = 450$  l/s, tilsvarande Q = 140 m<sup>3</sup>/s i fullskala, enn de andre utformingene. Plassert plastring parallelt er den minst stabile plastringen med Froude-tall på 1,3 – 1,7 med vannføring på  $Q_{lab} = 250 - 275$  l/s, Q = 80 - 90 m<sup>3</sup>/s i fullskala. Dumpet erosjonssikring er litt mer stabil enn plassert plastring parallelt med Froude-tall mellom 1,6 – 2,0 med  $Q_{lab} = 275 - 300$  l/s, 90 – 100 m<sup>3</sup>/s i fullskala. I fullskala (1:10) gjelder verdiene for steindiameter på d = 0.57 m.

Når det gjelder hastighetsmålinger, gir punkthastigheter målt med ADV og Propell rimelig form på hastighetsprofilene. ADV ser imidlertid ut til å undervurdere punkthastighetsverdiene i forhold til Propell. Det ble utført sammenligning av gjennomsnittshastighet oppnådd med alle metodene (dvs. ADV, Propell, overflatehastighet, ultralydsensorer). Fra sammenligningen er det klart at bruk av Propell gir best samsvar med gjennomsnittshastighet fra ultralydsensorer. ADV undervurderer vannhastigheten og målingene av hastighet på overflaten overvurderer den reelle vannhastigheten.

Evaluering av Robinsons formel for plastring i bratte elver ble utført. Steindiameter i forhold til Robinsons formel ble bestemt for kritiske strømningsforhold for alle forsøkene. Sammenligning av de beregnede

verdiene på steindiameter som ble brukt i forsøkene, indikerer at Robinsons formel gir større nødvendig steindiameter i alle tilfeller.



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